Design and Installation of Fluke Anchors

MAY 2012

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FOREWORD

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D) Systems
E) Special Facilities
F) Pipelines and Risers
G) Asset Operation
H) Marine Operations
J) Cleaner Energy
O) Subsea Systems
CHANGES

General
This document supersedes DNV-RP-E301, May 2000.

Text affected by the main changes in this edition is highlighted in red colour. However, if the changes involve a whole chapter, section or sub-section, normally only the title will be in red colour.

Main Changes

• General
Total revision of the document with same title from May 2000; i.e. all text is considered new text, and appears as clean black text. In addition to further clarification of the previous text the following have been added:
— New design chart allowing for anchor drag under certain conditions.
— Tentative guidance for design of fluke anchors in sand.
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1. General

1.1 Introduction

General references are found in Section 7 and given the format /no./.

This Recommended Practice features a substantial part of the design procedure developed in Part 1 /1/ of the joint industry project (JIP) on Design procedures for deep water anchors, and it was developed further through a pilot reliability analysis in Part 2 /2/. An overview of this JIP is given in /3/.

The experience gathered through a more recent JIP, which focused on the analytical procedure for design of fluke anchors in clay /13/, has led to significant improvements, which have been implemented in this revision of the RP.

The experience gathered through a number of anchoring projects for mobile drilling units and production platforms has also been considered.

1.2 Scope and Application

This Recommended Practice applies to the geotechnical design and installation of fluke anchors in clay for catenary mooring systems. However, the principles for design and installation of fluke anchors are applicable also to other types of soil; see Section 5.2.3 and Appendix B. The basis for calculation of the minimum anchor installation tension, which meets the governing safety requirements, is addressed in Section 5.4.

The design procedure outlined is a recipe for how fluke anchors in both deep and shallow waters can be designed to satisfy the requirements by DNV.

According to this recommendation the geotechnical design of fluke anchors shall be based on the limit state method of design. For intact systems the design shall satisfy the Ultimate Limit State (ULS) requirements, whereas anchor resistance following a one-line failure shall be treated as an Accidental Damage Limit State (ALS) condition.

For the anchors in a mooring system to satisfy the safety requirements, the anchor drag must be tolerable both during installation and during the governing design event. In Section 5.2.3 the focus is set on the significance of the soil conditions for the potential consequences of anchor drag during extreme environmental events.

If anchor drag may lead to unacceptable consequences for various reasons, as discussed in Section 5.3, the prediction of anchor drag during the ULS or ALS condition becomes a design issue.

The anchor failure related to excessive drag has been defined as either of the following events:

— Anchor failure
  Continuous anchor drag experienced before the required anchor resistance is reached;
— Excessive additional drag
  The additional drag required to resist the design tension in any of the lines leads to a breach of the safety factor with regard to breaking strength of the adjacent mooring lines;
— Threat to adjacent installations
  Predicted anchor drag length violates the safety distance between the moored structure/ anchor/ mooring line and adjacent structures after dragging;

The line tension model adopted herein splits the tension in a mean and a dynamic component; see background in /4/ and /5/.

Traditionally, fluke anchors have been designed with the mandatory requirement that the anchor line has to be horizontal (zero uplift angle) at the seabed level during installation and operation of the anchors. This requirement imposes significant limitations on the use of fluke anchors in deeper waters, and an investigation into the effects of uplift on fluke anchor behaviour, as reported in /1/, has provided a basis for assessment of an acceptable uplift angle.

The design rule presented herein has been calibrated based on reliability analysis of one test case as documented in /9/. The partial safety factors are considered to be tentative until further calibrations have been carried out.

This recommendation is in principle applicable to both long term (permanent) and temporary (mobile) moorings.

1.3 Structure of the RP

Definition of the main components of a fluke anchor is given in Section 2, followed by a description of the general behaviour of fluke anchors in clay in Section 3.

A brief overview of fluke anchor design methodologies is presented in Section 4.

The recommended procedure for design and installation of fluke anchors is presented in Section 5. The close and important relationship between the assumptions for design and the consequential requirements for the installation of fluke anchors is emphasized.

General requirements to soil investigations are given in Section 6 and in Appendix G.
The intention has been to make the procedure as concise as possible, but still detailed enough to avoid misinterpretation or misuse. For transparency, details related to the various design aspects are therefore found in the appendices.

A number of Guidance notes have been included as an aid in modelling of the anchor line, the anchor and the soil. The guidance notes have been written on the basis of the experience gained through the joint industry projects, see /1/, /2/, /3/ and /13/, and practical design and installation experience.

1.4 Definitions

<table>
<thead>
<tr>
<th>Term</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dip-down point</td>
<td>Point where the anchor line starts to embed.</td>
</tr>
<tr>
<td>Fluke</td>
<td>Main anchor load bearing component (see Figure 2-1)</td>
</tr>
<tr>
<td>Fluke angle</td>
<td>Angle between the fluke plane and a line passing through the rear of the fluke and the shackle (arbitrary definition).</td>
</tr>
<tr>
<td>Forerunner</td>
<td>Anchor line segment being embedded in the soil.</td>
</tr>
<tr>
<td>Inverse catenary</td>
<td>The curvature of the embedded part of the forerunner.</td>
</tr>
<tr>
<td>Shackle</td>
<td>Forerunner attachment point (at the front end of the shank).</td>
</tr>
<tr>
<td>Shank</td>
<td>Rigid connection between the fluke and the shackle (see Figure 2-1).</td>
</tr>
<tr>
<td>Touch-down point</td>
<td>Point where the anchor line first touches the seabed.</td>
</tr>
</tbody>
</table>

1.5 Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Term</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>AHV</td>
<td>Anchor handling vessel</td>
<td>Used to set the anchors</td>
</tr>
<tr>
<td>ALS</td>
<td>Accidental Damage Limit State</td>
<td></td>
</tr>
<tr>
<td>CC1, CC2</td>
<td>Consequence class 1 or 2</td>
<td>Consequence class with respect to failure, see Section 5.3.</td>
</tr>
<tr>
<td>MBL</td>
<td>Minimum Breaking Load</td>
<td>Breaking load of anchor line segment</td>
</tr>
<tr>
<td>MODU</td>
<td>Mobile Offshore Drilling Unit</td>
<td></td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate Limit State</td>
<td></td>
</tr>
</tbody>
</table>

