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Resistance-Displacement Relationship for Well Conductor
Fatigue Analysis**

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Development of a Novel Apparatus to Obtain a Soil Resistance-Displacement Relationship for Well Conductor Fatigue Analysis

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Abstract

An important aspect of deepwater well integrity is development of accurate conductor fatigue analysis due to cyclic loading during drilling operations. Fatigue damage in a structure occurs from stress changes in response to cyclic loading. In practice, the lateral cyclic soil response is typically modelled using Winkler p-y springs. However, recently developed soil models for conductor fatigue analysis were based on physical modelling in a geotechnical centrifuge. Notwithstanding the advantages of centrifuge modelling for investigating the conductor-soil interaction mechanism, development of simple laboratory tools to obtain p-y data directly from intact soil samples obtained from the field can also be very beneficial. This paper describes the development of a novel apparatus to obtain p-y and soil damping relationships from field samples specifically tailored for well conductor fatigue analysis. In addition, it compares the test results obtained using reconstituted kaolin clay and intact natural Onsøy clay to the centrifuge test results; ultimately demonstrating a satisfactory agreement between the two techniques. The results are highly encouraging and believe present a major step forward in deepwater well conductor fatigue analysis. The findings may also be beneficial to the offshore renewable energy sector.

Keywords

Offshore well conductor; fatigue analysis; p-y curve; soil-conductor interaction; cyclic soil response, centrifuge modelling.

Introduction

Deepwater oil and gas wells are drilled from anchored or dynamically positioned vessels (semi-submersible drill rigs and drillships). Exploratory and appraisal wells are drilled to determine the presence of hydrocarbons and to evaluate the size, quantity and extent of a potential field.

Development wells include production wells to extract hydrocarbons and water injection wells to enhance the production and/or to dispose of fluid by-products. Figure 1 presents a schematic of a typical, top-tensioned riser (TTR). The drilling riser systems consists of a blowout preventer (BOP) and a lower marine riser package (LMRP) that typically ranges between 2,000 and 4,500 kN (i.e., submerged or buoyant weight) atop the wellhead with its centre of gravity approximately 14 to 20 m above the mudline. A well is comprised of a series of concentric steel pipes termed "casing strings" that are telescopically drilled into the seabed to facilitate penetration of the final test well or production string (Figure 2). The first string of well casing or pipes is called a surface conductor or a structural casing. Outside diameter (D) of offshore conductors typically vary between 0.76 m to 0.91 m.

An important aspect of TTR well integrity involves fatigue and strength. Fatigue is defined as the weakening of a material caused by repeated changes in cyclic stresses as opposed to an absolute failure stress. Fatigue is a progressive and localized phenomenon that can lead to structural damage. Fatigue stresses in a conductor are primarily caused by water and or wind loads acting on the vessel and the riser system. Strength analysis involves estimating the stresses developing in the conductor and the casing strings during a potential drive-off or drift-off event and then comparing the stresses to the individual components' capacities. Fatigue stresses are often well below the yield stress of a typical conductor and thus result in small deformations. Both the fatigue and strength analyses require geotechnical input data to perform soil-structure interaction modelling. Soil response is typically modelled using springs characterized by lateral load-displacement (p-y) relationships which are highly dependent on the soil type, strength properties and its cyclic characteristics. Present engineering practice typically involves using the p-y curves recommend by the American Petroleum Institute (API 2011) for simulating soil-conductor interaction caused by fatigue events. These p-y curves were developed for ultimate limit state (ULS) design of pile foundations for steel jackets subjected to

monotonic and cyclic storm or hurricane loading and not for fatigue limit state (FLS) assessment of TTR well systems.

Past research in this area involved indirectly obtaining p-y relationships from back-analysis of bending moments measured during pile load tests (e.g., field tests, large or medium scale models tests and/or centrifuge modelling). Such an approach is often subjected to limitations and uncertainties associated with instrumentation resolution and measurement accuracy, determination of soil strength profiles, etc. This paper describes the development of a novel laboratory apparatus from which soil p-y relationships can be determined for input for the FLS assessment of TTR well systems. Although designed to encompass a wide range of soils and stress conditions, the apparatus has so far only been tested and validated for normally consolidated (NC) clays, which are prevalent in deepwater environments. A detailed description of the design requirements, challenges and the approach taken to address them are provided.. Furthermore, the results obtained for two types of NC clays: reconstituted Speswhite kaolin clay and natural Onsøy clay (a Norwegian type clay material), are provided for comparison to existing models that currently are used to represent soil behaviour in FLS assessment of TTR well systems. Finally, knowledge gaps and opportunities for future development are discussed.

