GUIDANCE NOTES ON

DESIGN AND INSTALLATION OF DYNAMICALLY INSTALLED PILES

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Updates

March 2018 consolidation includes:

- December 2017 version plus Corrigenda/Editorials

Foreword *(1 December 2017)*

These Guidance Notes provide ABS recommendations for the design and installation of dynamically installed piles for offshore service.

Included in these Guidance Notes are methodologies for the geotechnical design and structural assessment of dynamically installed piles. Recommendations in these Guidance Notes are based on finite element analysis using appropriate soil modeling and simulation of soil-pile interaction. Alternatively, other approaches that can be proven to produce at least an equivalent level of safety will also be considered.

These Guidance Notes are applicable to the design of dynamically installed piles, as a component of taut, semi-taut, or catenary mooring systems.

The December 2017 edition incorporates a new Appendix 4, "Methodology to Calculate the Anchor Reverse Catenary Line".

These Guidance Notes become effective on the first day of the month of publication.

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GUIDANCE NOTES ON

DESIGN AND INSTALLATION OF DYNAMICALLY INSTALLED PILES

CONTENTS

SECTION 1 General

1 Background

Dynamically installed piles are finned piles designed to be released from a height of 20-100 m above the seabed and reach velocities of 20-35 m/s at the seabed before self-embedment. They may be stabilized with multiple fins at the trailing edge. Tip penetration is expected to be two to three times the pile length and holding capacity after consolidation is expected in the range three to six times the pile weight.

The main purpose of these Guidance Notes is to provide recommendations for the design and installation of a dynamically installed pile for taut, semi-taut or catenary mooring systems. The design and installation of dynamically installed piles are to be based on all applicable requirements of the ABS *Rules for Building and Classing Offshore Installations (OI Rules),* the ABS *Rules for Building and Classing Floating Production Installations (FPI Rules),* and the ABS *Rules for Building and Classing Mobile Offshore Drilling Units (MODU Rules).*

3 Scope and Application

These Guidance Notes covers the geotechnical design, structural assessment and installation of a dynamically installed pile as a component of taut, semi-taut, or catenary mooring systems.

5 Submission of Documents

The design documentation to be submitted should include the reports, calculations, plans and other documentation necessary to verify the adequacy of the foundation and structure. The extensiveness of the submitted documentation should reflect the uniqueness of the pile design and its application.

5.1 Plans

Plans showing the scantlings, arrangements, specification for material, welding and fabrication, as well as construction details of the pile's structure should be submitted and approved before the work of construction is commenced. These plans are to indicate clearly the details of welding, welding consumables, mass, and vertical center of gravity of the pile, etc.

5.3 Reports

Reports indicating the soil conditions, holding capacity verification, and structural behavior should be submitted to ABS for review.

In general, the following documents should be submitted for review, reference, and file:

- Site investigation reports including complete geophysical/geological/geotechnical reports as well as an integration/interpretation report.
- Geotechnical design report to evaluate the adequacy of the holding capacity of the pile. Pile calculation documents should provide minimum penetration and maximum allowable inclinations of a pile from the vertical axis. After pile installation, if significant deviation is found, this report should be updated to verify the adequacy of the pile for the as-installed condition.
- Structural design report to evaluate the adequacy of the pile structure and the mooring padeye.
- Installation procedures and report for piles, including the design and final coordinates for pile installation, design and recovered monitoring data for pile penetration depth, orientation angle with vertical and azimuth angles of fins, when applicable.
- If the reference site is within a seismic zone, the site specific seismic hazard report should be submitted.

SECTION 2 Site Investigations

1 General

The soil investigation should satisfy the requirements given in 3-2-5/3 of the ABS *Rules for Building and Classing Offshore Installations (OI Rules)*. It is recommended that a high-quality, high-resolution geophysical survey be performed over the entire areal extent of the foundation. It is important that the geophysical and geotechnical components are planned together as integrated parts of the same investigation. Such an integrated study can then serve as a guide to develop a scope of work for the vertical and horizontal extent of the final geotechnical investigation and to aid in the interpretation of the acquired geotechnical data.

The sequence for the investigation program should be:

- Desk study
- Topographical and geophysical survey
- Geotechnical survey and laboratory testing
- Additional geophysical and/or geotechnical surveys and/or laboratory testing as required

Depending on the size of a project and/or the complexity of the geotechnical context and associated risks (geohazards), additional intermediate stages may be necessary.

The soil investigation for a dynamically installed pile should give the basis for a geotechnical design comprising evaluation of:

- Pile embedment depth
- Ultimate holding capacity
- Structural adequacy in term of strength and fatigue resistance

3 Desk Study

The desk study assembles existing data for preliminary site assessment and will formulate requirements for subsequent geophysical and geotechnical investigations. The desk study should include a review of all sources of appropriate information, collect and evaluate all available relevant data for the site including:

- Bathymetric information
- Geological information
- Information and records of seismic activity
- Existing geotechnical data and information
- Previous experience with foundations in the area
- Regional meteorological and oceanographic data

A desk study alone is not sufficient for detailed engineering purposes, but it should be sufficient for conceptual engineering to move forward in a focused manner and provide the basis upon which to design and plan subsequent site investigation work.

5 Geophysical Surveys

Geophysical investigation can provide information about soil stratigraphy, local soil condition and evidence of geological features. The survey area should cover the full extent of the pile spread. The areal extent of soil layers can sometimes be mapped if good correspondence is established between the soil boring and in-situ test information and the results from the seabed surveys. Site specific, high-resolution geophysical information of a regional character obtained from seafloor and sub-bottom surveys at or near the project site should be submitted for review.

The types of equipment for performing geophysical investigation can include the following:

- Echo sounder or swath bathymetric system for water depth and seafloor morphology.
- Sub-bottom profiler for structural features within the near-surface sediments.
- Side-scan sonar for seafloor features and seafloor reflectivity.
- Seismic sources for the structure to deeper depths up to approximately 100 m below the seafloor. The minimum depth of seismic profile should be the anticipated penetration depth plus a zone of influence of about ten times the equivalent pile diameter. The Equivalent Pile Diameter (EPD) is the diameter of a circle that circumscribes the pile structure including fins and shaft.
- Direct observation of the seafloor using a remotely operated vehicle (ROV) or manned submersible.

7 Geotechnical Surveys

The geotechnical evaluation includes sampling for soil classification and engineering property testing, and in-situ soil profiling and strength testing. To establish the soil characteristics at the foundation locations, at least one boring or probing should be taken at pile cluster locations to a depth of the anticipated penetration depth plus a zone of influence of 15.2 m. Additional bore holes may be needed if significant discontinuities of the soil characteristics are verified, from cluster to cluster. If the foundation location is at a significant distance from the boring/probing location, additional verification boring/probing may be needed to validate the extrapolated data.

In deep water, the relief of hydrostatic pore pressure and its resulting effect on any dissolved gases can produce soil properties significantly different from in-situ conditions. Because of these effects, in-situ testing is encouraged as it may provide a more reliable estimate of soil parameters with less sample disturbance. Typical tools used include the remote vane, the piezoprobe, PCPT (piezocone penetrometer tests), T-bar or ball penetrometer, etc. If the soil investigation is performed primarily using PCPT, it is recommended that at least one boring with sampling be taken to properly calibrate the PCPT results. This boring/core should be taken at one of the PCPT locations. The sampling interval should not exceed 1.0-1.5 m.

The soil parameters for pile design will require high quality sampling in combination with advanced laboratory testing and correlations with in-situ test results. Consideration should be given to the performance of permeability and consolidation tests in order to assess set-up effects and capacity consideration for the dynamically installed pile. The site investigation and laboratory testing program should provide the information needed for the reliable design of piles, as shown in Section 2, Table 1. The expected pile embedment and cyclic loading capacities are presented in the Appendices of these Guidance Notes.

If the reference site is within a seismic zone, geotechnical investigations should include tests to determine dynamic soil properties and liquefaction potential.

TABLE 1
Basic Soil Parameters

Clay	*Sand or Silt*
• General description, classification and index testing • Grain size distribution • Organic material content • Mineralogy • Total unit weight • Water content • Undrained shear strength • Remoulded shear strength and soil sensitivity • Permeability • Soil thixotropy • Soils stress history and over consolidation ratio • Strain rate effects and cyclic response • Creep effect for loss of strength under sustained load • Carbonate content	• General description, classification and index testing • Grain size distribution • Maximum and minimum densities • Relative density • Water content (silt) • Carbonate content • Organic material content • Angularity • Angle of shearing resistance • Soils stress history and over-consolidation ratio

SECTION 3 Geotechnical Design

1 Introduction

The main characteristics of a typical dynamically installed pile include: total mass, length and diameter of shaft, number and size of fins, center of gravity and buoyancy, etc. Pile ballast materials, such as lead, are normally used to provide a lower center of gravity, which reduces pile inclination during installation.

The penetration depth and ultimate holding capacity are a function of:

- Soil conditions and geotechnical soil properties
- Impact velocity at seabed
- Pile aspect ratio and tip geometry
- Fin geometry and number
- Pile mass and center of gravity
- Pile verticality after installation
- Mooring line orientation on top of pile
- Type and size of mooring line

Normally the pile's impact velocity increases with an increase in the dropping height. However, there is a threshold value of maximum impact velocity which does not increase beyond a certain dropping height. Instead of deploying the pile installation line at the water surface, a submerged release link can be used to reduce the drag effect from the line, hence increasing the pile's impact velocity.

3 Design Methodology

3.1 Embedment Depth Prediction in Cohesive Soil

Physical modeling including centrifuge tests, small scale tests and field tests are common methods of investigating the penetration depth and subsequent holding capacity of dynamically installed piles. The pile embedment depth is able to be assessed using True's method [Ref.1], accounting for both strain rate effect and inertia drag. The model starts with Newton's second law as follows [Ref.2]:

$$m\frac{d^2z}{dt^2} = W_s - F_{resistance} - F_b - F_{drag} \quad \text{(Eq.1)}$$

where

m	=	submerged mass of the pile, in kg (kg, lb)
t	=	time, in sec
z	=	embedment depth at the pile tip, in m (m, ft)
W_s	=	submerged weight of the pile in water, in N (kgf, lbf)
$F_{resistance}$	=	upward force resisting movement of the pile through the soil including bearing and frictional resistances, in N (kgf, lbf)
F_b	=	buoyancy weight of the displaced soil, calculated as the soil displaced volume which is limited to the pile volume multiplied by the soil submerged unit weight γ', in N (kgf, lbf)
F_{drag}	=	fluid mechanics drag term, in N (kgf, lbf)

F_{drag} should be considered for very soft viscous clay encountered at soil surface and is calculated using Morison's equation:

$$F_{drag} = \frac{1}{2} v^2 \rho_s A_{tip} C_d$$

where

v	=	pile velocity, in m/s (m/s, ft/s)
ρ_s	=	density of soil, in kg/m^2 (kg/m^2, lb/ft^2)
A_{tip}	=	projected area of the pile, in m^2 (m^2, ft^2)
C_d	=	drag coefficient

Field tests show that the equivalent drag coefficient, C_d, on a pile's frontal cross section is relatively small, on the order of 0.15-0.18 [Ref.3]. A value of 0.23 is suggested for the pile with the ratio of the pile length over the shaft diameter over 4 [Ref.2]. When the pile enters cohesive soil, the resisting force is calculated using the design method for driven piles:

$$F_{resistance} = R_f(F_{bear} + F_{friction})$$

$$F_{friction} = a(S_{u,ave}A_s + S_{u,avef}A_{sf}) \quad \text{(Eq.2)}$$

$$F_{bear} = N_c S_{u,tip} A_{tip} + N_{cf} S_{u,bf} A_{pf}$$

where

R_f	=	strain rate function
F_{bear}	=	end bearing resistance at tip of pile and fins, in N (kgf, lbf)
$F_{friction}$	=	frictional resistance along the shaft and fin walls, in N (kgf, lbf)
a	=	friction ratio of limiting shear strength to undrained shear strength, conveniently estimated as the inverse of soil sensitivity, S_t
$S_{u,ave}$	=	shear strength averaged of the soil over the embedded shaft length, in kN/m^2 (kgf/m^2, lbf/ft^2)
$S_{u,avef}$	=	shear strength averaged of the soil over the embedded fin length, in kN/m^2 (kgf/m^2, lbf/ft^2)
A_s	=	projected side surface area of the embedded pile shaft, in m^2 (m^2, ft^2)
A_{sf}	=	projected side surface area of the embedded pile fin, in m^2 (m^2, ft^2)
N_c	=	bearing capacity factor of pile tip
N_{cf}	=	bearing capacity factor of pile fin
$S_{u,tip}$	=	undrained shear strength at pile tip, in kN/m^2 (kgf/m^2, lbf/ft^2)
$S_{u,bf}$	=	undrained shear strength of the soil at bottom of pile fins, in kN/m^2 (kgf/m^2, lbf/ft^2)
A_{tip}	=	cross sectional area of pile tip, in m^2 (m^2, ft^2)
A_{pf}	=	projected cross-sectional area of the pile fin, in m^2 (m^2, ft^2)

Bearing capacity factors from theoretical and experimental cone penetration tests range from 6 to 20, which is a relatively wide range. In the absence of project-specific tests, the N_c value of 12 is recommended. Each pile fin possesses a shape similar to a strip footing, and bearing capacity factor N_{cf} of 7.5 is recommended [Ref.4].