1.6 Symbols and explanation of terms

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Term</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>α</td>
<td>Seabed uplift angle</td>
<td>Line angle with the horizontal at the dip-down point</td>
</tr>
<tr>
<td>α_{max}</td>
<td>Maximum possible uplift angle</td>
<td>Uplift angle, which makes the anchor drag at constant tension without further penetration at the actual depth</td>
</tr>
<tr>
<td>α</td>
<td>Anchor adhesion factor</td>
<td>Accounts for remoulding of the clay in the calculation of the frictional resistance at the anchor members</td>
</tr>
<tr>
<td>α_{min}</td>
<td>Minimum adhesion</td>
<td>Set equal to the inverse of the sensitivity, ( \alpha_{min} = 1/S_i )</td>
</tr>
<tr>
<td>α_{tr}</td>
<td>Thixotrophy strength ratio</td>
<td>( \alpha_{tr} = s_{u,th}/s_{u,r} ) after a given period of setup</td>
</tr>
<tr>
<td>α_{soil}</td>
<td>Line adhesion factor</td>
<td>To calculate unit friction in clay of embedded anchor line</td>
</tr>
<tr>
<td>A_{fluke}</td>
<td>Anchor fluke area</td>
<td>Based on manufacturer's data sheet.</td>
</tr>
<tr>
<td>AB</td>
<td>Effective bearing area</td>
<td>Per unit length (related to anchor line segment in the soil)</td>
</tr>
<tr>
<td>AS</td>
<td>Effective surface area</td>
<td>Per unit length (related to anchor line segment in the soil)</td>
</tr>
<tr>
<td>β</td>
<td>Anchor penetration direction</td>
<td>Angle of the fluke plane with the horizontal</td>
</tr>
<tr>
<td>c_v</td>
<td>Coefficient of consolidation</td>
<td>See Appendix G</td>
</tr>
<tr>
<td>d</td>
<td>Nominal diameter</td>
<td>Diameter of wire, rope or chain</td>
</tr>
<tr>
<td>DR</td>
<td>Disturbance Ratio</td>
<td>( = s_d/s_{u,d} ) (equal to or less than ( S_i ))</td>
</tr>
<tr>
<td>ds</td>
<td>Element length</td>
<td>Related to embedded anchor line calculation</td>
</tr>
<tr>
<td>e</td>
<td>Lever arm</td>
<td>Between shackle and the line of action of the normal resistance at the fluke</td>
</tr>
<tr>
<td>f</td>
<td>Unit friction</td>
<td>Resistance, both frictional and cohesive, along embedded part of anchor line or anchor</td>
</tr>
<tr>
<td>γ_m</td>
<td>Partial safety factor on anchor resistance</td>
<td>Accounts for the uncertainty in ( \Delta R_{setup}, \Delta R_{cy}, \Delta R_{fric}, s_u, s_{u,di} ) and ( s_{u,r} )</td>
</tr>
<tr>
<td>γ_{m,i}</td>
<td>Partial safety factor on seabed friction</td>
<td>Accounts for the uncertainty in the predicted seabed friction to be overcome during anchor installation</td>
</tr>
<tr>
<td>γ_{mean}</td>
<td>Partial safety factor on mean line tension</td>
<td>Accounts for the uncertainty in mean line tension</td>
</tr>
<tr>
<td>Symbol</td>
<td>Term</td>
<td>Explanation of term</td>
</tr>
<tr>
<td>----------</td>
<td>-------------------------------------------</td>
<td>-------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>( \gamma_{\text{dyn}} )</td>
<td>Partial safety factor on dynamic line tension</td>
<td>Accounts for the uncertainty in dynamic line tension</td>
</tr>
<tr>
<td>( L_F )</td>
<td>Fluke length</td>
<td>Related to fluke area: ( L_F = 1.25 \sqrt{A_{\text{fluke}}} ) (approximation)</td>
</tr>
<tr>
<td>( L_s )</td>
<td>Line length on seabed</td>
<td>For the actual mooring line configuration and characteristic line tension ( T_C )</td>
</tr>
<tr>
<td>( L_{s,i} )</td>
<td>Line length on seabed at anchor installation</td>
<td>For the anchor installation configuration between stern roller and anchor shackles, and the installation tension ( T_{\text{min}} )</td>
</tr>
<tr>
<td>( \mu )</td>
<td>Coefficient of seabed friction</td>
<td>Average friction coefficient (both frictional and cohesive) over line length ( L_s ) or ( L_{s,i} )</td>
</tr>
<tr>
<td>( n )</td>
<td>Exponent</td>
<td>Used in empirical formula for loading rate effect</td>
</tr>
<tr>
<td>( N_c )</td>
<td>Bearing capacity factor for clay</td>
<td>Corrected for relative depth of embedment, layering, orientation of respective anchor members, etc</td>
</tr>
<tr>
<td>( N_{\text{eqv}} )</td>
<td>Equivalent number of cycles to failure</td>
<td>The number of cycles at the maximum cyclic shear stress amplitude that will give the same effect as the actual cyclic load history (see Appendix E)</td>
</tr>
<tr>
<td>( OCR )</td>
<td>Overconsolidation ratio</td>
<td>Ratio between maximum past and present effective vertical stress on a soil element</td>
</tr>
<tr>
<td>( q )</td>
<td>Normal stress</td>
<td>Related to embedded anchor line</td>
</tr>
<tr>
<td>( \theta )</td>
<td>Orientation of anchor line at anchor shackle</td>
<td>( \theta = 0 ) for a horizontal anchor line</td>
</tr>
<tr>
<td>( Q_1, Q_2 )</td>
<td>Pile resistance</td>
<td>Pile resistance at loading rates ( v_1 ) and ( v_2 ), respectively</td>
</tr>
<tr>
<td>( R )</td>
<td>Anchor resistance</td>
<td>Resistance in the line direction with reference to penetration depth ( z ) and including the contribution from the embedded anchor line up to the dip-down point.</td>
</tr>
<tr>
<td>( \Delta R_{\text{cons}} )</td>
<td>Consolidation effect</td>
<td>Added to ( R_i ).</td>
</tr>
<tr>
<td>( R_{\text{setup}} )</td>
<td>Anchor resistance including setup effects</td>
<td>Anchor resistance at the dip-down point, including thixotropy or consolidation effects</td>
</tr>
<tr>
<td>( \Delta R_{\text{cy}} )</td>
<td>Cyclic loading effect</td>
<td>Added to ( R_{\text{cons}} )</td>
</tr>
<tr>
<td>( R_{\text{cy}} )</td>
<td>Cyclic anchor resistance</td>
<td>Anchor resistance at the dip-down point, including effects of consolidation and cyclic loading</td>
</tr>
<tr>
<td>( R_C )</td>
<td>Characteristic anchor resistance</td>
<td>Anchor resistance at the touch-down point with effects of consolidation, cyclic loading and seabed friction included</td>
</tr>
<tr>
<td>( R_d )</td>