Background information

The soil-conductor interaction process is complex. Initially, cyclic forces cause the conductor to translate. Under cyclic loading, the soil strength of normally consolidated clays gradually degrades and, as a result, the displacements increase as the conductor's maximum bending moment location shifts deeper in the soil. Following degradation of the soil's strength, the entire TTR well system reaches a *steady-state* condition, whereby the lateral displacement of the conductor remains constant, limited by the lateral movement of the drilling vessel. The cyclic loads could occur either symmetrically about the well's central axis or offset from it when the vessel moves during the Loop current or a storm event. The latter loading condition adds extra complexity to the conductor-soil interaction analysis.

Soil response modelling plays a significant role in well fatigue analysis. An appropriate model that accurately represents a soil's response for conductor-soil interaction analysis is based on the absolute magnitudes of stresses and their changes. These can accurately be predicted for well integrity evaluation. Zakeri et al. (2016a; 2016b; 2015) conducted an extensive study involving four series of centrifuge model tests and finite element (FE) numerical analyses to fundamentally study conductor-soil interaction under a wide spectrum of loading conditions. The tests simulated conductor installations in normally to over consolidated clays, and medium-dense clean sands. Approaches were developed specifically for conductor fatigue analysis for each of the soil types used in the centrifuge tests. The approaches involved representing the cyclic soil behaviour at the steady-state condition (i.e. degraded soil) using soil springs for input to the FLS assessment of TTR well systems.

Zakeri et al. (2016a; 2016b; 2015) also demonstrated that the approaches perform very well in predicting conductor fatigue life when compared to the centrifuge test results. This was a major step forward in FLS assessment of TTR well systems. Jeanjean et al. (2015) described in detail, the implications of these approaches in FLS assessment of offshore wells. The bending and axial stresses in a conductor and top portion of the well have to be determined as accurately as possible in well fatigue analysis. Fatigue hotspots can occur both above and below the seafloor. Under or over estimation of soil stiffness can lead to erroneous predictions of fatigue hotspot locations and damage magnitudes. Jeanjean et al. (2015) demonstrated this point and concluded that the use of softer p-y curves in system fatigue analysis does not necessarily result in a conservative outcome.

Zakeri et al. (2016a; 2016b; 2015) approaches have also performed satisfactorily in predicting field response obtained from monitored offshore wells. Russo et al. (2016) compare measured deformations and stresses obtained from monitoring a well in the North Sea to those predicted using the p-y relationships recommended by API (2011) and Zakeri et al. (2016a; 2016b; 2015) and conclude that the latter approach yields predictions much closer to the measurements despite uncertainties in the site's soil properties. Kannala et al. (2016) used the NC clay formulation from Zakeri et al. (2015) with site specific soil information for a deepwater well in the

Gulf of Mexico (GoM) that was instrumented for monitoring purposes. He obtained a precise match between the measured and predicted stiffness of the BoP stack from below the LMRP to 50 m below the mudline (BML).

Goal and objectives

The goal was to develop a laboratory apparatus from which p-y data and hysteretic damping input required for offshore conductor-soil interaction analysis could be directly obtained. As such, the device was developed as a small scale model test, in which a small model conductor is pushed into a soil specimen, thereby mimicking the installation process as accurately as possible. The technical objectives of the apparatus were to:

- Develop an actuator loading system capable of producing cyclic (harmonic and random) and monotonic motions that are scalable with respect to displacements, frequencies and velocities found in offshore well conductors,
- Develop a type of chamber to retain soil specimens obtained from field samplers such as piston core, jumbo piston core and or thin-walled tubes, and
- Develop a consolidation system that can reproduce a soil's vertical stress history and maintain the *in-situ* soil stresses throughout the test, thereby simulating a wide range of soil and stress conditions.

Validation requirement and approach

The apparatus next had to be validated using the approaches outlined by Zakeri et al. (2016a; 2016b; 2015), which focus on unload-reload stiffness (secant stiffness) of the disturbed soil at the *steady-state* condition, $K_{\text{sec_SS}}$. As such, a similar approach was adopted for the validation process for NC clays which are predominantly encountered in deep waters. Obtaining a satisfactory validation for NC clays was considered appropriate for functional testing of the apparatus and acceptable as the first step in the validation process before investigating other soil types. Thus, the $K_{\text{sec_SS}}$ and hysteretic damping ratio, ξ , were calculated using the p-y relationship obtained from the apparatus and were subsequently compared to the NC clay model outlined by Zakeri et al. (2016a; 2016b; 2015).