The most uncertain factor in the prediction of embedment depth is the strain rate function, R_f. During installation, it is reasonable to assume that at any given location, the operational strain rate may be proportional to v/d, where v is the pile impact velocity and d is the shaft diameter.

$$R_f = \left(\frac{v/d}{(v/d)_{ref}} \right)^{\beta}$$

where $(v/d)_{ref}$ is the reference strain rate associated with the reference value of undrained shear strength.

Typically, the "strain rate parameter", β has been back-calculated from measured experimental data, which is found to vary for different kinds of clay. For preliminary purposes, in the absence of project-specific test values, a β of 0.11–0.136 may be used with $(v/d)_{ref}$ of $0.17s^{-1}$ [Ref.2].

The differential equation for the velocity at any given depth or time is solved numerically using a finite difference approach. The predicted embedment corresponds to the depth of pile tip where the velocity of the pile goes to zero. The detailed numerical procedure is given in Appendix 1 as well as the results of a sample calculation.

3.3 Holding Capacity Prediction *(1 December 2017)*

With regard to the pile capacity, the API RP 2A [Ref.5] criteria for pile capacity are typically used for loads applied in the axial or horizontal directions. However, the API approach is unable to explicitly account for a particular mooring line loading angle. The pile capacity can be assessed using three dimensional finite element analysis considering an assumed in-place condition. A potential drawback of the analytical and finite element capacity prediction methods, however, is that they fail to simulate the dynamic pile installation process, which may influence the validity of the subsequent pile capacity prediction.

Recently a computational fluid dynamics (CFD) approach has been used to model the complete installation event including: the release and free-fall through the water column, the transition from water into the seabed and the subsequent motion of the pile through the soil. The calibration and validation of these methods against field or experimental data should be pursued.

5 Estimation of Pile Capacity in Cohesive Soil

The simplified solutions predict the lower bound of the measured value, demonstrating that simplified capacity calculations may be used in situations where an estimate of long term capacity is needed.

5.1 Axial Pile Capacity

The theoretical capacity under axial loading is provided by the friction from pile shaft and fins, and pile weight. The ultimate pile pull-out capacity under axial loading F_v in cohesive soil can be calculated by the equation:

$$F_v = W_s + F_{friction} \quad \text{(Eq.3)}$$

where

W_s = submerged weight of the pile, in N (kgf, lbf)

$F_{friction}$ = frictional resistance along the shaft and fin walls, in N (kgf, lbf), see Equation 2.

5.3 Lateral Pile Capacity

For static lateral loads the ultimate unit lateral bearing pressure of pile in soft clay varies between $8S_u$ and $12S_u$ (S_u is soil's undrained shear strength) except at shallow depths where failure occurs in a different mode due to minimum overburden pressure. In the absence of more definitive criteria, the following is recommended:

$$F_h = 9S_u A_{s,h} \quad \text{(Eq.4)}$$

where

$A_{s,h}$ = laterally projected area of the pile's main body, in m^2 (m^2, ft^2)

To be conservative, the projected area of pile's fins is not taken into account.

7 Numerical Modeling

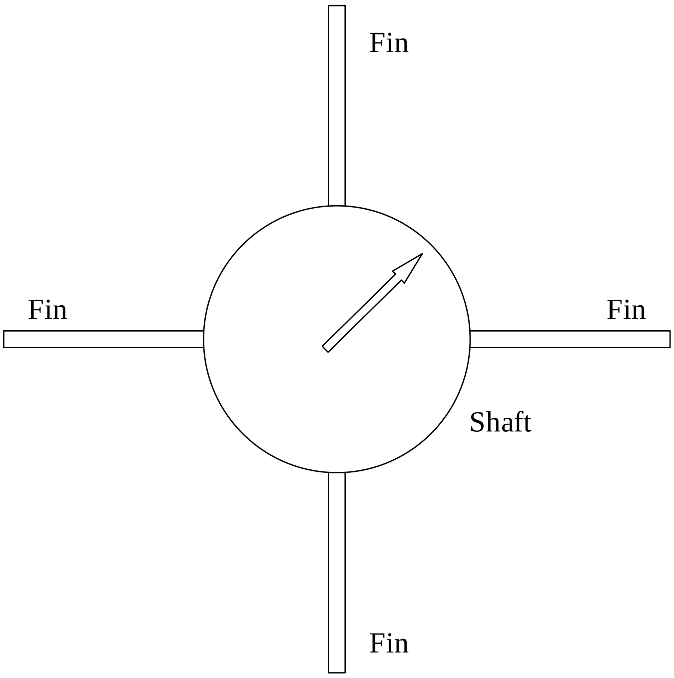
7.1 General

The holding capacity of dynamically installed piles should be preferably determined by three-dimensional numerical analysis. The installation position of the pile is established assuming the soil's stress state is undisturbed by the installation process. Since the pile's penetration depth is one of the key parameters that influences the ultimate holding capacity, the actual penetration depth is obtained by measurement after installation. The range of loading directions considered is from axial to lateral at a constant interval in the elevation view. The pile may include fins to increase its holding capacity.

During the pile design, the following aspects of the holding capacity evaluation should be considered:

- The contribution of suction on the base of the dynamically installed piles is not considered in the holding capacity evaluation.
- The inclination angle of the pile is to be considered so that the pile has the most critical direction with the least load capacity.
- For piles with fins, the mooring load is to be applied in a direction that generates the smallest fin lateral shadow area producing the minimum holding capacity, as shown in Section 3, Figure 1.
- Following soil disturbance and remoulding during installation, the soil set-up effect needs to be considered to make sure that enough pile capacity is built up due to the combined effect of consolidation and thixotropy.
- The effect of the inverse catenary line in the soil on the pile load directions should be considered. Both an upper and lower bound inverse catenary should be considered to ensure the worst-case pile loading is established.
- If the effect of cyclic loading is taken into account on the pile design (i.e., the combined effect of loading rate and cyclic degradation), the pile capacity should be calculated using the cyclic shear strength, see Appendix 2 for details.

FIGURE 1
Illustration of Critical Mooring Load Direction from Plan View



7.3 Finite Element Model

At the boundary of the Finite Element Analysis (FEA) model, soil deformation normal to the vertical faces and the base of the boundary are set to zero, while tangential components are allowed. If a plane of symmetry is used, the out of plane displacements are restrained, see Section 3, Figure 2. The extent of the soil model should be as large as necessary so that boundary conditions do not affect the soil-pile interaction. The Equivalent Pile Diameter (EPD) is the circle diameter that circumscribes the pile shaft and fins. As shown in Section 3, Figure 2, the vertical extent of the soil model below the pile's lower end is suggested to be not less than 2 times the EPD. The lateral extent of the soil model away from pile's extreme ends is not less than 2.5 times EPD in any position. Alternative lateral and vertical model extents may be used if the pile response proves to be unaffected by the boundary conditions.

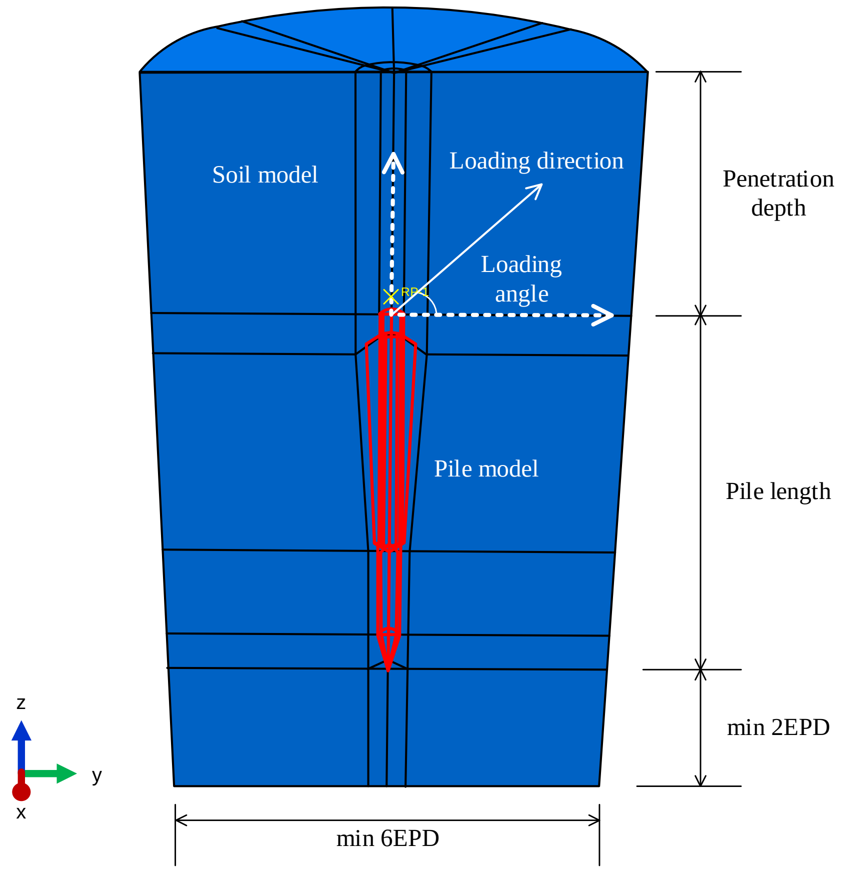
Soil should be modeled by solid elements. A perfectly elasto-plastic material with physical properties changing with soil depth may be used for soil model. The soil strength properties are based on site-specific conditions, see Subsection 2/7. A "best estimate" soil profile should be established for use in pile holding capacity calculations.

Pile structure may be modeled by shell elements or solid elements.

Finer mesh should be used for the regions close to the pile where high plastic strains are expected to occur, and a coarser mesh may be used in regions far from the pile. Similarly, a typical FE mesh for the pile is more refined at its top and at the shaft/fin connection area where stress concentration may occur. Since numerical stability and computational time are mesh dependent, a mesh convergence study is highly recommended.

The relative position of the pile model to the soil model regarding penetration depth, and its inclination between the pile axis and soil cylinder axis, should vary with the designer's specified pile operating range. When the installation report shows a significant difference between the design conditions and the installed conditions, additional calculations should be performed to demonstrate the attainment of sufficient holding capacity in the "as-installed" condition.