<td>Design anchor resistance</td>
<td>With specified partial safety factors included (( R_{d,CC1} ) for CC1 conditions and ( R_{d,CC2} ) for CC2 conditions)</td>
</tr>
<tr>
<td>( \Delta R_{\text{drag}} )</td>
<td>Increase in resistance due to anchor drag</td>
<td>For a given additional anchor drag length</td>
</tr>
<tr>
<td>( \Delta R_{\text{fric}} )</td>
<td>Seabed friction</td>
<td>Over line length ( L_s ).</td>
</tr>
<tr>
<td>( R_i )</td>
<td>Installation anchor resistance</td>
<td>Set equal to ( T_i ) (if ( T_i ) is properly verified at installation)</td>
</tr>
<tr>
<td>( R_{L,\alpha} )</td>
<td>Anchor line resistance</td>
<td>Resistance of embedded anchor line for uplift angle ( \alpha )</td>
</tr>
<tr>
<td>( R_{L,\alpha=0} )</td>
<td>Anchor line resistance</td>
<td>Resistance of embedded anchor line for uplift angle ( \alpha=0 )</td>
</tr>
<tr>
<td>( R_{\text{ult}} )</td>
<td>Ultimate anchor resistance</td>
<td>The anchor drags without further increase in the resistance during continuous pulling, which also defines the ultimate penetration depth ( z_{\text{ult}} )</td>
</tr>
<tr>
<td>( R_{\text{ult}, CC1}, R_{\text{ult}, CC2} )</td>
<td>Design ultimate anchor resistance</td>
<td>Calculated ultimate anchor resistance including material factor for CC1 or CC2</td>
</tr>
<tr>
<td>( R_{ai} )</td>
<td>Sum of soil resistance at anchor components</td>
<td>Excluding soil resistance at the fluke</td>
</tr>
<tr>
<td>( R_{FN} )</td>
<td>Soil normal resistance</td>
<td>At the fluke</td>
</tr>
<tr>
<td>( R_{FS} )</td>
<td>Soil sliding resistance</td>
<td>At the fluke</td>
</tr>
<tr>
<td>( R_{ma} )</td>
<td>Moment contribution</td>
<td>From ( R_{ai} )</td>
</tr>
<tr>
<td>( R_{NT} )</td>
<td>Moment contribution</td>
<td>From ( R_{FS} )</td>
</tr>
<tr>
<td>( R_{TIP} )</td>
<td>Moment contribution</td>
<td>From ( R_{TIP} )</td>
</tr>
<tr>
<td>( R_{TIP} )</td>
<td>Tip resistance</td>
<td>At anchor members</td>
</tr>
<tr>
<td>( S_t )</td>
<td>Soil sensitivity</td>
<td>The ratio between ( s_u ) and ( s_{u,r} ), as determined e.g. by fall cone or UU triaxial tests</td>
</tr>
<tr>
<td>( s_u )</td>
<td>Intact strength</td>
<td>For fluke anchor analysis, DSS strength or the UU triaxial strength is assumed to be most representative.</td>
</tr>
<tr>
<td>( s_{u,\text{di}} )</td>
<td>Disturbed undrained shear strength</td>
<td>Undrained shear strength of partly (not fully) remoulded clay (&gt; ( s_{u,r} ))</td>
</tr>
<tr>
<td>( s_{u,r} )</td>
<td>Remoulded shear strength</td>
<td>The strength of a fully remoulded clay measured e.g. in a fall cone or a UU triaxial test (see also ( s_{u,\text{di}} )).</td>
</tr>
<tr>
<td>( s_{u,\text{th}} )</td>
<td>Thixotrophy strength</td>
<td>Undrained shear strength after a given period of thixotropy effect</td>
</tr>
</tbody>
</table>
### Symbol | Term | Explanation of term
--- | --- | ---
\( f_{cy} \) | Cyclic shear strength | Accounts for both loading rate and cyclic degradation effects on \( s_u \).
\( t_{cons} \) | Consolidation time | Time elapsed from anchor installation to time of loading.
\( t_{cy} \) | Time to failure | Rise time of line tension from mean to peak level during the design storm (\( =1/4 \) load cycle period).
\( t_{hold} \) | Installation tension holding period | Period of holding \( T_{min} \) at the end of anchor installation.
\( t_{su} \) | Time to failure | Time to failure in a laboratory test for determination of the intact undrained shear strength (typically 0.5 – 2 hours).
\( T \) | Line tension | Line tension model following suggestion in /4/.
\( T_v, T_h \) | Components of line tension at the shackle | Vertical and horizontal component of the line tension at the anchor shackle for the actual anchor and forerunner.
\( T_C \) | Characteristic line tension | Split into a mean and dynamic component.
\( T_{C-mean} \) | Characteristic mean line tension | Due to pretension and the effect of mean environmental loads in the environmental state.
\( T_{C-dyn} \) | Characteristic dynamic line tension | The increase in tension due to oscillatory low-frequency and wave-frequency effects.
\( T_d \) | Design line tension | With specified partial safety factors included (\( T_{d, intact, CC1} \) or \( T_{d, intact, CC2} \) for the ULS case and \( T_{d, damaged, CC1} \) or \( T_{d, damaged, CC2} \) for the ALS case).
\( T_i \) | Target installation tension | Installation tension at the dip-down point.
\( T_{min} \) | Minimum installation tension | Installation tension if \( L_{s,i} > 0 \) (for \( L_{s,i} = 0 \) \( T_{min} = T_i \)).
\( \Delta T_{min} \) | Drop in tension | Double amplitude tension oscillation around \( T_{min} \) during period \( t_{hold} \).
\( T_{pre} \) | Pretension in mooring line | As specified for the mooring system.
\( U_{cons} \) | Soil consolidation factor | \( U_{cons} = (1 + \Delta R_{cons}/R_i) \), where ratio \( \Delta R_{cons}/R_i \) expresses the effect of consolidation on \( R_i \).
\( U_{cy} \) | Cyclic loading factor | \( U_{cy} = (1 + \Delta R_{cy}/R_{cons}) \), where ratio \( \Delta R_{cy}/R_{cons} \) expresses the effect of loading rate and cyclic degradation on \( R_{cons} \).
\( U_r \) | Loading rate factor | \( U_r = (v_i/v_2)^a \).
\( v_1 \) | Loading rate | Loading rate at extreme line tension.
\( v_2 \) | Loading rate | Loading rate at the end of installation.
\( W_{a'} \) | Submerged anchor weight | Taken as 0.87 \cdot anchor weight in air.
\( W_m \) | Moment contribution | From anchor weight \( W \).
\( W_{l'} \) | Submerged weight of anchor line | Per unit length of actual line segment.
\( z \) | Anchor penetration depth | Depth below seafloor of the fluke tip.
\( z_i \) | Installation penetration depth | For \( R = R_i \).
\( z_{ult} \) | Ultimate penetration depth | For \( R = R_{ult} \).
2. Fluke Anchor Components