Development of the apparatus

General

The apparatus was developed in four phases: Phase 1 was concept development and involved FE numerical modelling to address the following objectives:

- Scaling and establishing the model conductor and soil chamber material properties and geometries to minimize boundary effects during testing,
- Ensuring a constant volume condition and that a full flow-around failure mechanism occurs in the soil during testing, and
- Defining the load, displacement and frequency requirements for the actuator system.

Phase 2 included the detailed design and fabrication of the device. Phase 3 was functional testing and troubleshooting. Phase 4 involved production testing and verification.

Design specifications

The design specifications were as follows:

- Actuator loading system:
 - Displacement range normalized to conductor diameter: ± 0.005 to 0.5
 - Frequencies: 0.1 to 4 Hz
 - Multiple loading events capable of providing a minimum of 1,000 cycles per event
- Consolidation frame:
 - Substantially rigid
 - Application pressures up to 800 kPa
 - Capable of applying a constant stress during consolidation, then locking the frame to maintain a constant volume during testing
- Soil chamber:
 - Inner diameter dimensions: minimum of 68 mm and maximum of 100 mm
 - Height-to-inner diameter ratio: greater than 1:1 (2:1 preferred)
 - Suitable for clays (with potential for future modification to conduct tests on silts and fine sands)

Concept development

The general concept considered using a solid rod to representing an offshore conductor that moves inside the prepared soil specimen contained in a cylindrical tube. A method then had to be developed to consolidate the soil specimen, while maintaining the consolidation pressure and allowing for the relative movement between the rod and the soil. Three general concepts were initially developed, one of which was selected as the final design. These early concepts (upper section) are shown on Figures 3a, 3b and 3c. In order to generate the consolidation pressure, the first idea was to react against a fixed frame while allowing excess pore pressures to dissipate via porous platens at the top and bottom of the soil column. The platens had to be restrained against the consolidation frame (outer cell) in order to prevent them from rotating during movement of the rod (model conductor). Concepts A and C allowed the rod to extend outside the consolidation frame on both sides for connection to the actuator system, but in Concept B the rod was flush with the inside of the platen.

Free movement of the rod in Concept A was envisaged by having a gap in the platen and a pressurized membrane to prevent soil from escaping the tube. However, these concepts were considered impractical. In Concept C, sliders facilitated free movement of the rod while maintaining a constant soil volume inside the container. Two small vertical plates at the end of the slider allowed deformation during vertical consolidation and helped contain the soil during the conductor's lateral cyclic action. Concept C was selected for design, with the focus on minimizing the internal horizontal friction between the slider and the soil.

A pile-in-pile option (Figure 4) was considered for the model conductor in order to reduce the friction at the slide-soil contact. This option or a variation of it was actually equally applicable to the other two concepts. However, the limitation associated with displacement and technical complexities such as rod stiffness and deformation due to the change in diameter, rod-soil interaction at the geometrical nonconformity as well as other practical issues resulted in this option being discarded. Eventually, it was decided to slightly modify Concept C by bringing the slider seal plates to the inside of the soil container tube and use this setup for detailed

numerical modelling, design and construction. The conductor rod protruding out from the either end of the soil specimen chamber is connected to the loading arms of the actuator.

Numerical modelling

General

As part of the design of the new device, numerical analyses were performed. The purpose of the numerical modelling was to investigate optimal model conductor and specimen geometries and to minimize boundary effects, elastic deformations and friction from the sliders at the top and bottom caps. Two different numerical models were used; a 2D plane strain model and a 3D model. The 2D model was used to assess the maximum conductor diameter, whereas the 3D model was used to assess the minimum specimen height. The boundary effects were investigated in both models.

Finite Element Model

The commercial Finite Element (FE) software ABAQUS/Standard was used in this study. Figure 5 shows the 3D FE model results utilizing symmetry (i.e. only one quarter of the specimen cylinder, soil sample and model conductor, and one half of the cap, was modelled). The specimen has a diameter to height ratio of 1 to 0.8, meaning that the modelled specimen height is 0.4 times its diameter. Diameters of the soil specimen and model conductor shown in Figure 5 are 68 mm and 10 mm, respectively.