FIGURE 2
FE Model Dimensions



7.5 Soil-Pile Interaction

Contact elements should be considered in the model to account for the interaction between soil and pile. The contact surfaces between the soil elements and the pile tip, or the soil elements and the bottom of a fin should be allowed to detach from each other without any tension force. The Young's modulus of soil, the friction coefficient and the maximum allowable shear stress along soil/pile interface are the key parameters that significantly influence the pile's holding capacity subjected to inclined loading.

The maximum allowable shear stress, *f*, are obtained per API RP 2A [Ref.5] as follows:

i) For cohesive soil:

$$f = \propto S_u$$

where

$\propto$ = adhesion factor

$$= 0.5\left(\frac{S_u}{\sigma'_v}\right)^{-0.5} \quad \text{when } \frac{S_u}{\sigma'_v} \leq 1.0$$

$$= 0.5\left(\frac{S_u}{\sigma'_v}\right)^{-0.25} \quad \text{when } \frac{S_u}{\sigma'_v} > 1.0$$

S_u = undrained shear strength, in kN/m^2 (kgf/m^2, lbf/ft^2)

σ'_v = effective overburden pressure, in kN/m^2 (kgf/m^2, lbf/ft^2)

ii) For cohesionless soil:

$$f = K\sigma'_v \tan\delta$$

where

K = coefficient of lateral earth pressure

δ = friction angle between soil and pile wall, in degrees

For the contact properties, the surface contact elements may be used to define the normal and tangential interactions between soil and pile.

7.7 Load Steps

The first step is to establish the initial soil geostatic stresses around the pile with proper pile weight and geometry. The second step is to determine the holding capacity of pile by displacement control on the pile top.

7.9 Acceptance Criteria for Ultimate Capacity

The holding capacity analysis is nonlinear. When calculating the ultimate capacity of the pile, the pile displacement is not to exceed 7% of EPD.

The ultimate holding capacity of the pile should be determined as follows:

i) If a continuous increase in the displacement with almost no variation in the loads is obtained for a deformation less than 7% of EPD, the ultimate holding capacity is to be defined as the asymptotic value (i.e., when the gradient of the curve is less than or equal to 0.02 kN/m), see Section 3, Figure 3.

ii) If it is not possible to identify an asymptotic load value in the resultant force-pile displacement curve, or if the asymptotic condition arises for a deformation greater than 7% of EPD, the ultimate holding capacity is to be considered as the value that corresponds to a pile top total displacement of 7% of EPD, see Section 3, Figure 4.

iii) If the numerical analysis stops due to convergence issues before the pile top total displacement reaches 7% of the EPD, the ultimate holding capacity is to be considered as the last convergent load value obtained in the analysis.

The estimated pile penetration depth for geotechnical analysis can be predicted based on the drivability test in-situ or analytical solution per 3/3.1. However, the pile may not penetrate to the exact depth as predicted. Hence, the pile capacity analysis should be conducted for at least three different penetration levels in a reasonable range. Similarly, at least 5 different loading angles between horizontal and axial force are to be evaluated, including 0°, 45°, 90°, and any other two angles, see Section 3, Figure 2. Second order regression can be applied to provide a holding capacity vs loading angle curve with variable penetration depths, see Section 3, Figure 5. The holding capacity may be obtained by interpolation for the actual penetration depth and loading angles with those from numerical analysis. No extrapolated estimates should be used for the ultimate holding capacity of piles when the penetration depths and loading angle are outside the analyzed range.

FIGURE 3
Example of Ultimate Force for Asymptotic Analysis Results

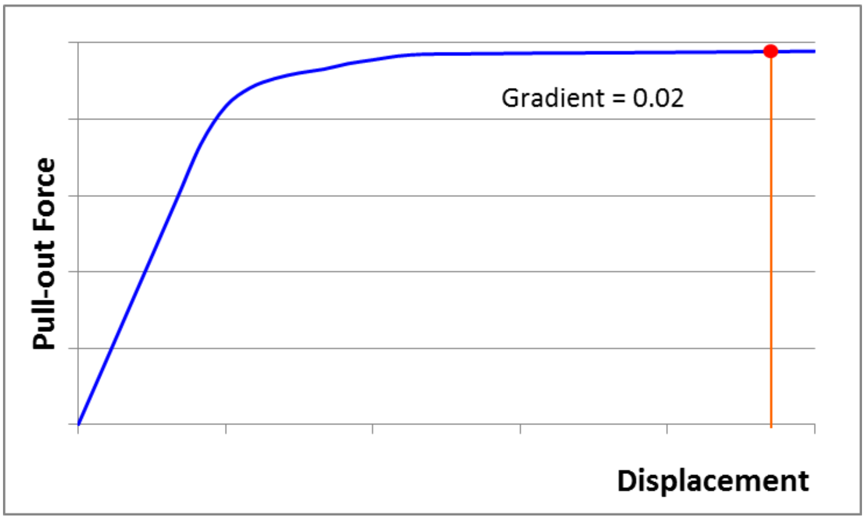


FIGURE 4
Example of Ultimate Force for Non-Asymptotic Analysis Results

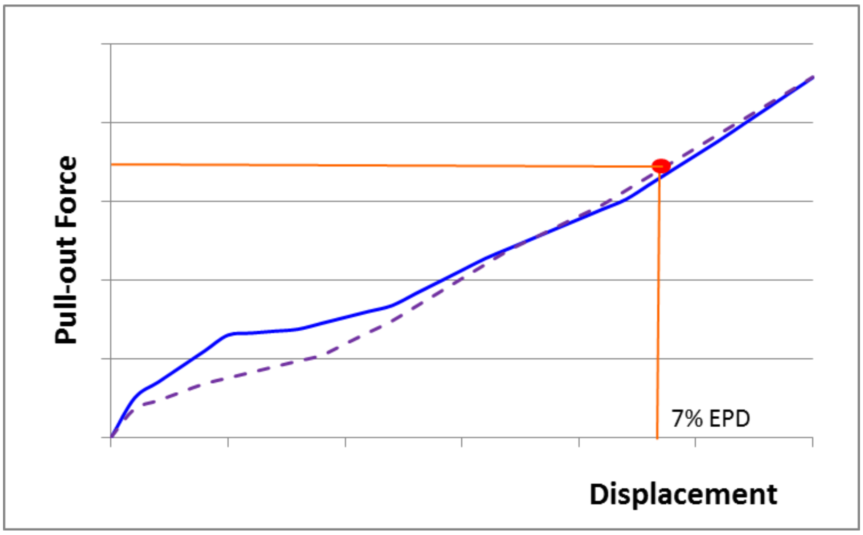
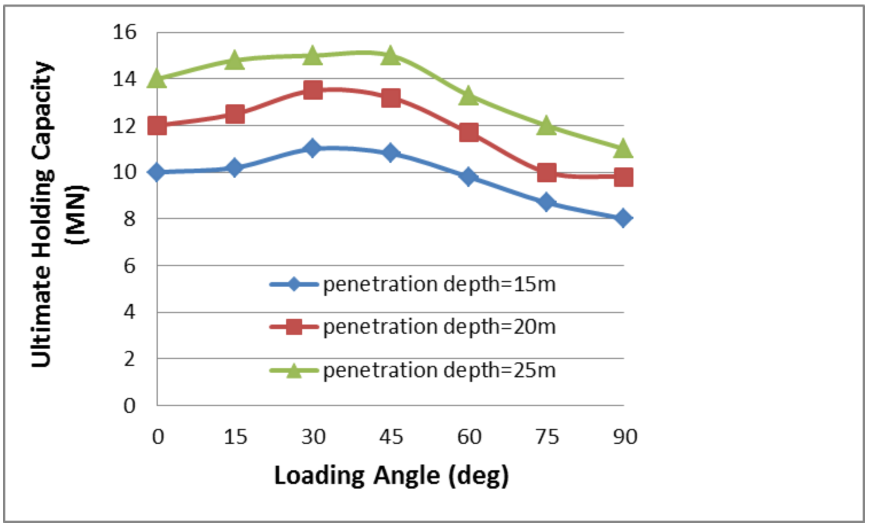


FIGURE 5
Example of Ultimate Holding Capacity Analysis Results



Factors of safety for holding capacity of dynamically installed piles should be checked for the intact condition and the broken line (damaged) condition respectively, see Appendix 3 for details.

9 Set-Up of Pile Capacity in Normally Consolidated Clay

9.1 Set-Up Effect

Following disturbance and remoulding during installation, soil set-up and shear strength in the vicinity of the pile can gradually increase with time due to the combined effect of consolidation and thixotropy. The effect of thixotropy and increased effective stress are not additive, and these two effects should be considered as independent [Ref.6].

Thixotropy can be defined as a process of softening caused by remoulding, followed by a time dependent return to the original harder state at a constant water content and constant porosity [Ref.7]. The thixotropic effect works soon after installation and even before the pore pressure dissipation occurs. The ratio between the shear strength after a certain time with thixotropic strength gain and the shear strength just after remoulding is referred to as the '*thixotropy strength factor*'. [Ref.6] gives upper and lower bound values of the thixotropy factor as function of time and plasticity. However, if the thixotropy effect is significant in design, laboratory testing for the site specific soil should be considered.

The dynamic pile installation process, particularly for those piles without fins, can be likened to the installation of a solid driven pile, which in fine-grained soils, results in the generation of large excess pore pressures due to the combined effects of changes in mean effective stress due to shearing, and increases in total stress as the soil is forced outwards to accommodate the volume of the pile [Ref.8]. Subsequent to installation, the dissipation of excess pore pressures leads to an increase in effective stresses along the pile shaft, resulting in higher frictional resistance and therefore higher pile capacity. Field measurements of excess pore pressure distributions around the driven piles show that the major pore pressure gradients are radial [Ref.8]. Thus, the horizontal coefficient of consolidation, c_h, rather than the vertical one, c_v, becomes the dominant consolidation parameter.

The increase in pile capacity mainly depends on time scale, pile diameter, horizontal coefficient of consolidation c_h, dissipation of excess pore pressure, and soil rigidity index $I_r = G/S_u$ (where G is shear modulus of soil and S_u is undrained shear strength).

9.3 Short-Term and Long-Term Pile Capacity in Cohesive Soil

The short-term pile capacity represents the capacity immediately after installation, while the long-term one represents the capacity after complete dissipation of the excess pore pressure around the pile. The dynamic installation process results in lower short-term capacity than quasi-static installation. Traditional driven pile installation is considered a quasi-static event since it occurs at relatively low penetration velocities when compared to dynamic pile impact velocities of 20-30 m/s expected in the field. The higher impact velocity leads to the lower short-term capacity. This is possibly due to the entrainment of water in the boundary layer close to the pile, resulting in lower effective stress.