The main components of a fluke anchor (Figure 2-1) are:

— the shank
— the fluke
— the shackle
— the forerunner

The *fluke angle* is the angle arbitrarily defined by the fluke plane and a line passing through the rear of the fluke and the anchor shackle. It is important to have a clear definition (although arbitrary) of how the fluke angle is being measured.

Normally the fluke angle is fixed within the range 30° to 50°, the lower angle used for sand and hard/stiff clay, the higher for soft normally consolidated clays. Intermediate angles may be more appropriate for certain soil conditions (layered soils, e.g. stiff clay above softer clay). The advantage of using the larger angle in soft normally consolidated clay is that the anchor penetrates deeper, where the soil strength and the normal component on the fluke is higher, giving an increased resistance. However, when a larger angle is used in stiffer soils, the anchor could experience difficulties in penetrating the seabed.

The *forerunner* is the line segment attached to the anchor shackle, which will embed together with the anchor during installation. The anchor penetration path and the ultimate depth/resistance of the anchor are significantly affected by the type (wire or chain) and size of the forerunner, see Figure 3-1.

The *inverse catenary* of the anchor line is the curvature of the embedded part of the anchor line, see Figure 3-1.

3. General fluke anchor behaviour

The resistance of an anchor depends on its ability to penetrate and reach the target installation tension ($T_i$), together with its ability to penetrate further and gain a necessary additional resistance within a tolerable additional drag length during a potential overloading situation.

The penetration path and ultimate penetration depth is a function of

— the soil conditions (soil layering, variation in intact and remoulded undrained shear strength, geotechnical properties of the soil in general)
— the type and size of anchor,
— the anchor’s fluke angle,
— the type and size of the anchor forerunner (wire or chain)
— the line uplift angle $\alpha$ at the seabed level.
— Installation procedure and execution (installation speed, start penetration of the anchor, end position of the anchor, ratio dynamic vs. static load during installation…)

In clay without significant layering a fluke anchor normally penetrates along a path, where the ratio between incremental penetration and drag decreases with depth, see Figure 3-1. At the ultimate penetration depth $z_{\text{ult}}$ the anchor is not penetrating any further. The anchor is “dragging” with a horizontal (or near horizontal) fluke and no general increase in anchor resistance can be seen. At the ultimate penetration depth the anchor reaches its ultimate resistance $R_{\text{ult}}$.

Since reaching the ultimate penetration depth is associated with drag lengths in the range 5 to 10 times the penetration depth, it is impractical to design an anchor under the assumption that it has to be installed to its
ultimate penetration depth. A more rational approach is to assume that only a fraction of the ultimate anchor resistance is utilized in the anchor design, as illustrated by the intermediate penetration depth in Figure 3-1. This will also lead to more predictable drag, and should drag occur, the anchor may have reserve resistance, which can be mobilized through further penetration.

The cutting resistance of a chain forerunner will be greater than the resistance of a steel wire, with the result that a chain forerunner will have a steeper curvature (inverse catenary) at the anchor shackle than a wire forerunner, i.e. the angle $\theta$ at the shackle is larger. This increases the upward vertical component $T_v$ of the line tension $T$ at the shackle with the consequence that a fluke anchor with a chain forerunner penetrates less than one with a wire forerunner, and mobilizes less resistance for a given drag distance.

It has been demonstrated in the JIP on deep-water anchors /1/ that a non-zero uplift angle $\alpha$ at the seabed see Figure 3-1, can be acceptable under certain conditions as discussed in Appendix F. If the uplift angle becomes excessive during installation the ultimate penetration depth may be reduced. The anchor resistance $R(z)$ is defined as the mobilized resistance against the anchor plus the resistance along the embedded part of the anchor forerunner. However, for anchoring systems with a high uplift angle at the seabed the contribution from the anchor line to the anchor resistance will be greatly reduced, see Eq. (F-1).

![Figure 3-1](image)

**Figure 3-1**
Illustration of fluke anchor behaviour, and definition of $R_{ult}$. 

4. Methodology for fluke anchor design

4.1 General

Traditionally, the methods used for design of fluke anchors have been highly empirical, using power formulae in which the ultimate anchor resistance is related to the anchor weight, but, in cohesive soils, analytical methods are now gradually replacing these crude methods. The need for calibrating the methods used for fluke anchor design against good anchor test data is, however, as great as ever.

The data bases for fluke anchor tests is quite extensive, but there are gaps in many data sets, in the sense of missing pieces of information, which makes the back-fitting analysis and calibration less reliable than it could have been. In most cases there are uncertainties attached to the reported installation data, e.g. soil stratigraphy, soil strengths, anchor installation tension, contribution from friction resistance along the anchor line segment on the seabed, depth of anchor penetration, possible effect of anchor roll during penetration, etc.

Also, the difficulty (and cost) to mobilise the most powerful installation vessels for anchor tests explains why only the smallest anchors in the test data base have been pulled to their ultimate resistance ($R_{ult}$) during installation. The bigger anchors were most often only pulled to a certain percentage of their rated ultimate holding capacity which leads to an uncertainty in the ultimate holding capacity derived from design charts. Extrapolation from small to medium size anchor tests to prototype size anchors should be made with due consideration of possible scale effects.

It is of a general interest that future fluke anchor testing, and monitoring of commercial anchor installations, be carefully planned and executed, such that the test database gradually improves, see guidance in Appendix C.

New types of anchor shall be tested under controlled conditions at locations where high quality site specific soil data are available, see guidance in Appendix G.