To replicate in-situ conditions, the contact between the model conductor and soil as well as the contact between the soil and the cylinder were modelled as a rough interface preventing both sliding and separation. The interface between the soil and the cap was modelled using a linear elastic, perfectly plastic material model applying the undrained remoulded Direct Simple Shear (DSS) strength of the adjacent soil as its yield strength limit. In addition, a smooth contact allowing free sliding of the soil and the cap was investigated in order to compare the 3D model with the 2D model. Separation between soil and cap was allowed in this interface for both contact conditions. Frictional contact between the slider and cap as well the slider and model conductor were assumed using a linear elastic, perfectly plastic material model utilizing

Coulomb failure criteria in which the shear strength depends on the applied effective normal stress. A friction coefficient of 0.2 was adopted. Similar contact conditions were adopted in the 2D model (Figure 6) when needed. Fully integrated four-node and eight-node continuum elements were used in the 2D and 3D FE models, respectively.

Displacement controlled analyses were performed in which the lateral displacement of the model conductor is prescribed and the reaction force is calculated. The sample cylinder was constraint for lateral and axial movements. The displacement of the model conductor was prescribed at the loading arms (connecting the cyclic actuator to the conductor rod protruding out from either end of the soil specimen cylinder) in the 3D model, or at the entire model conductor in the 2D model, respectively.

Constitutive model

The soil was modelled using the NGI-ADP model, which is an anisotropic undrained, total stress based constitutive model proven suitable to describe the material behaviour of normal- and over-consolidated soft soils. Grimstad et al. (2012) present a detailed description of the model. Undrained static stress-strain curves measured in DSS, triaxial compression and triaxial extension tests are directly inputted into the model. In addition, parameters describing the transition between the different corresponding failure surfaces are required.

The parameters applied in this study are listed in Table 1. Given the flow-around mechanism being representative for the case studied, isotropic strength parameters were applied by using only the DSS shear strength. These parameters correspond to a NC soil subjected to a consolidation stress of 40 kPa. In addition, conditions corresponding to higher consolidation stresses and or higher consolidation ratios (i.e. OCR greater than one) were evaluated.

Numerical modelling results

Based on a parametric study using the 2D FE model, it was found that a model conductor diameter of 10.0 mm can be used without being adversely affected by the rigid boundary of the specimen cylinder.

Figure 7 compares the soil resistance-displacement (p - y) curves calculated with the 3D and 2D models plotted against the theoretical plasticity solution. In the figure, the soil resistance is represented by the bearing capacity factor, N_p , which is the ratio of soil resistance divided by the product of projected area and undrained shear strength. Two cases were investigated in the 3D model; smooth and full rough interface between the soil and end caps. Figure 8 presents the results in Figure 7, but with the soil displacement normalized by the model conductor diameter in the form of percentages. The curves for the 2D model and the 3D model with the smooth interface are almost identical and agree well with the analytical solution. However, with the rough interface along the end-caps, the lateral resistance increases. For a soil specimen with an aspect ratio of 1.0 to 0.8, the lateral resistance is greater by approximately 15%. Based on that result, a higher aspect ratio should be used if large displacement amplitudes are applied and rough contact is expected along the interfaces. For small displacement amplitudes, however, it appears that end-friction has no noticeable effects on resistance. This allows using small aspect ratios when one is interested solely in establishing cyclic p - y curves for small displacements. During the laboratory model tests, the surfaces of the sliding bars contacting the soil specimen were lubricated.

Correction for elastic flexure

The rod (model conductor) is not a rigid object and elastically flexes under loading, especially at small displacements. Therefore, a degree of compliance exists within the system. Further, the displacement amplitude is commanded to the actuator whereas the actual displacement is measured at the end of the rod. This requires investigation and quantification of the elastic deformation to be used for data correction. A simple FE beam model was created in Abaqus/Standard to calculate the elastic flexure in an iterative manner. The model consisted of two point loads where the loading arm is connected, rotation fixity at the sliding bars and uniform loading along the soil-rod contact estimated from the measured unload-reload stiffness. The elastic correction magnitude depends on the load amplitude and soil resistance. The maximum correction value estimated for the tests reported herein was approximately 0.35% of $\Delta y/D$ (normalized peak-to-peak displacement).

Design and fabrication

Figure 9 presents the final design and details of the soil container (i.e. the cylinder, end caps, sliders and rod). Figure 10 shows the fabricated apparatus in its final form along with its components. The design and fabrication was completed in approximately three months, at which time the apparatus was available for validation testing. The final design allowed for the following:

- a) Insertion of the rod (model conductor) with high precision through holes on either ends of the supports (Figure 9, left) aligned precisely at the centre of the soil column prior to consolidation. A pre-augered hole is used to minimize soil sample disturbance during rod insertion.
- b) Consolidation of the soil specimen to a desired stress level with the installed rod.
- c) Sustaining the consolidation stress and the final consolidated soil height to maintain a constant volume in the consolidated soil specimen during the p-y testing.