A plot for evaluation of pile capacity regain vs non-dimensional time $T = c_h t/d^2$ is presented in Section 3, Figure 6 [Ref.9]. A preliminary assessment of the regain of the pile capacity Fv could be conservatively obtained using the following equation:

$$\frac{F_v - W_s}{F_{\max}} = 1.1 - \frac{1.08}{1 + \left(\frac{T}{6.5}\right)^{0.42}}$$

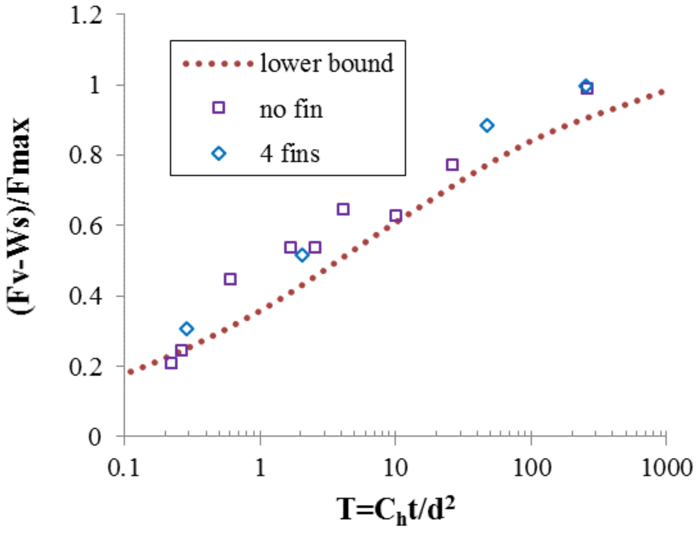
$$T = \frac{c_h t}{d^2}$$

where

W_s	=	submerged weight of the pile, in N (kgf, lbf)
$F_{\max}$	=	long term maximum pile capacity, in N (kgf, lbf)
T	=	non-dimensional time
t	=	consolidation time after installation, in sec
d	=	pile shaft diameter, in m (m, ft)
c_h	=	horizontal coefficient of consolidation, in m^2/year (m^2/year, ft^2/year)

Based on the consolidation data, for typical consolidation values of c_h = 3-30 m^2/year, the time required for 50% consolidation of a prototype dynamic pile with a shaft diameter of 1.2 m ranges from approximately 35–350 days, and 2.4-24 years to achieve 90% consolidation. Therefore, the dynamically installed piles should be installed for a sufficiently long period to allow the development of the pull-out capacity.

FIGURE 6
Regain of Normalized Pile Capacity with Consolidation Time



SECTION 4 Structural Assessments

1 General *(1 December 2017)*

This section provides guidance on the structural assessments of dynamically installed piles using Finite Element Analysis (FEA). The mesh density should be sufficient to assess both global and local structural strength and fatigue in the 3-dimensional FEA model. A local, very fine mesh model may be required when a structural detail or critical structural area needs to be analyzed. Structural idealization, load application, analysis procedures and evaluation of analysis results should be carried out in a consistent manner based on sound engineering practice. Inclination of the pile should be considered so that the pile is analyzed in the most critical direction, which leads to the maximum stress for structural assessment.

The strength assessment can be based on a "gross" scantling approach, wherein the nominal design thicknesses indicated on the drawings can be used in the FEA. To evaluate the pile's structure with reasonable accuracy, the FEA model should include the soil-pile interaction. The boundary conditions of the FEA model should comply with 3/7.3.

The method of calculating the anchor reverse catenary line is specified in Appendix 4. This method calculates the tension force along the mooring line and at the pile attachment point as well as the shape of the mooring line embedded in the soil.

3 Structural Model

In the finite element analysis, the pile should be modeled with a suitable non-linear stress-strain curve, indicating yield stress, ultimate strength, and strain hardening.

5 Element Types

In general, 3D solid elements may be used in the pile's FEA model, to simulate the geometry, configuration, mass and stiffness of the actual pile structures. However, where appropriate for structural elements, 2D plate, shell, and beam elements can also be used. The interface between two and three dimensional elements should be appropriately considered.

7 Loads

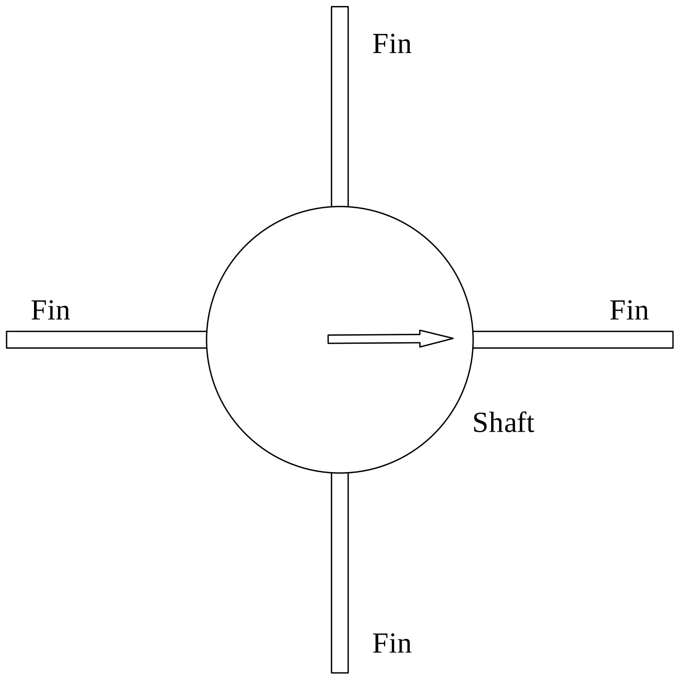
The most critical scenarios, including the magnitude of the load at the pile top and mooring load angle should be determined for the "*intact*" and "*broken line*" conditions, respectively, as defined in the Part 6 of the *FPI Rules*. In general, a mooring load applied in a horizontal direction in the elevation view generates the largest bending moment compared to the loads applied at an inclined angle. The inclination angle of the pile after installation is to be considered so that the pile is installed in the most critical direction which leads to the maximum stress.

For the structural evaluation of piles considering fins, pile top force is to be applied in a direction orthogonal to the fin in the plan view, aiming to produce the highest possible local stresses as shown in Section 4, Figure 1.

9 Fatigue and Yielding Check *(1 December 2017)*

The results of the finite element analyses of the structural models should be checked for yielding, see Appendix 3. The fatigue assessment should be performed to demonstrate the adequacy of the mooring line attachment components for the expected service life of the mooring system.

FIGURE 1
Direction of Pile Top Force to Produce Maximum Stress in a Fin



SECTION 5 Installation Recommendations

1 General

This Section contains recommendations during the installation of the pile. These recommendations apply to all piles covered by these Guidance Notes.

3 Installation Procedures Report and Records

A pile and mooring line installation procedures report should be prepared, which should include, but is not limited to, the following:

i) Rigging arrangements for the pile, monitoring devices, and all pre-assembled mooring parts such as studless chains, pre-assembled chains with triangular plate, shackles, release devices, eye hoist hooks with latch, buoys, all connecting shackles, and polyester fiber ropes that will be installed together with the pile.

ii) Precautions taken in order to prevent any twisting of the lines during installation, including procedures for installation and handling of chains and polyester fiber rope. See Subsection 4/9 of the ABS *Guide for the Certification of Offshore Mooring Chain*, Section 12 of the ABS *Guidance Notes on the Application of Fiber Rope for Offshore Mooring* and 8.2, 8.3 and 8.4 of API RP 2SM.

iii) Definition of removable monitoring device to locate planned marks on the mooring chain connected at the pile's padeye, so that an ROV can determine pile top depth and the configuration of the pile after installation.

iv) Verification of the proper calibration of the removable monitoring device

v) Layout of lines at planned depths for gravity installation and criteria for release device trigger including ROV surveying and positioning prior to pile release.

vi) Recovery of removable monitoring device and criteria for data analysis taking into consideration a contingency plan for pile inclination, pile azimuth and pile tip depth determination in case of a failed monitoring device and/or data recovery. Monitoring device recovery failure should be limited to a maximum of one pile per cluster and no more than three piles total.

vii) Pile penetration, inclination and holding capacity acceptance criteria established by design and pile refusal criteria.

viii) Tensioning of mooring line to be temporarily abandoned in the direction of moored unit's planned position with the aim of creating an inverse catenary configuration on buried mooring chain between pile pad-eye and seabed.

5 Pre-Installation Verification

Before installation, in addition to the recommendations contained in the Installation procedures report, the following items should be verified or witnessed, where applicable:

i) Pile over-boarding can be done by the vessel that carries it or with help of a support vessel through a wire rope connected to the pile tip. In the case of using two vessels for installation, the prevailing weather conditions should be considered to avoid twisting or excessive stress in the lines used to lower the piles and any attached mooring chain.

ii) Pile orientation before activation of the release device to allow pile azimuth calculation through monitoring device accelerometers.

iii) Procedures for ROV identification of pile top depth after installation. In case of limited visibility, alternative means of verifying the installation may be considered.

iv) Contingency plan for pile removal or abandonment, taking into consideration the prevailing weather conditions.

v) Mooring site seabed condition and contingency procedures for removing any obstacles found on site; the inspection should be carried out by ROV in acceptable underwater visibility.

vi) Survey of pile coordinates using Underwater Acoustic Positioning System (Long Baseline preferred due to better accuracy) made by ROV. Submersible buoys should be installed for site identification during installation.

vii) Installation vessel equipment including, but not limited to, winches, capacity of the winch brake, chain lockers, chains and polyester fiber rope storage capacity and cranes.

viii) Pile and line handling on board the installation vessel taking into consideration safety conditions.

ix) Examination of all mooring components for transit damages prior to installation; any damage found should be addressed.

x) Certification of all applicable components, including pile, showing that they are within the allowable design tolerance.

xi) In case of mooring chain abandonment at the seabed, maximum allowable environmental conditions for submersible buoys.

xii) Correct sizes and lengths of anchor chain segments and line identification procedures before rigging it to installation vessel.

xiii) Release device inspection to prevent accidental opening during installation.

xiv) Examination of the pre-assembled chain to be installed between the release device and the mooring chain, to witness the presence of the flounder plate.

xv) Setup and installation procedure of removable monitoring device.

7 Verification of Pile Installation

The installation of a dynamically installed pile should be monitored to make sure that the installation proceeds as expected and the pile is installed as designed. The measurements should include coordinates of piles installation, designated penetration depth, acceptable tolerances for pile position, actual penetration depth, inclination angle and azimuth. In case the installed pile position and/or penetration differ from the planned conditions, additional calculations and inspections should be performed to determine the specific ultimate holding capacity of the pile, and the as-installed mooring system’s capability to provide the required ultimate holding capacity.

APPENDIX 1 **Commentary on the Calculation of Pile Embedment Depth in Cohesive Soil**

1 General

Reliability of prediction of the embedment depth of a dynamically installed pile depends on the accurate determination of resistances acting on the pile during the penetration. The prediction should take into account strain rate effects on frictional and bearing resistances. Procedures are given below as guidance on determining the pile embedment depth following 3/3.1. An example calculation is also provided.