In the following the limitations of design charts and the requirements to analytical methods are discussed. It is recommended herein that the use of analytical methods, utilising recognised theoretical models and geotechnical principles replace the design practice based on design charts.
4.2 Design charts
The design curves published by the American Petroleum Institute in /6/ and in ISO 19901-7 /12/, which are based on work by the Naval Civil Engineering Laboratory (NCEL), give the ultimate anchor resistance $R_{ult}$ of the respective anchors versus anchor weight. These relationships, which plot as straight lines in a log-log diagram, suffer from the limitations in the database and the inaccuracies involved in simple extrapolation of the $R_{ult}$ measured in small size anchor tests to larger anchors. The diagrams assume an exponential development in the resistance for each type of anchor and generic type of soil based on the so-called Power Law Method.

The anchor resistance resulting from these diagrams is for ultimate penetration of the anchor and represents a safety factor of 1.0. The anchor types included in the test data base for these design charts are no longer used by the offshore industry. Assumed design curves for some of the modern fluke anchors have, however, been added to the charts.

In addition to the API/ISO design curves, the anchor manufacturers maintain their own test databases and publish design charts based on these databases.

4.2.1 Limitations
As mentioned above, anchors are seldom or never installed to their ultimate depth, which means that the anchor resistance derived from these diagrams must be corrected for depth of penetration, or degree of mobilization. After such correction the resulting anchor resistance may be comparable with the installation anchor resistance $R_i$ defined in this recommendation, although with the important difference that it represents only a predicted resistance until it has been verified by measurements during anchor installation. Further anchor behaviour beyond the installation tension may be predicted by means of the graph, but caution should be exercised for the same reasons as stated above and in Section 4.1. When available, analytical methods should be used to confirm the anchor behaviour predicted from the design charts. If no other methods than the design charts are available to predict the anchor behaviour and there are doubts about the validity of the basis of the design charts relative to the actual conditions at the location, the installation tension should be set high enough to ensure that the required safety factors can be satisfied.

Since the soils are divided into stiffness classes from very soft to very hard, an anchor penetrating into a soil where the shear strength increases linearly with depth, or where the soil is layered, may ‘jump’ from one stiffness class to another in terms of resistance, penetration depth and drag.

Most of the anchor tests in the database, being the basis for the design charts, are with a chain forerunner. The effect of using a wire forerunner therefore needs to be estimated separately.

There are many other limitations in the design methods relying on the Power Law Method, which justifies using a design procedure based on geotechnical principles.

As shown in Sections 5.2.1 and 5.2.2, setup and cyclic loading effects, and possible friction resistance along the length of anchor line on the seabed, may be added to $R_i$.

Further description and discussion of the design charts can be found in the forthcoming ISO 19901-7 /12/.

4.3 Analytical tools
4.3.1 General
The analytical tool should be based on geotechnical principles, be calibrated against high quality anchor tests, and validated.

With an analytical tool the designer should be able to calculate:

— the relationship between line tension, anchor penetration depth and drag for the actual anchor and line configuration in the prevailing soil conditions
— how this relationship is affected by changing the type and/or size of the anchor, the type and/or size of the forerunner, or the soil conditions
— the effect on anchor resistance of soil consolidation from the time of anchor installation until the occurrence of the design event, see guidance in Appendix D
— the effects on the anchor resistance of cyclic loading, i.e. the combined effect of loading rate and cyclic degradation, see guidance in Appendix E
— the effect on the penetration trajectory and design anchor resistance of changing the uplift angle at the seabed, see guidance in Appendix F.

4.3.2 Equilibrium equations for fluke anchor analysis
The analytical tool must satisfy the equilibrium equations both for the embedded anchor line and for the fluke anchor.

The inverse catenary of the embedded anchor line is resolved iteratively such that equilibrium is obtained between the applied line tension and the resistance from the surrounding soil, see /7/. For the fluke anchor both force and moment equilibrium is sought. The equilibrium equations for the anchor line and the anchor as included in an analytical tool developed by DNV are given in Appendix A.
4.3.3 Limitations

In order to be able to use analytical tools, reliable geotechnical input data must be available to the designer, this is discussed in Section 6. As a minimum, in cohesive soils, the input to such models will be the intact and remoulded shear strength profile at the anchor location, together with information about the unit weight of the soil.

There is, as per today, no well documented analytical methods to predict anchor behaviour in other soils than soft to firm cohesive soils. Therefore, in non-cohesive soils, one will have to rely on results from anchor tests/design charts for designing the anchor. Unless the test data on which the design is based is site specific and well documented, this will introduce significant uncertainties in the anchor behaviour. In such case the installation tension should be set high enough to insure that the safety factors can be satisfied, see Section 4.2.1.

Large scatter can be observed between the results of different calculation methodologies /16/. Therefore, analytical tools need to be calibrated against high quality anchor test data in order to validate the results produced by such tools.

4.4 Anchoring risk assessment

Due to the large extension of a mooring spread, there might be significant variations in soil properties from one anchor location to another. Insufficient, or lacking, soil data will also introduce uncertainties in the design soil profiles to be selected (see Section 6 and Appendix G).

For difficult soil types (such as calcareous soils, dense sands or stiff clays...) there are, as per today, no recognised design methods to compute the anchor capacity, thus introducing uncertainties in the actual design resistance of the anchors.

These factors will emphasize the need to pay extra attention to the anchor installation tension if the consequences of anchor drag may be serious, see also Section 5. An increase of the installation tension may be used to reduce the uncertainties in the actual design resistance of the anchors.

Other factors such as type of moorings, seabed congestion, bathymetry, etc. might increase the risk associated with anchoring at a particular location.

For these reasons, whenever a location is selected for anchoring of a floating structure, it is recommended to carry out an assessment of the potential anchoring problems that could occur. An approach for evaluation of anchoring problems at a particular location is presented in /14/. This method is based on rating the risk linked to different factors and summing all the risk factors in order to obtain an overall risk level for a particular site.

The outcome of such study can then be used to determine the need for additional soil data in order to minimize uncertainties in the anchor design or the need to prepare alternative solutions in case problems are encountered during anchor installation.

5. Recommended design procedure

5.1 General

The philosophy for design of fluke anchor in soft clay should take into account the specifics of fluke anchors that can be considered as partially “self-installing”. This is an advantage in the sense that, in favourable soils, the requirements for installation can be relaxed and one may rely on further anchor drag during an extreme event in order to gain additional resistance. However, this introduces an uncertainty that needs to be addressed by carefully assessing the achievable anchor resistance / penetration for a given installation tension in the site specific soil condition, and the additional drag related to the design event.