Validation tests

Soil and prepared sample properties

Two types of clays were used in the validation tests: reconstituted Speswhite kaolin clay and a natural clay from a site in Onsøy, Norway. Onsøy clay consists of a thick deposit of highly uniform marine clay, generally normally consolidated to lightly over-consolidated, located below a surficial layer (typically a couple of metres thick) of desiccated clay crust. The clay is fully saturated below this crust. Due to the thickness of the clay deposit and its high homogeneity, the Onsøy site has been used for research purposes by NGI for many years (Lunne et al. 2003). As such, there is an extensive database of soil properties available for Onsøy clay.

Table 2 summarizes the index properties of the kaolin and Onsøy clays. The kaolin clay was first reconstituted in the laboratory and then subjected to approximately 60 kPa pre-consolidation stress in order to facilitate trimming and insertion into the cylinders. The Onsøy clay samples (retrieved between 4 and 5 m depth) were obtained from NGI's sample inventory. Once placed in the cylinders, both the kaolin and Onsøy clays were consolidated to the target

test consolidation pressures σ'_p , in the apparatus. Table 3 summarizes the soil conditions and properties used in the tests. All specimens were tested under normally consolidated conditions. For kaolin, the s_u was estimated, based on $s_u/\sigma'_p = 0.185$ which was determined from previously conducted DSS laboratory tests. For Onsøy, the s_u was obtained directly from NGI's database of sophisticated in-situ and laboratory test results from the particular location where the sample was obtained.

Test matrix

Table 4 presents the test matrix. The loads were cyclically applied with a minimum of 1,000 cycles at a frequency of 0.5 Hz. The displacements presented are two-way (i.e. peak-to-peak), normalized by the model conductor diameter. Furthermore, the normalized displacements in the table include a correction for the elastic deformation of the model conductor.

Results of validation tests and comparisons

Soil resistance-displacement (p-y) data

Figure 11 presents the soil resistance versus time histories for the kaolin clay test (SCTD001). A steady-state condition was achieved in all four displacement motions and the results exhibited characteristics very similar to those observed in the centrifuge tests reported by Zakeri et al. (2016a; 2015). Figure 12 presents the soil resistance time histories obtained for the Onsøy clay test (SCTD005) for motions m1 to m4, and Figure 13 shows the corresponding p-y data. Again, the results are very similar to the centrifuge tests. It should be noted that the long cylinder test produced the same results as the tests done with the short cylinders within the displacement range reported, except that it required a considerably longer consolidation time. The primary objective of developing the apparatus was to facilitate the detailed investigation of dynamic soil behaviour based on directly measured p-y data under constant volume conditions. The results of the validation tests to date demonstrate that this objective was achieved. For example, Figure 14 shows the hysteretic p-y loops from the first to the 1,000th cycle for motion m4 in the Onsøy clay test (SCTD005). It is interesting that the m4 motion was preceded by smaller motions in the previous cyclic loading stages, and soil deformations can easily be observed

transforming from a classical oval shaped hysteresis loop, to an s-shaped loop, typically observed for soils subjected to large strain cyclic loading.

Conventional geotechnical cyclic tests (e.g. cyclic direct simple shear, cyclic triaxial extension or compression tests) are typically conducted using either strain-controlled or stress-controlled conditions. It is possible that actual loading in deepwater is not clearly defined and hence may not be perfectly modelled using purely strain-controlled or stress-controlled loading conditions. An example is a TTR system in which the lateral cyclic loading of the soil follows a complex path. Immediately after installation of the BOP stack, the soil response to lateral loading of the conductor could be modelled using a stress-controlled condition since lateral displacements at the mudline are often only a few millimetres during typical sea conditions. In fact, they reduce from typically less than 25 mm at the mudline rapidly to a couple of conductor diameters just below the mudline. However, during drilling operations, when the vessel is subject to larger seas and or storm conditions, the soil behaves plastically and its resistance quickly reduces. However, the tension of the riser, together with a dynamically positioned or moored vessel, prevents a complete lateral collapse of the BOP and the well. In such conditions, the degraded soil response is best modelled as strain-or displacement-controlled. The p-y apparatus is capable of simulating a wide range of loading conditions, which in turn, facilitates the detailed investigation of dynamic soil behaviour.