3 Procedure

To solve Equation 1 in Section 3, a finite difference approach can be used. The incremental acceleration, a_i can be expressed as:

$$a_i = \frac{W_s - F_{resistance,i} - F_{b,i} - F_{drag,i}}{m}$$

where

W_s	=	submerged weight of the pile
$F_{resistance,i}$	=	incremental upward force resisting movement of the pile through the soil, in N (kgf, lbf)
$F_{b,i}$	=	incremental buoyancy weight of the displaced soil, in N (kgf, lbf)
$F_{drag,i}$	=	incremental fluid drag force, in N (kgf, lbf)
m	=	submerged mass of the pile, in kg (kg, lb)
i	=	time step

The incremental penetration depth, which is defined as the embedment depth at the pile tip, can be determined using a central difference solution:

$$z_i = \Delta t^2 a_{i-1} + 2z_{i-1} - z_{i-2}$$

where

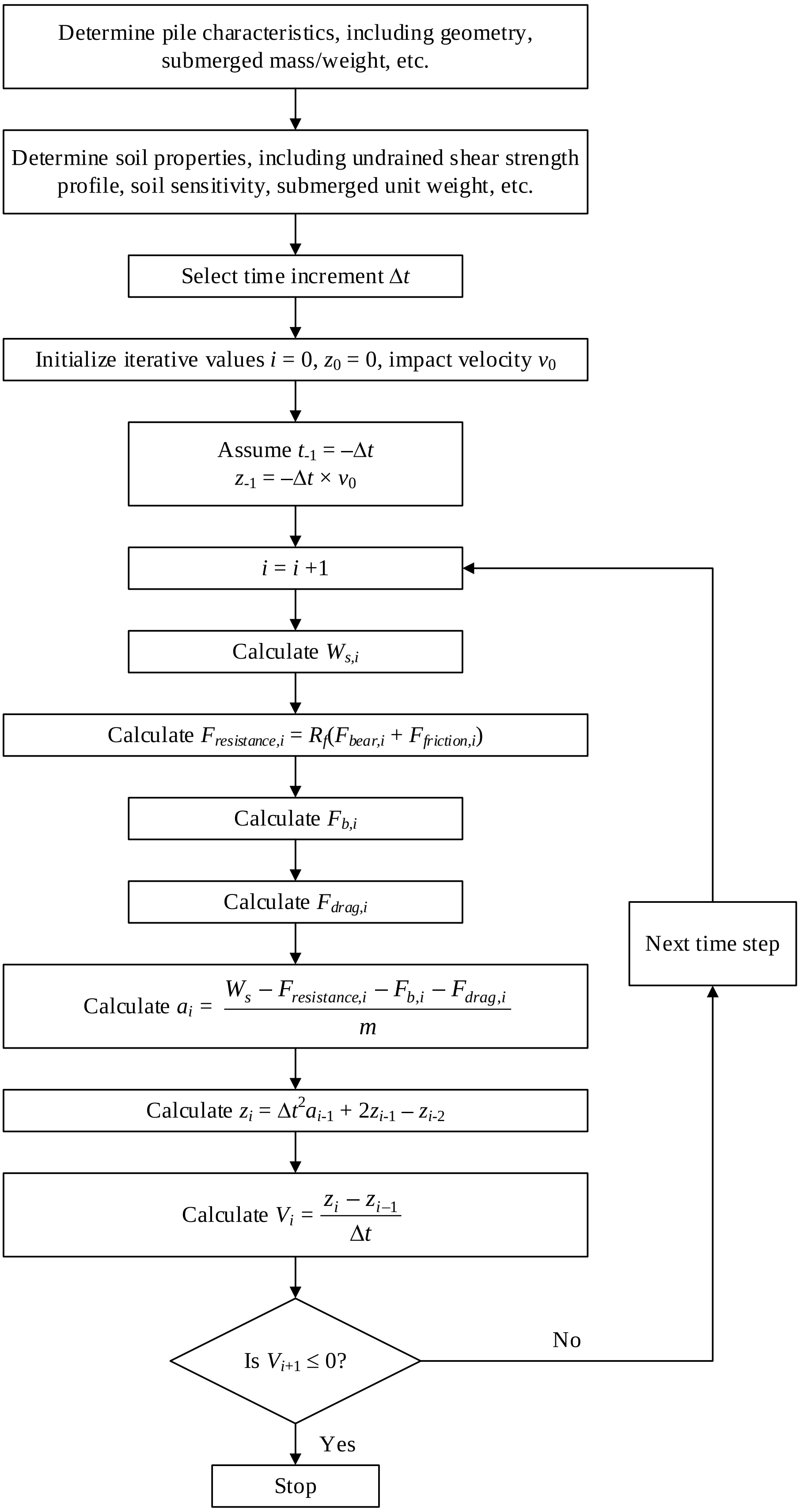
Δt = specified time increment

Velocity at each time step can be calculated as:

$$V_i = \frac{z_i - z_{i-1}}{\Delta t}$$

The final embedment depth is determined when the velocity is zero. If the velocity at the final step is a negative value, the embedment depth can be calculated by interpolation using the final two values of embedment depth and velocity. A flowchart for the procedure on determining the pile embedment depth is presented in Appendix, 1 Figure 1.

FIGURE 1
Flowchart for Procedure on Determining Pile Embedment Depth



5 Example of Determining Pile Embedment Depth

5.1 Pile Parameters

A typical lightweight pile is selected for the sample calculation with the pile parameters, as listed in Appendix 1, Table 1. The pile geometry is shown in Section 3, Figure 2.

TABLE 1
Pile Parameters

Parameter	*Symbol*	*Units*	*Value*
Number of fins	---	---	4
Pile Diameter	d	m	0.75
Pile Length	L	m	13.4
Submerged weight	W_s	kN	290
Impact Velocity	v_0	m/s	20

5.3 Soil Parameters

The pile is installed in very soft clay with the soil properties in Appendix 1, Table 2.

TABLE 2
Soil Parameters

Parameter	*Symbol*	*Units*	*Value*
Soil submerged unit weight	γ'	kN/m^3	6.0
Soil undrained shear strength at surface	$S_{u,f}$	kPa	0
Soil undrained shear strength gradient	k	kPa/m	1.8
Soil sensitivity	S_t	---	1, 4, 8
Strain rate parameter	β	---	0.06, 0.10, 0.136

5.5 Embedment Assessment

Appendix 1, Figure 2 shows the penetration resistance profile due to frictional resistance, bearing resistance, fluid drag force, and buoyancy weight of the displaced soil. The stain rate effect is taken into account using the parameter, R_f, which initially increases the frictional and bearing resistances by 1.67 times; see Appendix 1, Figure 3. The parameter R_f decreases as the pile velocity decreases at the late embedment stage, and finally becomes 1.0. The frictional resistance exceeds that from bearing once the full fin is embedded into the soil. Due to the relatively large cross-sectional area of the pile's fins, the final frictional resistance contributes significantly to the total resistance. The fluid drag force is only dominant at very shallow embedment depths.

Appendix 1, Figure 4 shows the velocity profile during pile embedment into the soil. The pile velocity slightly increases initially at shallow embedment due to the positive acceleration. The final embedment depth at pile tip is obtained when the velocity reduces to zero. The effects of strain rate and soil sensitivity are investigated. These are found to be key parameters that influence the pile's embedment depth. An increase in the strain rate parameter, β, increases the strain rate function, R_f; which then increases the pile's bearing and frictional resistances. Hence, a smaller β always leads to a relatively larger embedment depth. Soil sensitivity is the estimate of a soil's ability to maintain its original strength when remoulded. The remoulded undrained shear strength is normally defined as the undisturbed undrained shear strength divided by the sensitivity value. A highly sensitive soil means a lower remoulded shear strength, hence greater pile penetration depth. Therefore, accurate determination of β and S_t values from laboratory testing are prerequisites for reliable pile embedment depth prediction.

FIGURE 2
Penetration Resistance Profile

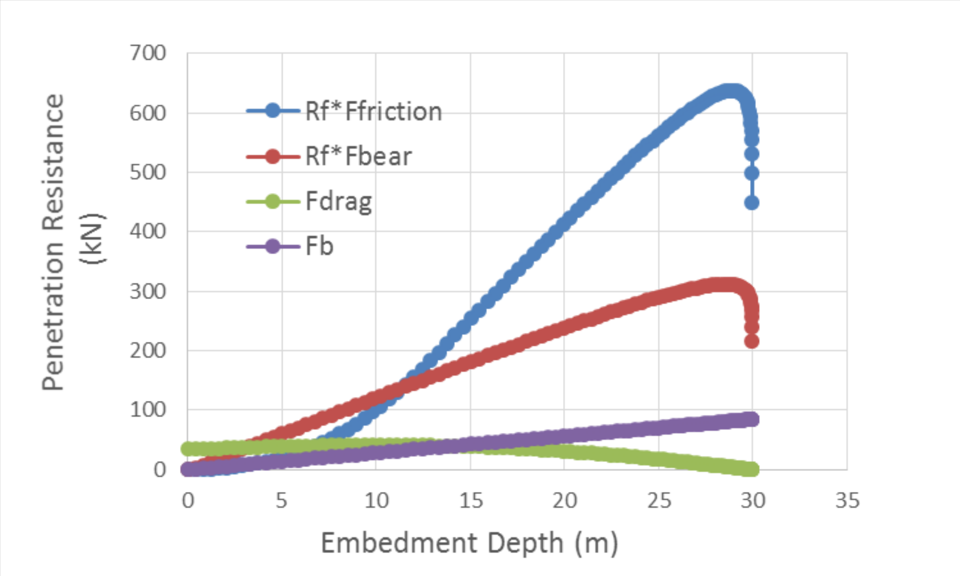


FIGURE 3
Strain Rate Function R_f vs Embedment Depth at Pile Tip

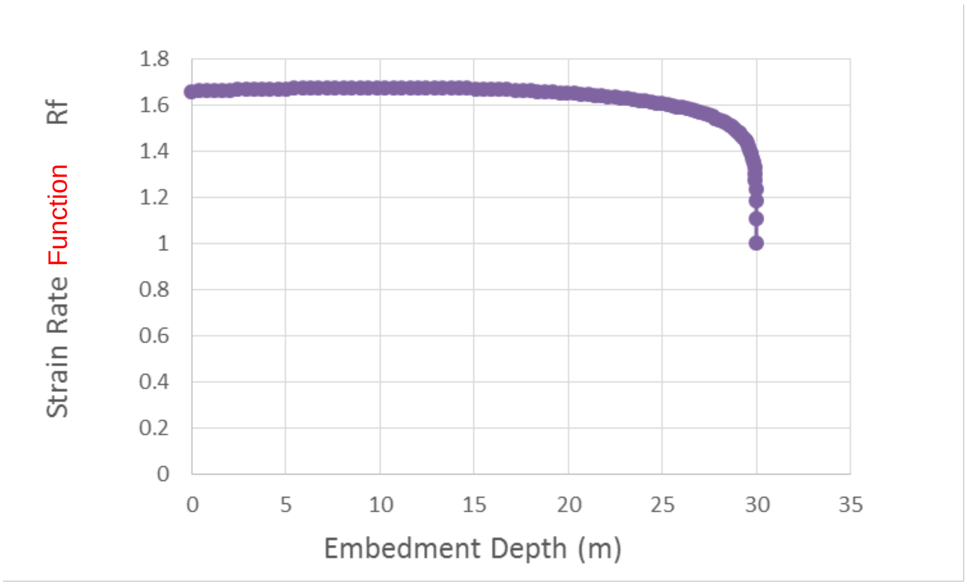
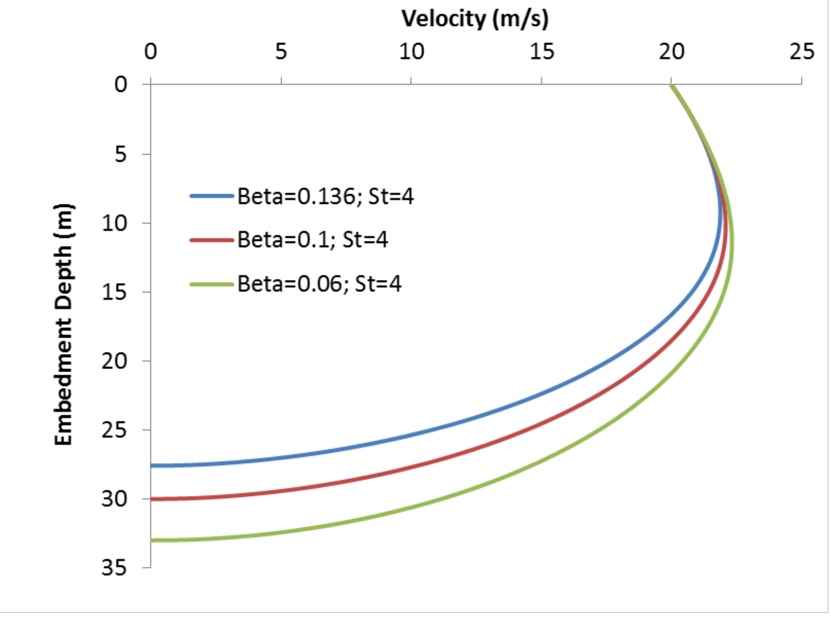
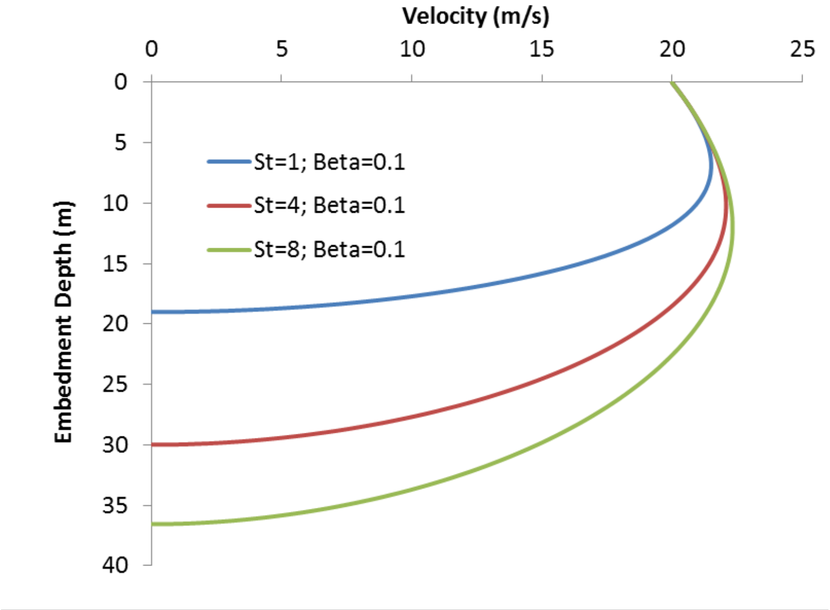