The fluke anchor resistance is directly related to the ability of the anchor to penetrate and to the installation line tension applied, which means that requirements to anchor installation are closely linked to the anchor design assumptions. The installation aspects will therefore have to be considered already at the anchor design stage.

In the geotechnical design of fluke anchors the following issues need to be addressed:

— minimum anchor installation tension and installation procedures
— anchor resistance, penetration and drag vs. installation line tension
— acceptable uplift angle during installation and design extreme line tension
— post-installation effects due to consolidation and cyclic loading
— additional drag due to overloading under ULS and ALS situations and allowable additional drag.

If the penetration path is predictable, like in soft to stiff clay, it will be possible to predict the consequences of possible overloading of an anchor during extreme environmental events. Anchors in this type of soil are likely to continue penetrating if they start to drag.

In other soils like hard clay, dense sand, layered soil, cemented carbonate sand, etc. neither the penetration path nor the penetrability may be predictable. If the soil conditions are such that the penetration path is not possible to predict, it will not be advisable to design and install the anchors under the assumption that they will continue to penetrate should they be overloaded during a severe storm.
This difference in anchor behaviour, depending on the actual soil conditions, must be taken into consideration when the strategy for design and installation of fluke anchors is laid, since it becomes a factor in the assessment of the safety of the as installed anchors.

In order to ensure the same reliability of the as-installed fluke anchors irrespective of the site specific soil conditions, it is recommended herein that the anchor installation tension ($T_i$) is set correspondingly higher if there is a potential risk of uncontrolled drag without further increase in anchor resistance in a possible overloading situation during operation.

Sound engineering judgement should always be exercised in the assessment of the characteristic resistance of a chosen anchor, giving due consideration to the reliability of the analytical tool or empirical data and the uncertainty in the design parameters provided for the site.

The recommended procedure for design of fluke anchors is outlined step-by-step in Section 5.5. The procedure is based on the limit state method of design, and tentative safety requirements are given in Section 5.3. Anchor installation requirements are presented in Section 5.4, and guidance for installation and testing of fluke anchors is given in Appendix C.

Guidance for calculation of the effects of setup and cyclic loading and for assessment of a safe uplift angle at the seabed is given in Appendix D, Appendix E and Appendix F, respectively. Requirements to soil investigations are given in Section 6 and Appendix G.

In an actual design situation the designer would benefit from having an adequate analytical tool at hand for parametric studies, see Section 4.3 for requirements to such analytical tools.

### 5.2 Alternative design procedures

Below, two alternative design methods for fluke anchors are presented. The choice of one method will depend on whether or not anchor drag can be accepted for a given mooring system, and on the type of soil and reliability of the anchor behaviour prediction methods for a location (see Section 5.2.3).

#### 5.2.1 Anchors designed for no additional drag

The basic nomenclature used in the anchor design procedure proposed herein is shown in Figure 2-1.

The beneficial effect of soil set up and cyclic loading on the anchor resistance may be utilized in the design of the fluke anchors, such that the target installation tension ($T_i$) can be reduced by a factor corresponding to the calculated increase in the anchor resistance due to these two effects. For anchor fluke tip penetrations less than one to two fluke width, small or no post installation effect should be accounted for.

— **Setup effects**

These are partly overlapping effects of clay thixotropic and clay consolidation after anchor installation. The thixotropy effect tends to dominate during the first days or weeks until the consolidation effect takes over. The two effects are not suitable for addition. See Appendix D for details.

In other soils than clay, no or very limited set up effects can be expected.

— **Cyclic loading effects**

These are the sum of the beneficial loading rate effect and the unfavourable strength degradation effect caused by repeated cyclic loading during storm loading. The latter effect is most pronounced when the clay is subjected to two-way cyclic loading, i.e. when shear stress reversals occur at a potential failure plane, and increases with increasing over consolidation ratio (OCR) of the clay. In a mooring system the anchors are always subjected to one-way cyclic loading (no shear stress reversals), which leads to much less strength degradation of the clay. The net result of cyclic loading in soft to medium stiff clay on the anchor resistance is therefore mostly beneficial. See Appendix E for details.

In hard/dense soils, as discussed in Section 5.2.3 in relation to Class 1 and 2 soils and in Appendix B, a conservative approach will be to disregard completely the effect of setup. The resistance in the direction of the line tension (break-out) may in these cases be governing for the anchor resistance, and needs to be checked, especially if the overlying soft layer is very weak.

The break-out resistance may also be of concern in the assessment of a safe uplift angle at the seabed, when small anchor penetrations are achieved in layered soils or stiff soils, see more about allowable uplift angle at the seabed in Appendix F.

The characteristic anchor resistance $R_C$ is the sum of the installation anchor resistance $R_i$ and the predicted post-installation effects of consolidation and cyclic loading, $\Delta R_{\text{cons}}$ and $\Delta R_{\text{cy}}$, see Figure 5-1. To this resistance in the dip-down point is added the possible seabed friction $\Delta R_{\text{fric}}$ as shown in Figure 5-1 b). Eq. (1) below shows the expression for $R_C$ when $L_s > 0$.

$$ R_C = R_i + \Delta R_{\text{setup}} + \Delta R_{\text{cy}} + \Delta R_{\text{fric}} \quad (1) $$

See guidance for assessment of the consolidation effect $\Delta R_{\text{cons}}$ in Appendix D, the cyclic loading effect $\Delta R_{\text{cy}}$ in Appendix E and the seabed friction contribution $\Delta R_{\text{fric}}$ in Appendix A.
Figure 5-1 a) illustrates the anchor installation phase, with the length of line on the seabed equal to $L_{s,i}$. The installation anchor resistance $R_i$ at the dip down point is equal to the target installation line tension $T_i$ assuming that $T_i$ is adequately measured and documented. The required characteristic anchor resistance is then obtained by adding the predicted contributions $\Delta R_{\text{cons}}, \Delta R_{\text{cy}}$ and $\Delta R_{\text{fric}}$ to $R_i$ as demonstrated in Eq. (1).