Soil resistance degradation and hysteretic damping ratio

Figure 15 presents calculated soil pressures versus load cycles and time for the Onsøy clay test (SCTD005). The soil pressure, P , was calculated by:

$$P = \frac{p}{D \cdot L} \quad (1)$$

Where, L is the length of the rod in contact with the soil. Figure 16 presents the corresponding damping ratios for the same test. There seems to be a slight kinematic hardening taking place for the smallest motion, in which its hysteretic loops closely resemble unload-reload on the initial portion of a monotonic p-y curve (Figure 13). In the centrifuge tests reported by Zakeri et al.

(2016a), a steady-state condition was primarily reached within the first 300 cycles. A similar trend was observed in all the tests conducted in the p-y apparatus.

K_{sec._SS} and comparison with the centrifuge tests

Figure 17 compares the normalized secant stiffness (unload-reload stiffness) obtained from the steady-state condition tests ($K_{Sec_SS_Norm.}$) against those obtained from the centrifuge tests reported by Zakeri et al. (2016a; 2015). Their paper describes how the approach should be used for calculating p-y relationships for conductor fatigue analysis. In the figure, the secant stiffness values have been normalized by the maximum lateral soil pressure, P_{max} , which is the product of N_p and s_u . A value of 12.0 was used for N_p . Equations 2 and 3 outline the steps used to calculate the $K_{Sec_SS_Norm.}$ presented in Figure 17.

$$K_{Sec_SS} = \frac{P}{\Delta y/D} = \frac{p/D \cdot L}{\Delta y/D} = \frac{p}{\Delta y \cdot L} \quad (2)$$

$$K_{Sec_SS_Norm.} = \frac{K_{Sec_SS}}{N_p \cdot s_u} \quad (3)$$

The values fall well within the centrifuge data and compare well with the best fit trend. The two data points from the Onsøy clay tests at a normalized displacement of approximately 0.15, plot slightly below the best fit line. This may be the actual behaviour of the clay or due to limitations of the apparatus at larger displacements. This will be investigated in the future. .

Post-test observations and comparison with FE modelling

Following completion of each test, the soil specimen was extruded and sliced for observation and for moisture content determination. The observations were quite interesting. A zone of plasticized clay from plastic strains, was clearly seen in all specimens. Figure 18 (left) shows a photograph of the sliced specimen from the Onsøy clay test (SCTD005). The plasticized zone is evident by the slightly darker contrast in colour. Figure 18 (right) compares this photo with a plot of the plastic strains generated in the FE analysis. It should be noted that the gap that exists between the soil and the removed rod in Figure 18 (left) was created as a result of the consolidation stress being removed in addition to extrusion of the specimen from the chamber.

No gap existed between the rod and the soil during the NC tests. The issue of gap formation was carefully investigated during the apparatus development stage. It was found that when the specimen was loaded to a vertical effective stress below its consolidation stress, a gap could form at cyclic amplitudes greater than about $0.05D$. This was evident from distortions in the shape of the soil resistance-displacement loops at small displacements.

Discussions and conclusions

The apparatus presented herein was developed specifically to model the soil resistance-displacement response (including unload-reload secant stiffness and hysteretic damping ratio) of offshore well conductors, explicitly fatigue analysis. The soil container was sized to accommodate “undisturbed” field samples with diameters as small as 72 mm (e.g. gravity piston cores). The apparatus was intended to simulate well conductor-soil interaction in typical deepwater conditions as closely as possible by consolidating a clay specimen to a desired vertical stress, maintaining the vertical stress and then maintaining a constant soil volume under a wide range of cyclic motions. It also was designed to minimize boundary effects during testing. FE analysis played a significant role in the design process. A key outcome was to obtain reasonable agreement with the centrifuge data presented by Zakeri et al. (2016a; 2015). The results of the validation tests, comparison with the centrifuge data and numerical modelling simulations all suggest that the apparatus satisfactorily met its purpose and objectives within the range of test conditions presented herein. This is an important step forward for conductor-soil interaction analysis in the field of well integrity evaluation.

These results are highly encouraging and indicate significant potential for the apparatus. Additional tests are required to fully investigate the potential limitations of the apparatus, as well as to make any necessary improvements so as to expand its application to include a wider range of soils such as over consolidated clays, sands and carbonate clays and loading conditions (e.g. monotonic and random). Finally, it should be pointed out that technical issues associated with fatigue damage caused by environmental loads are also important to the offshore wind energy industry. Development of this apparatus may also be beneficial to the offshore renewable energy sector.