FIGURE 4
Pile Velocity Profiles vs Embedment Depth at Pile Tip



(a) Effect of Strain Rate Parameter β



(b) Effect of Soil Sensitivity S_t

APPENDIX 2 Cyclic Loading Effect

1 Background (1 December 2017)

Dynamically installed piles should withstand severe cyclic wave loading during storm conditions. Cyclic loading can affect the soil's undrained shear strength in two ways. The soil's undrained shear strength can increase due to the high loading rate from wave frequency load cycles. Conversely, there can be a degrading effect on the soil's undrained shear strength due to repeated cyclic loading. Since the mooring line is always in tension, there is less of a degradation effect on the shear strength for the one-way cyclic loading than for the two-way cyclic loading with load reversals. Therefore, the pile design should ensure that there is sufficient pile capacity to resist the mooring line tension, and to provide the required safety factor.

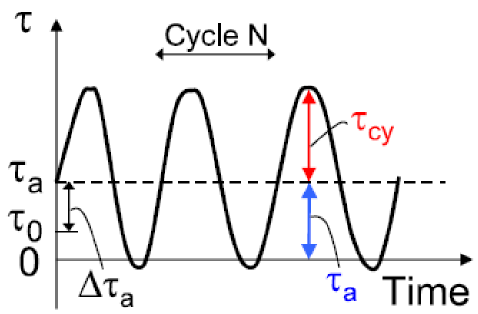
In order to take the loading rate and degradation effects into account the cyclic shear strength, $\tau_{f,cy}$, is to be determined. The fundamental work concerning cyclic soil parameters for offshore foundation design has been published by Andersen [Ref. 10].

This Appendix presents a guideline for cyclic loading effect assessment in soil design parameters adopted in 3/3.1. The pile capacity should be calculated using the cyclic shear strength, $\tau_{f,cy}$, which is discussed in Subsection A2/3 below.

3 Cyclic Shear Strength

Cyclic loadings generally tend to break down the soil structure and cause a tendency for volumetric compression. If the soil is saturated and the loading condition is undrained, the volumetric change is prevented. Instead there is an increase in pore water pressure and the effective stresses in the soil decreases accordingly. The soil cyclic behavior (e.g., strength and deformation characteristics) depend on both average and cyclic shear stress. The load cycles with a single amplitude shear stress, τ_{cy}, around a constant average shear stress, τ_a, is shown in Appendix 2, Figure 1.

FIGURE 1
Typical Cyclic Shear Stress



The cyclic shear strength, $\tau_{f,cy}$, is defined as the maximum shear stress that can be mobilized during the cyclic loading and it can be determined using the following equation [Ref.11].

$$\tau_{f,cy} = (\tau_a + \tau_{cy})_f$$

where

$(\tau_a + \tau_{cy})_f$ = sum of average and cyclic shear stress at failure, in kN/m^2 (kgf/m^2, lbf/ft^2)

τ_a = average shear stress, in kN/m^2 (kgf/m^2, lbf/ft^2)

τ_{cy} = cyclic shear stress amplitude, in kN/m^2 (kgf/m^2, lbf/ft^2)

The cyclic shear strength $\tau_{f,cy}$ depends on the average shear stress, τ_a, the cyclic loading history (i.e., number and magnitude of load cycles, load frequency), over-consolidation ratio OCR, grain size distribution, plasticity index, stress path etc. The cyclic degradation effect increases with the increase in the OCR of the clay. The calculations of the cyclic shear strength should be based on anisotropic, stress path dependent, undrained shear strengths.

There are three types of cyclic shear strength following various stress paths [e.g., compression, extension and direct simple shear (DSS)].

$\tau_{f,cy}^{com}$ = triaxial compression cyclic shear strength, in kN/m^2 (kgf/m^2, lbf/ft^2)

$\tau_{f,cy}^{ext}$ = triaxial extension cyclic shear strength, in kN/m^2 (kgf/m^2, lbf/ft^2)

$\tau_{f,cy}^{DSS}$ = DSS cyclic shear strength, in kN/m^2 (kgf/m^2, lbf/ft^2)

The average shear stress, τ_a, is calculated as follows.

$$\tau_a = \tau_0 + \Delta\tau_a$$

where

τ_0 = initial soil shear stress prior to the installation of structure, in kN/m^2 (kgf/m^2, lbf/ft^2), see Appendix 2, Figure 1

$\Delta\tau_a$ = addition soil shear stress induced by submerged weight of structure and/or average environmental loads, in kN/m^2 (kgf/m^2, lbf/ft^2), see Appendix 2, Figure 1

The difference between triaxial and DSS cyclic shear strength is that in the DDS test τ_0 = 0; while in the triaxial test,

$$\tau_0 = 0.5(1 - K_0)\, p'_0$$

where

K_0 = coefficient of earth pressure at rest

p'_0 = vertical effective overburden pressure, in kN/m^2 (kgf/m^2, lbf/ft^2)

Since the soil cyclic behavior depends on the loading history the cyclic shear strength is different in triaxial and DSS tests.

5 Procedure to Calculate Cyclic Shear Strength

5.1 Design Storm History *(1 December 2017)*

The cyclic load history should provide the mean tension and variation in line tension with time at the pile's padeye. The Design Storm is the sea state of specified duration with a return period of 100 years, characterized by spectral formulation, significant wave height and peak period. The duration of the storm depends on season and location, which produces the smallest cyclic shear strength. Based on the mooring line analysis for the storm condition it is possible to obtain the mean and design line tensions. Since a dynamically installed pile is usually used for a taut mooring, the parameter τ_a may be approximated using the following equation:

$$\frac{\tau_a}{s_u} = \frac{P_a}{P_{f.s}} \quad \text{(Eq.A2.1)}$$

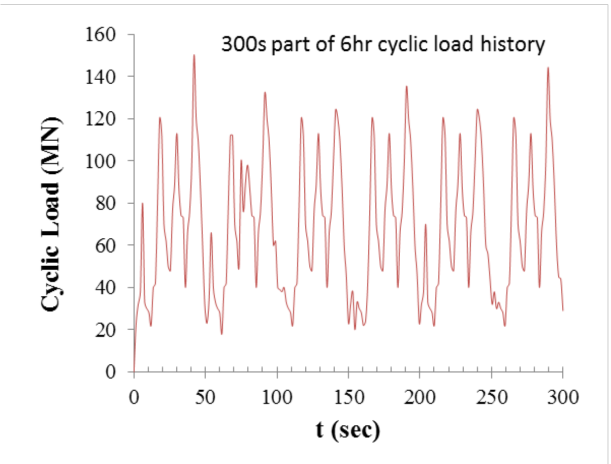
where

P_a = mean line tension, in kN (kgf, lbf)

$P_{f.s}$ = design line tension, in kN (kgf, lbf)

In a storm, the wave height and period vary continuously from one wave to another, as does the cyclic shear stress. To create the cyclic contour diagram for constant cyclic shear stress conditions, the irregular loading history needs to be transformed into a number of "parcels" with different constant cyclic loads. The irregular design cyclic load history can be transformed into parcels of constant cyclic load using the "rain flow" method as shown in in Appendix 2, Figure 2 [Ref.12].

FIGURE 2
Example of Transformation of Cyclic Loading History to Constant Cyclic Parcels



No of Cycles, N	Amplitude of Cyclic Load in Percentage of max. Cyclic Load (%)
1	100
1	96
2	90
4	80
30	70
92	60
175	50
340	40
542	30
623	18

The tabulated parcels in Appendix 2, Figure 2 are arranged in ascending order of the amplitude of cyclic load. Applying the loads in this order and with the maximum load at the end will cause the maximum cyclic degradation, which is conservative for pile capacity assessment.

5.3 Equivalent Number of Cycles to Failure *(1 December 2017)*

The laboratory tests are run with one constant cyclic shear stress amplitude throughout each test, and the soil strength for real storm loading should be predicted from tests with constant τ_{cy}. The equivalent number of cycles N_f is defined as the number of cycles of the maximum load that give the same cyclic effect as the real irregular cyclic load history. N_f can be determined either by the pore pressure accumulation procedure [Ref. 13] or the strain accumulation procedure [Ref. 14] depending on the soil's characteristics. The pore pressure accumulation procedure is the preferred procedure in cases where there is possible drainage during the cyclic load history, whereas the strain accumulation procedure may be more suited for clay soils [Ref. 10].

5.5 Cyclic Contour Diagram

The Contour Diagram provides a practical design basis of an offshore foundation. More details on the construction of a contour diagram and soil testing strategy can be found in [Ref.16]. The cyclic behavior of the soil can be represented in a contour diagram, where the cyclic parameters are given as functions of average and cyclic shear strength normalized by the monotonic undrained shear strength, and number of cycles to failure. The normally consolidated Drammen Clay is used as an example for DSS and triaxial tests, respectively, in Appendix 2, Figure 3 (a) and (b) from [Ref. 10].

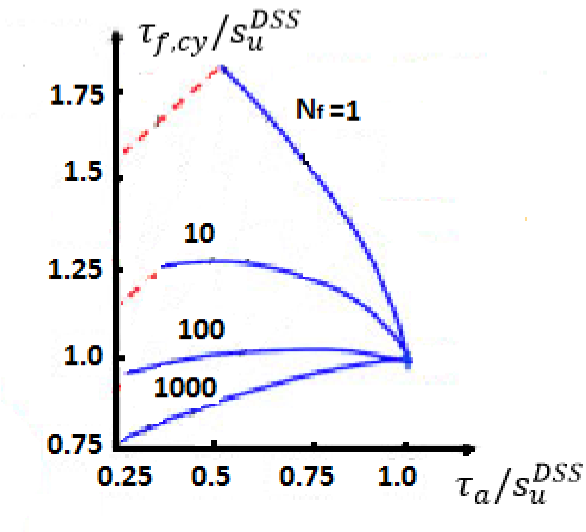
Databases with parameters for cyclic loading of clay are available for Drammen Clay [Ref.11&15], Troll Clay [Ref.16], and Gulf of Mexico Clay [Ref.17&18]. The databases may be used to plan testing programs, interpret test results, and reduce the number of site specific cyclic tests. It is recommended to acquire site specific cyclic test data to construct the contour diagram if the cyclic loading effect is to be accounted for in the design. If relevant cyclic test data for the site is not available, the designer should be conservative in the assessment of cyclic shear strength. Empirical data from databases and literature should be used with caution.