Figure 5-1 c) and d) illustrate a situation when the anchor is installed under an uplift angle $\alpha_i$ (angle corresponding to final anchor penetration) and an uplift angle $\alpha$ (not necessarily equal to $\alpha_i$) has been predicted also for the characteristic line tension. In this case Eq. (1) simplifies to

$$R_c = R_i + \Delta R_{\text{setup}} + \Delta R_{\text{cy}}$$

(2)

### 5.2.2 Anchor design relying on additional drag

In certain cases the anchors cannot be installed to a sufficient installation tension to avoid additional drag or it is not practical or economical to do so. In this case, if the soil properties allow for relying on further anchor penetration during a design event, the additional drag needed to mobilize a sufficient resistance must be assessed. In certain cases the calculation may show that the required anchor drag cannot be tolerated for various reasons (e.g. violation of the integrity of adjacent installations, operational criteria, breaking strength of neighboring lines). This may be resolved by increasing the size of the anchor or the installation tension.

Once the anchor starts dragging, all effects of the consolidation will be lost. However due to the travel speed of the anchor in the soil, one may allow for some increase of the resistance due to loading rate effect. In such a case the characteristic resistance of the anchor can be expressed as shown in Eq. (3) below.

$$R_c = R_i + \Delta R_{\text{drag}} + \Delta R_{\text{cy}} + \Delta R_{\text{fric}}$$

(3)

where $\Delta R_{\text{drag}}$ is the resistance gained by further penetration of the anchor and the anchor line during its loading beyond the installation tension (See Figure 5-2). $\Delta R_{\text{drag}}$ depends on the additional drag/penetration beyond the installation position.

At the ultimate penetration depth $z_{\text{ult}}$, the resistance will reach its maximum $R_{\text{ult}}$ and any increase in the line tension will lead to continuous drag.

The minimum anchor installation tension $T_{\text{min}}$ is the installation tension in the touch-down point required to be able to resist the maximum design line tension without dragging more than the allowable additional drag.

### 5.2.3 Definition of soil classes

The specification of the target anchor installation resistance must take into account the site specific soil conditions and the consequences of potential anchor drag during operation. In the following the significance of the site specific soil conditions is addressed, and for this purpose the following three soil classes have been defined, see also Appendix B:

<table>
<thead>
<tr>
<th>Soil classes</th>
<th>Soil properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>1)</td>
<td>Hard or dense soils e.g. hard clays, dense sands, cemented soils, rock…</td>
</tr>
<tr>
<td>2)</td>
<td>Layered soil e.g. soft clay over hard clay, sand inter-beds in clay soil, cemented layers in sands…</td>
</tr>
<tr>
<td>3)</td>
<td>Soft to medium stiff clays, possibly layered</td>
</tr>
</tbody>
</table>

For Class 1 soils, the anchor penetration at the end of installation may be limited to a few meters and parts of the anchor may remain visible above the seabed. For such soils there are no recognized methods available for prediction of the ultimate penetration and resistance of a fluke anchor. In these conditions, the anchors should be installed to the design anchor resistance. This is to verify already during installation that the anchors have reached their target resistance and this is a precaution against uncontrolled anchor drag, and potential anchor failure. It is always recommended to compile results from reliable and relevant anchor installations in similar soil although the anchor behavior may vary significantly from anchor point to anchor point in Class 1 soil. On this basis it is recommended that the increase in resistance ($\Delta R_{\text{drag}}$) associated with potential additional anchor drag is set to zero for Class 1 soils. The target anchor installation tension ($T_i$), which is the installation tension at the dip-down point, should be assessed based on the methodology described in Section 5.2.1.

For Class 2 soils, the anchor may penetrate for example through a soft clay layer before meeting an underlying harder layer. In this case, depending on the thickness and geotechnical properties of the soft clay layer, the anchor orientation when reaching the harder layer will vary significantly. The selection of the fluke angle should therefore be based on the assessment of the anchor’s ability to reach the hard layer and its ability, for the given fluke angle, to penetrate into the hard layer. The best suited fluke angle can therefore vary from the “sand” angle to the “mud” angle. If the anchor penetration at the end of installation is sufficient and it is possible to reliably predict the anchor behavior when tensioned beyond its installation tension, a design relying on additional drag may be allowed (see Section 5.2.2). Else, as for Class 1 soils, the design has to be based on the methodology described in Section 5.2.1.

For Class 3 soils, if installed to a sufficient depth, the anchor can gain extra resistance due to set up effects of the soil around the anchor (see Appendix D). In addition, the cyclic effects will contribute to the anchor
resistance (Appendix E). Due to these contributions the governing safety criteria can be satisfied with a reduced installation tension compared to Class 1 and 2 soils even if the anchors are designed for no drag (see Section 5.2.1). In addition, if the mooring design allows for additional drag, one may rely on further anchor penetration and drag to achieve the necessary resistance (see Section 5.2.2). In that case all effects of the set up will be lost and the minimum installation tension $T_{\text{min}}$ will be governed by the maximum allowable drag.

a) At installation:
   (no uplift)

   ![Diagram](image1)

   

b) At operation:
   (no uplift)

   ![Diagram](image2)

   

c) At installation:
   (uplift angle $\alpha_i$)

   ![Diagram](image3)

   

d) At operation:
   (uplift angle $\alpha$)

   ![Diagram](image4)

   

Figure 5-1
Basic nomenclature, anchor designed for no additional drag.
5.3 Tentative safety requirements

According to this recommendation the geotechnical design of fluke anchors shall be based on the limit state method of design. For intact systems, the design shall satisfy the Ultimate Limit State (ULS) requirements, whereas one-line failure shall be treated as an Accidental Damage Limit State (ALS) condition. These two conditions will in the following sometimes be referred to as “intact” and “damaged” conditions, respectively. The line tension model adopted herein splits the tension into a mean and a dynamic component; see background in /4/ and /5/. The calibration of the partial safety factors adopted herein is based on the results from a pilot reliability study only /9/. Until the design rule has been calibrated based on a more detailed reliability analysis the partial safety factors for the anchor design proposed herein will therefore be tentative.

The primary function of an anchor, in an offshore mooring system, is to hold the lower end of a mooring line in place under all environmental conditions. Since extreme environmental conditions give rise to the highest mooring line tensions, the designer must focus attention on these conditions. If the extreme line tension causes the anchor to drag, then the anchor has failed to fulfill its intended function. However, limited drag of an anchor need not lead to the complete failure of a mooring system. In fact, it may be a favourable event, leading to a redistribution of line tensions, and reducing the tension in the most heavily loaded line. But this is not always the case. If the soil conditions exhibit significant differences between anchor locations, then a less heavily loaded anchor may drag first, and lead to an increase in the tension in the most heavily loaded line. If designing the anchors and selecting an installation tension such that the anchors will have to drag to resist the extreme line tension, the installation tension should be chosen carefully such that it will limit the additional drag to acceptable values for all lines; i.e. the possible additional drag in any lines should not lead to a breach of the safety factors in any of the other mooring lines.
The purpose of the calculations or testing on which the design is to be based, is to maintain the probability of reaching a limit state below a specified value. In the context of designing a mooring system, the primary objective with the ULS design is to ensure that the mooring system stays intact, i.e. to guard against occurrence of a line failure or occurrence of continuous anchor drag.