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Nomenclature (list of symbols)

API	American Petroleum Institute
BOP	Blowout Preventer
d	Model conductor outside diameter
d _i	Model conductor (pile-in-pile) inside diameter
d _o	Model conductor (pile-in-pile) outside diameter

D	Conductor outside diameter
DSS	Direct Simple Shear test
FE	Finite Element
FLS	Fatigue Limit State
GoM	Gulf of Mexico
h	Height of cylindrical tube containing the soil test specimen
$K_{\text{sec_SS}}$	Soil secant (unload-reload) stiffness at steady-state condition
$K_{\text{sec_SS_Norm.}}$	Normalized soil secant (unload-reload) stiffness at steady-state condition
L	Length of model conductor (rod) in contact with soil
LMRP	Lower Marine Riser Package
NC	Normally consolidated
N_p	bearing capacity factor
p	Lateral soil resistance
P	Lateral soil pressure
P_{max}	Maximum lateral soil pressure
TTR	Top-Tensioned Riser
ULS	Ultimate Limit State
y	Lateral soil displacement
s_u	Undrained shear strength of soil
Δy	Lateral soil displacement, two-way loading or peak-to-peak in a cycle
σ'_p	Consolidation stress
ξ	Soil damping ratio

Table captions

Table 1 NGI-ADP parameters used in the parametric FE analyses

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Table 3 Test conditions and properties

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Figure captions

Figure 1. Schematic diagram of a typical top-tensioned riser (TTR) system

Figure 2. Schematic diagram of a typical well system below the mudline

Figure 3. Concepts initially developed: (left, top) Concept A, (left, bottom) Concept B, and (right) Concept C

Figure 4. Pile-in-pile option

Figure 5. 3D FE model showing lateral displacement contours

Figure 6. 2D FE model showing the in-plane logarithmic strains

Figure 7. Normalized load-deflection curve

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Figure 9. Final design: (top) overview of the apparatus, and (bottom) details of the cylinders, sliders, consolidation and cyclic loading directions

Figure 10. Photo of the fabricated apparatus and all components

Figure 11. Soil resistance time histories: kaolin clay test (SCTD001), motions m1 to m4

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Figure 15. Onsøy clay test (SCTD005): secant stiffness, $K_{Sec.}$, degradation against number of cycles (left) and cumulative time (right)

Figure 16. Onsøy clay test (SCTD005): soil damping ratio, ξ , against number of cycles (left) and cumulative time (right)

Figure 17. Normalized secant stiffness, $K_{Sec_Norm_SS}$: p-y apparatus test results comparison with Series 1 centrifuge data (Zakeri et al. 2015; 2016a)

Figure 18. Onsøy clay test (SCTD005): (left) post-test photo of cut soil specimen showing plasticized zone (the gap between the soil and the rod was formed as a result of stress relief unloading the consolidation stress and extruding the specimen out of the chamber), and (right) comparison of observed plasticized zone with that calculated using FE modelling

Figure 1. Schematic diagram of a typical top-tensioned riser (TTR) system

Figure 2. Schematic diagram of a typical well system below the mudline

138x113mm (300 x 300 DPI)

Figure 3: Concepts initially developed: (left, top) Concept A, (left, bottom) Concept B, and (right) Concept C

Figure 4. Pile-in-pile option
144x180mm (300 x 300 DPI)

Figure 5. 3D FE model showing lateral displacement contours

Figure 6. 2D FE model showing the in-plane logarithmic strains

Figure 7. Normalized load-deflection curve

74x54mm (600 x 600 DPI)

Figure 8. The results shown in Figure 7 in log scale with focus on the small displacement range
74x53mm (600 x 600 DPI)

Figure 9. Final design: (top) overview of the apparatus, and (bottom) details of the cylinders and sliders, consolidation and cyclic loading

Figure 10. Photo of the fabricated apparatus and all components
731x548mm (180 x 180 DPI)

Figure 11. Soil resistance time histories: kaolin clay test (SCTD001), motions m1 to m4
184x141mm (300 x 300 DPI)

Figure 12. Soil resistance time histories: Onsøy clay test (SCTD005), motions m1 to m4
184x141mm (300 x 300 DPI)

Figure 13. Soil resistance-displacement (p - y): Onsøy clay test (SCTD005), motions m1 to m4
200x200mm (300 x 300 DPI)

Figure 14. Soil resistance-displacement (p - y): Onsjø clay test (SSTD005), motion m4, highlighting first and last cycles

160x160mm (300 x 300 DPI)

Figure 15. Onsøy clay test (SCTD005): secant stiffness, K_{Sec} , degradation against number of cycles (left) and cumulative time (right)

125x60mm (300 x 300 DPI)