5.7 Recommended Procedure

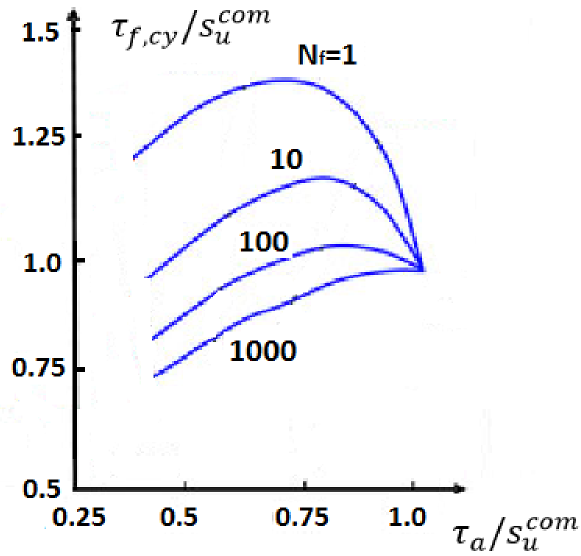
The following approach can be used to establish the permissible value of cyclic shear strength, $\tau_{f,cy}$:

1. From the irregular design cyclic load history, determine parcels of different constant cyclic loads using rain flow method.
2. Determine the equivalent number of cycles to failure, N_f, following either the pore pressure accumulation procedure or the strain accumulation procedure, whichever is more suitable.
3. Assume an initial value of $\tau_{f,cy}$.
4. Obtain a possible range for τ_a/S_u based on the mooring line analysis using Eq.A2.1.
5. Construct the cyclic contour diagram using site specific cyclic test data, supplemented with appropriate empirical data from recognized databases.
6. Determine cyclic shear strength, $\tau_{f,cy}$, using contour diagram.
7. Repeat Steps 4 through 6 with newly updated $\tau_{f,cy}$ until $\tau_{f,cy}$ converges to the previously assumed value.

FIGURE 3
Contour Diagram Showing Number of Cycles to Failure as a Function of Normalized Average and Cyclic Shear Stresses [Ref.10]



(a) DSS Test



(b) Triaxial Test

APPENDIX 3 Commentary on Acceptance Criteria

1 General

The acceptance criteria for a dynamically installed pile are specified in 6-1-2/8 of the *FPI Rules*. This Appendix gives the criteria for easy use and reference. Users are advised to check periodically on the ABS website www.eagle.org for the latest version of the *FPI Rules*.

3 Factor of Safety for Pile Holding Capacity

The factor of safety for holding capacity is defined as the pile's ultimate holding capacity divided by the maximum load obtained from a dynamic analysis that accounts for mooring line dynamics. Factors of safety for dynamically installed piles should be 2.0 for the *intact* condition and 1.5 for the *broken line* (*damaged*) condition.

The maximum mooring forces for intact and damaged conditions should be considered according to Section 6-1-1 of the *FPI Rules* or Section 3-4-1 of the *MODU Rules*. The dynamic analysis method should suitably determine the mooring line forces at touchdown point.

For a clustered area, the most critically loaded line should be selected as representative of the entire cluster for geotechnical calculation purposes. The reverse catenary line attached to the pile's padeye below the seabed should be determined, including the mooring line load on the top of pile and its orientation. The line angle should be based on unfactored pile loads without taking the safety factor into consideration.

5 Acceptance Criteria for Yielding

5.1 General

Provided the pile is free-flooding, the results of the finite element analyses of the structural models can be checked for yielding only. The evaluation of yielding of the pile structure should be based on the results of local fine mesh models where more accurate determination of local stress is obtained. Buckling evaluation can be omitted since the pile is under axial tension.

Fatigue evaluation of the main body of the pile may be omitted, since it should not experience significant cyclic loads. However, the mooring line attachment padeye or lug is a critical structural detail. The attachment padeye should be designed to satisfy yielding criteria, and should have full penetration welding to the pile structure. The fatigue assessment should be performed to demonstrate the adequacy of the mooring line attachment components for the expected service life of the mooring system, considering Fatigue Design Factor (FDF) of 10, since the pile structure is non-inspectable and non-repairable. Alternatively, in order to meet the fatigue resistance criteria without such detailed analyses, the padeye can be fabricated as an integral casting with the base structure.

5.3 Yielding

For a plate element subjected to biaxial stress, a specific combination of stress components, rather than a single maximum normal stress component constitutes the limiting condition. In this regard, the following equivalent stress σ_{eqv}, given by the Hencky von-Mises theory, should be compared to a maximum allowable percentage of the material's yield strength:

$$\sigma_{eqv} = [\sigma_X^2 + \sigma_Y^2 - \sigma_X\sigma_Y + 3\,\tau_{XY}^2\,]^{1/2}$$

where

σ_X = normal stress in the X direction (local axis system of the element), in kN/m^2 (kgf/m^2, lbf/ft^2)

σ_Y = normal stress in the Y direction, in kN/m^2 (kgf/m^2, lbf/ft^2)

τ_{XY} = shear stress, in kN/m^2 (kgf/m^2, lbf/ft^2)

or using principal stresses, σ_1 and σ_2:

$$\sigma_{eqv} = [\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2]^{1/2}$$

For a solid element subjected to triaxial stress, the following equivalent stress σ_{eqv}, given by the Hencky von-Mises theory, should be compared to a maximum allowable percentage of the material's yield strength:

$$\sigma_{eqv} = [\sigma_X^2 + \sigma_Y^2 + \sigma_Z^2 - \sigma_X\sigma_Y - \sigma_X\sigma_Z - \sigma_Y\sigma_Z + 3(\tau_{XY}^2 + \tau_{XZ}^2 + \tau_{YZ}^2)]^{1/2}$$

where

σ_X = normal stress in the X direction (local axis system of the element), in kN/m^2 (kgf/m^2, lbf/ft^2)

σ_Y = normal stress in the Y direction, in kN/m^2 (kgf/m^2, lbf/ft^2)

σ_Z = normal stress in the Z direction, in kN/m^2 (kgf/m^2, lbf/ft^2)

$\tau_{XY}, \tau_{XZ}, \tau_{YZ}$ = shear stress components, in kN/m^2 (kgf/m^2, lbf/ft^2)

or using principal stresses, σ_1, σ_2 and σ_3:

$$\sigma_{eqv} = [\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - \sigma_1\sigma_2 - \sigma_1\sigma_3 - \sigma_2\sigma_3]^{1/2}$$

The von-Mises stress (obtained from the finite element stress components), should not exceed the allowable stress specified in A3/5.5 below.

5.5 Allowable Yielding Stresses

The equivalent von Mises stress generated by applied loads should be limited to the following stresses for the *line broken* and *intact* conditions, respectively.

$\sigma_{eqv} \leq 0.9\sigma_{yield}$ (line broken)

$\sigma_{eqv} \leq 0.67\sigma_{yield}$ (intact)

where

σ_{yield} = yield stress of the considered structural component, in kN/m^2 (kgf/m^2, lbf/ft^2)

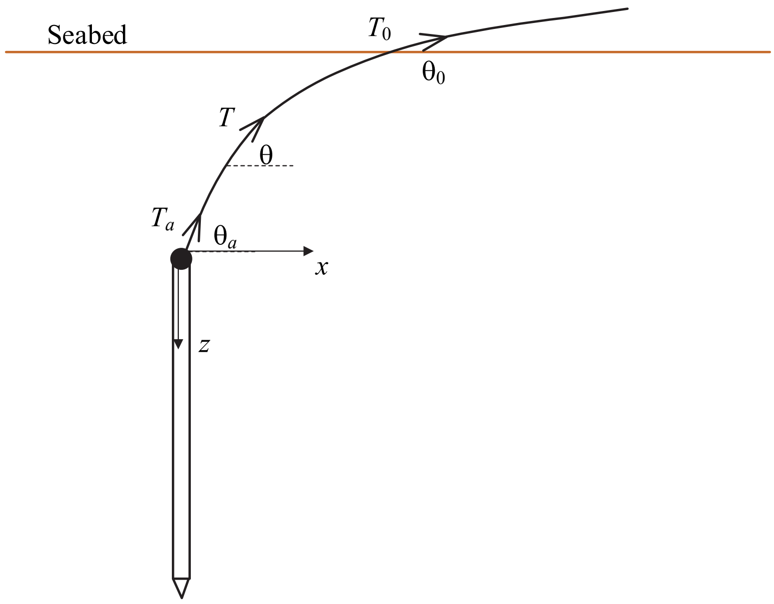
For those stresses of a highly localized nature, local yielding of the structure may be accepted provided it can be demonstrated that such yielding does not lead to progressive collapse of the overall structure and that the general structural stability is maintained.

APPENDIX 4 Methodology to Calculate the Anchor Reverse Catenary Line *(1 December 2017)*

1 General

Due to the normal resistance and friction generated by the soil, the portion of anchor line embedded in the soil will form a profile with a reverse catenary from the mudline to the attachment point, as illustrated in Appendix 4, Figure 1. Analysis of embedded anchor line performance is important for two reasons. First, the friction capacity of the anchor line itself can be a major component of the overall anchor capacity. Second, the anchor line angle at the attachment point determines the relative horizontal and vertical components of forces on the anchor. It is crucial to account for anchor line angle and tension at the attachment point during padeye structural design.

FIGURE 1
General Arrangement of Anchor Line *(1 December 2017)*



where

T_a	=	tension of the anchor line at the attachment point
θ_a	=	anchor line angle at the attachment point
T_0	=	tension of the anchor line at the mudline
θ_0	=	anchor line angle at the mudline
T	=	tension along the anchor line
θ	=	orientation of the anchor line to the horizontal

3 Equilibrium Equations of Embedded Anchor Line

The force equilibrium of an anchor line element is shown in Appendix 4, Figure 2. The differential equations for the embedded section of the anchor line element are:

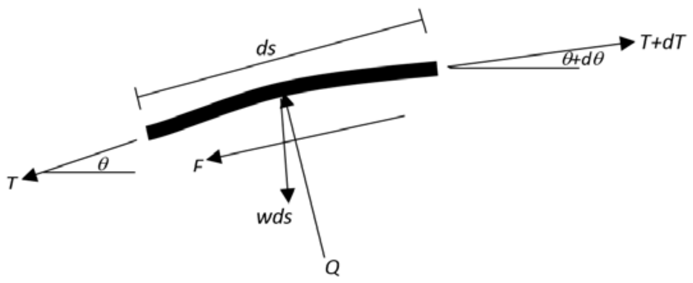
$$\frac{dT}{ds} = F + w \sin \theta \quad \text{(Eq.A4.1)}$$

$$T\frac{d\theta}{ds} = -Q + w \cos \theta \quad \text{(Eq.A4.2)}$$

where

s	=	length measured along the anchor line
F	=	resistance offered by the soil tangential to the anchor line (per unit length)
w	=	anchor line self-weight per unit length
Q	=	resistance offered by the soil normal to the anchor line (per unit length)

FIGURE 2
Force Equilibrium of Anchor Line Element *(1 December 2017)*



The friction force F on the anchor line is calculated from the following equation:

$F = E_t d \alpha s_u$ for cohesive soils

$F = E_t d \tan \delta$ for noncohesive soils .... (Eq.A4.3)

The normal force, Q, on the anchor line is calculated from the following equation:

$Q = E_n d N_c s_u$ for cohesive soils

$Q = E_n d N_q \gamma' z'$ for noncohesive soils .... (Eq.A4.4)

$F = \mu Q$ .... (Eq.A4.5)

where

E_t	=	tangential factor, correlating effective widths in the tangential direction, depend on the configuration of the anchor line, see Appendix 4,Table 1
d	=	nominal diameter of chain, or diameter of wire or rope
α	=	adhesion factor for anchor line
s_u	=	undrained shear strength at that position (average, or as measured in simple shear s_{uDSS})
δ	=	interface friction angle at soil-anchor line interface

E_n = normal factor, correlating effective widths in the normal direction, see Appendix 4, Table 1

N_c = bearing capacity factor, typically in the range of 7.6 to 14, depend on buried depth, shape and orientation, etc.

N_q = bearing capacity factor, depending on the friction angle

$$= \exp(\pi \tan \varphi)\tan^2\left(45° + \frac{\varphi}{2}\right)$$

γ' = effective unit weight of the soil

z' = embedment depth of the anchor line from the mudline

μ = friction coefficient between anchor line and soil, the value is usually in the range of 0.4-0.6.

TABLE 1
Effective Surface and Bearing Area for Anchor Line *(1 December 2017)*

	Chain	*Wire/Rope*
E_t	8-11	3.14
E_n	2.5	1

5 Simplified Solution for the Mooring Catenary Line

The governing equation Eq.A4.1 and Eq.A4.2 can be solved numerically with an iterative scheme. According to the study by Neubecker and Randolph [Ref.19], the self-weight of the anchor line has negligible effect on the chain profile and tension distribution when used in hard soils. The equilibrium equation and the differential equation can be simplified to give the tension profile as:

$$T = T_a e^{\mu(\theta_a - \theta)} \quad \text{(Eq.A4.6)}$$

$$-\frac{T_a}{1+\mu^2}\left[e^{\mu(\theta_a-\theta)}(\cos\theta + \mu\sin\theta)\right]_{\theta}^{\theta_a} = \int_{z'}^{D} Q dz' \quad \text{(Eq.A4.7)}$$

The left hand side of Eq.A4.7 may be simplified for small values of θ, to give:

$$\frac{T_a}{2}\left(\theta_a^2 - \theta^2\right) = (D - z')Q_{ave} \quad \text{(Eq.A4.8)}$$

where

D = buried depth of anchor attachment point

Q_{ave} = average bearing resistance per unit length of anchor line over the soil depth, D

The shape of the reverse catenary line can be derived as:

- For the case of uniform soil, the shape of the reverse catenary line is:

$$\frac{x^*}{\sqrt{2T^*}} = \left(\sqrt{\frac{T^*\theta_0^2}{2}+1} - \sqrt{\frac{T^*\theta_0^2}{2}+z^*}\right) \quad \text{(Eq.A4.9)}$$

when $\theta_0 = 0$, the equation can be written as:

$$z^* = \left[1 - \left(\frac{x^*}{\sqrt{2T^*}}\right)\right]^2 \quad \text{(Eq.A4.10)}$$

- For the case in which the bearing resistance of the soil increases proportionally with depth (s_{u0} = 0):

  $$Q = kz' \quad \text{(Eq.A4.11)}$$

  The shape of the reverse catenary line is:

  $$\sqrt{\frac{2}{T^*}}x^* = \ln\left[\frac{1+\sqrt{\frac{T^*\theta_0^2}{2}+1}}{z^*+\sqrt{\frac{T^*\theta_0^2}{2}+(z^*)^2}}\right] \quad \text{(Eq.A4.12)}$$

  when $\theta_0 = 0$, the equation can be written as:

  $$z^* = e^{-x^*\sqrt{2/T^*}} \quad \text{(Eq.A4.13)}$$

- Another approach proposed by Aubeny et al., 2011 [Ref. 20] for the bearing resistance of the soil increasing proportionally with depth (s_{u0} = 0):

  $$x^* = \sqrt{\frac{1}{2Q_2}}\ln\left[\frac{Q_2 + Q_1/2 + \sqrt{Q_2^2 + Q_1Q_2 + Q_2\,\theta_0^2/2}}{Q_2 z^* + Q_1/2 + \sqrt{Q_2^2(z^*)^2 + Q_1Q_2 z^* + Q_2\,\theta_0^2/2}}\right] \quad \text{(Eq.A4.14)}$$

where

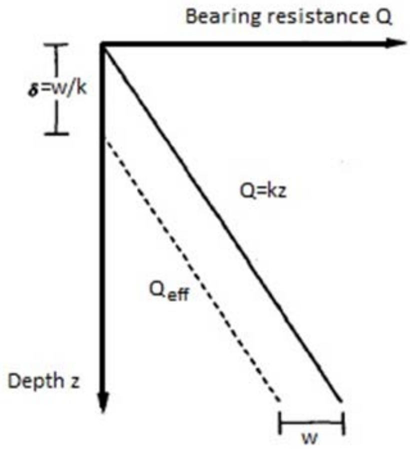
x	=	horizontal length of the anchor line from anchor
x^*	=	x/D
z^*	=	z'/D
T^*	=	normalized tension
	=	$\dfrac{T_a}{DQ_{ave}}$
k	=	gradient of bearing resistance with depth
s_{u0}	=	undrain shear strength at mudline
Q_1	=	normalized soil resistance due to mudline strength
	=	$\dfrac{E_n N_c d s_{u0} D}{T_a}$
Q_2	=	normalized soil resistance due to strength gradient
	=	$\dfrac{E_n N_c d k D^2}{2T_a}$

In soft soil with a heavy anchor line, the self-weight of anchor line is balanced by the bearing resistance of the soil. The analytical results can be applied to anchor line with weight by assuming an effective bearing resistance per unit length. Appendix 4, Figure 3 shows the effective bearing resistance profile, Q_{eff}, and the effective embedded depth, D_{eff}.

$$Q_{eff} = Q - w \quad \text{(Eq.A4.15)}$$

$$D_{eff} = D - \delta = D - w/k \quad \text{(Eq.A4.16)}$$

FIGURE 3
Soil Strength Adjustment to Account for Anchor Line Weight *(1 December 2017)*



In this case, the analytical solution for the anchor line profile is obtained from Eq. A4.8 and Eq.A4.11 with the following updated T^* and Q_{ave}:

$$T^* = \frac{T_a}{(D-\delta)Q_{ave}} \qquad \text{(Eq.A4.17)}$$

The load development equation is:

$$T \approx T_a e^{\mu(\theta_a - \theta)} + \mu\omega\left(x + \frac{D_{eff}\,\theta_a}{4}\right) \qquad \text{(Eq.A4.18)}$$

For the bearing resistance of soil increasing proportionally with depth:

$$Q_{ave} = \frac{k(D-\delta)}{2} \qquad \text{(Eq.A4.19)}$$

The simplified solution allows for an instant appraisal of the length of submerged anchor line, the tension and inclination of the chain at attachment point or at the mudline. Although strictly valid only for small *ds* and θ, Neubecker and Randolph [Ref.19] reported reasonable agreement with more rigorous solutions.

In order to yield reliable predictions, the results should be calibrated against well controlled and instrumented test data.

7 Description of Procedure

The following approach can be used to predict the anchor line tension, T_a, and anchor line angle, θ_a, at padeye/shackle:

i) Select mooring pattern, line configuration, anchor model and size.

ii) Determine the maximum line tension, T_0, and anchor line angle at seabed, θ_0, for design environmental condition for both intact case and damaged case with one broken line.

iii) Determine the anchor penetration depth, *D*.

iv) Assume an anchor line angle at padeye/shackle θ_a.

v) Determine the anchor line tension, T_a, at padeye/shackle using Eq.A4.18.

vi) Determine the anchor line angle at padeye/shackle, θ_a, using Eq.A4.8.

Repeat steps *iv)* through *vi)* with newly updated θ_a until θ_a is consistent with the assumed value in *iv)*.

APPENDIX 5 References

1. True, D.G. 1976. Undrained vertical penetration into ocean bottom soils, PhD thesis, University of California, Berkeley.

2. Loughlin, O., Richardson, M.D., Randolph, M.F. and Gaudin C. 2013. Penetration of dynamically installed anchors in clay. *Geotechnique* 63 (11): 909-919.

3. Visintini, L. 1982. Hydrodynamic analysis and design of high level radioactive waste disposal model penetrators," *Memorandum to C. C. R. EURATOM Ispra* from Aermacchi spa, Memo 419/STE/82, 1-63.

4. Skempton, A. W. 1951. The bearing capacity of clays. *Proceedings of the building research congress*, London, Vol. 1, pp.180–189.

5. API RP 2A, 2005. Recommended practice for planning, designing and constructing fixed offshore platforms, *21st Edition, American Petroleum Institute*, Washington, D.C.

6. Jeanjean, P. 2006. Setup characteristics of suction anchors for soft Gulf of Mexico clays: experience from field installation and retrieval. *OTC 18005*.

7. Mitchell, J.K. 1960. Fundamental aspects of thixotropy in soils. *ASCE J. of Geotech. Eng.*, Vol. 86, No. SM3, June 1960.

8. Randolph, M.F. 2003. Science and empiricism in pile foundation design. *Geotechnique*, 53(10), 847-875.

9. Richardson, M.D., O'Loughlin, C.D., Randolph, M.F. and Gaudin, C. 2009. Setup following installation of dynamic anchors in normally consolidation clay. *Journal of Geotechnical and Geoenvironmental Engineering*. ASCE. pp.487-496.

10. Andersen, K.H. 2015. Cyclic soil parameters for offshore foundation design. *Frontiers in Offshore Geotechnics*: ISFOG, Oslo, Norway, pp.5-82.

11. Andersen, K.H. & Lauritzsen, R. 1988. Bearing capacity for foundations with cyclic loads. *ASCE, J. of Geotech. Engineering*, 114 (5): 540–555.

12. ASTM E1049-85. 2011. Standard practices for cycle counting in fatigue analysis, *ASTM International*, West Conshohocken, PA. www.astm.org.

13. Andersen, K.H., Allard, M.A. & Hermstad, J. 1994. Centrifuge model tests of a gravity platform on very dense sand II. *Interpretation. 7th Int. Conf. on Behavior of Offshore Structures. BOSS'94*. Cambridge, Mass. Proc. (1): 255–282.

14. Andersen, K.H. 1976. Behavior of clay subjected to undrained cyclic loading. *Int. Conf. on Behaviour of Offsh. Struct., BOSS'76*. Trondh. Proc. (1): 392–403. Also NGI Pub. 114.

15. Andersen, K.H., A. Kleven and D. Heien. 1988. Cyclic soil data for design of gravity structures. *J of Geotech. Engineering., ASCE*, Vol. 114, No. 5, pp. 517-539.

16. By, T. and E. Skomedal. 1992. Soil parameters for foundation. *Int. Conf. on the Behaviour of Offshore Structures*, London, Proc., Vol. 2, pp 909-920.

17. Dutt, R.N., E.H. Doyle and R.S. Ladd. 1992. Cyclic behaviour of a deepwater normally consolidated clay. *V Int. Conf. on Civil Engineering. in the Oceans*, Texas, Proc., pp. 546-559.

18. Jeanjean. P, Andersen K.H. and Kalsnes B. 1998. Soil parameters for design of suction caissons for Gulf of Mexico deepwater clays. *Offshore Technology Conference*, Paper OTC 8830, pp. 505-519. Houston.

19. Neubecker, S.R. and Randolph, M.F. 1995. Profile and frictional capacity of embedded anchor chains. *Journal of Geotechnical Engineering* 121(11): 797-803.

20. Aubeny C, Gilbert R., Randall R., Zimmerman E., McCarthy K., Chen C-H, Aaron D., Yeh P., Chi C-M. and Beemer R. 2011. The performance of Drag Embedment Anchors (DEA). Final Project Report, prepared for ABS Consulting, OTRC Project 32558-A6960.