Figure 16. Onsøy clay test (SCTD005): soil damping ratio, ξ , against number of cycles (left) and cumulative time (right)

125x60mm (300 x 300 DPI)

Figure 17. Normalized secant stiffness, $K_{\text{Sec_Norm_SS}}$: p-y apparatus test results comparison with Series 1 centrifuge data (Zakeri et al. 2015; 2016a)

217x176mm (300 x 300 DPI)

Figure 18. Onsøy clay test (SCTD005): (left) post-test photo of cut soil specimen showing plasticized zone (the gap between the soil and the rod was formed as a result of stress relief unloading the consolidation stress and extruding the specimen out of the chamber), and (right) comparison of observed plasticized zone with that calculated using FE modelling

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Table 1 NGI-ADP parameters used in the parametric FE analyses

G_{ur}/s_{uA}	$s_{uA,ref}$	s_{uDSS}/s_{uA}	s_{uP}/s_{uA}	ε_{fA}	ε_{fA}	ε_{fA}	τ_0/s_{uA}	η
300	8 kPa	1.0	1.0	6%	6%	6%	0.0	0.495

Table 2 Index properties¹ of kaolin and Onsøy clays

Clay Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (PI) (%)	Specific Gravity	Clay Fraction (CF) (%)	Activity Ratio PI/CF
Kaolin	63	32	31	2.65	76.0	0.407
Onsøy ²	60	30	30	2.71	50 to 60	0.6 to 0.5

Notes:

1) The above data were obtained based on ASTM (D422 - 63; D854; D2216 - 10; D4318 - 10) procedures.

2) Values of the specific gravity and clay fraction were obtained from Lunne et al. (2003).

Table 3 Test conditions and properties

Test Label	Soil	Cylinder type	Specimen Length (mm)		Water content (%)					σ'_p (kPa)	s_u^5 (kPa)
			Before ¹	After ²	Before ³	After ⁴					
						Top	Centre	Bottom	Average		
SCTD001	Kaolin	long	167	165.0	Note 7	52.1	55.5	54.3	54.0	65	12.1
SCTD004	Onsøy	short	100	99.0	71.7	68.2	68.7	69.1	68.7	59	13.0
SCTD005	Onsøy	short	100	82.7		52.3	56.0	52.3	53.5	200	50.0
SCTD007	Kaolin ⁶	short	102	91.5	Note 7	-	-	-	-	90	16.7
			91.5	85.4		46.3	43.8	47.2	45.8	180	33.5

Notes:

- 1) Represents specimen height inside the cylinder, but prior to application of consolidation stress.
- 2) Represents specimen height inside the cylinder after completion of consolidation and prior to testing.
- 3) Water content measured prior to application of consolidation stress.
- 4) Water contents measured after completion of the test at three locations and then averaged.
- 5) All specimens were tested under normally consolidated condition. For kaolin, the s_u was estimated based $s_u/\sigma'_p = 0.185$ established from previously conducted laboratory tests. For Onsøy, the s_u was obtained from NGI's database of extensive sophisticated in-situ and laboratory tests at the particular location where the sample was retrieved. The s_u values are equivalent to those obtained using a direct simple shear test.
- 6) SCD007 test was first consolidated to 90 kPa, tested and then immediately consolidated to 180 kPa without removing the specimen from the cylinder.
- 7) The kaolin clay specimens were reconstituted by mixing dry powder to approximately 2 times the Liquid Limit water content (i.e. ~126%) and then consolidated to about 60 kPa.
- 8) Diameters of the model conductor (rod) and soil specimens in all tests were 10.0 mm and 68.0 mm, respectively.

Table 4 Validation test matrix

Test Label		Motion Sequence Number and Normalized Displacement Value ($\frac{\Delta y}{d}$)				
		m1	m2	m3	m4	m5
SCTD001	(Kaolin)	0.0375	0.0875	0.1380	0.1380 ^b	
SCTD004	(Onsøy)	0.0042	0.0103	0.0290	0.0686	0.1477
SCTD005	(Onsøy)	0.0034	0.0126	0.0201	0.0641	0.1489
SCTD007	$\sigma'_p = 90kPa$	0.0010	0.0039			
	$\sigma'_p = 180kPa$	0.0135	0.0085			
Note: a) The normalized displacements are two-way peak-to-peak loading. b) All tests were conducted at a harmonic loading frequency of 0.5 Hz, except m4 of SCT001, which was done at 0.1 Hz to investigate frequency effects. c) The loads were applied consecutively with no consolidation time allowed.						