For calibration and quantification of the partial safety factors for ULS and ALS design, probabilistic analyses will be necessary. Such studies have been carried out by DNV through the Deepmoor Project with respect to both catenary and taut (synthetic fibre rope) mooring systems /8/. A pilot reliability analysis of fluke anchors, using the extreme line tension distributions from /8/ as a realistic load input, has been performed for one test case as part of the JIP on deep water anchors /9/.

Based on this pilot reliability analysis partial safety factors have been proposed for design of fluke anchors in clay. These safety factors, which are considered to be conservative, may be revised when a formal calibration of the design rule proposed herein has been performed.

5.3.1 General

The safety requirements are based on the limit state method of design, where the anchor is defined as a load bearing structure. For geotechnical design of the anchors this method requires that the following two limit state categories be satisfied by the design:

— the Ultimate Limit State (ULS) for intact system, and
— the Accidental Damage Limit State (ALS) following one-line failure or thruster failure

The design line tension $T_d$ at the touch-down point is the sum of the two calculated characteristic line tension components $T_{C-mean}$ and $T_{C-dyn}$ at that point multiplied by their respective partial safety factors $\gamma_{\text{mean}}$, $\gamma_{\text{dyn}}$, i.e.

$$T_d = T_{C-mean} \cdot \gamma_{\text{mean}} + T_{C-dyn} \cdot \gamma_{\text{dyn}}$$

(4)

where

$T_{C-mean} =$ the characteristic mean line tension due to pretension $(T_{pret})$ and the effect of mean environmental loads in the environmental state

$T_{C-dyn} =$ the characteristic dynamic line tension equal to the increase in tension due to oscillatory low-frequency and wave-frequency effects

The characteristic tension components may be computed as suggested in /5/.

If the anchor is designed for no additional drag (See Section 5.2.1), the design anchor resistance ($R_d$) is defined as

$$R_d = R_i + \left( \Delta R_{\text{setup}} + \Delta R_{\text{cy}} + \Delta R_{\text{fric}} \right) / \gamma_m$$

(5)

If the anchor is designed to rely on additional drag (See Section 5.2.2), the design anchor resistance ($R_d$) is defined as

$$R_d = R_i + \left( \Delta R_{\text{drag}} + \Delta R_{\text{cy}} + \Delta R_{\text{fric}} \right) / \gamma_m$$

(6)

$R_i$ is known with the same confidence as $T_i$, and the partial safety factor is set equal to 1.0 under the assumption that the installation tension is measured with sufficient accuracy, e.g. by the DNV Tentune method /10/ or with an anchor tracker (see Appendix C). If it cannot be documented that the installation tension $T_{min}$ has been achieved the partial safety factor on that contribution will have to be set higher than 1.0.

The partial safety factor $\gamma_m$ (on the predicted characteristic anchor resistance) shall account for the uncertainty in the intact undrained shear strength, as far as it affects the calculation of the mentioned contributions to $R_C$. This factor is intended for use in combination with characteristic anchor resistance calculated by geotechnical analysis as described in Section 4.3. If the anchor resistance is based on a simplified analysis, e.g. using design charts as discussed in Section 4.2, then the expression for the design resistance $R_d$ in Eq. (5) and the value of the partial safety factor $\gamma_m$ in Table 5.1 has to be reconsidered on a case to case basis.

5.3.2 Consequence classes

Two consequence classes are considered for the ULS and ALS, defined as follows:

1) Failure is unlikely to lead to unacceptable consequences such as loss of life, collision with an adjacent platform, uncontrolled outflow of oil or gas, capsize or sinking,

2) Failure may well lead to unacceptable consequences of these types.

The target reliability level for consequence class 1 should be set to avoid mooring system failure, but without a high level of conservatism, since the consequences are not unacceptable. The target reliability level for consequence class 2 should be higher in view of the consequences.
5.3.3 Partial Safety Factors for the ULS - intact system

For the ULS case, tentative partial safety factors are suggested in Table 5-1.

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>Type of analysis</th>
<th>$\gamma_{\text{mean}}$</th>
<th>$\gamma_{\text{dyn}}$</th>
<th>$\gamma_{\text{m}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dynamic</td>
<td>1.10</td>
<td>1.50</td>
<td>1.30</td>
</tr>
<tr>
<td>2</td>
<td>Dynamic</td>
<td>1.40</td>
<td>2.10</td>
<td>1.30</td>
</tr>
<tr>
<td>1</td>
<td>Quasi-static</td>
<td>1.70</td>
<td></td>
<td>1.30</td>
</tr>
<tr>
<td>2</td>
<td>Quasi-static</td>
<td>2.50</td>
<td></td>
<td>1.30</td>
</tr>
</tbody>
</table>

5.3.4 Partial Safety Factor for the ALS - one-line failure

The purpose of the accidental damage limit state (ALS) is to ensure that the anchors in the mooring system provide an adequate amount of resistance to avoid subsequent mooring system failure, if one mooring line should initially fail for reasons outside of the designer's control. Such an initial mooring line failure may also be considered to include the possibility of anchor drag for that line (in case additional drag is not acceptable).

Detailed analysis of the ALS has not been carried out yet, but some reduction of the resistance factor $\gamma_{\text{m}}$ applied to the ULS seems appropriate for consequence class 1. The partial safety factors given in Table 5-2 are tentatively suggested when the characteristic anchor resistance is defined as for the ULS.

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>Type of analysis</th>
<th>$\gamma_{\text{mean}}$</th>
<th>$\gamma_{\text{dyn}}$</th>
<th>$\gamma_{\text{m}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dynamic</td>
<td>1.00</td>
<td>1.10</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>Dynamic</td>
<td>1.00</td>
<td>1.25</td>
<td>1.3</td>
</tr>
<tr>
<td>1</td>
<td>Quasi-static</td>
<td>1.10</td>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>Quasi-static</td>
<td>1.35</td>
<td></td>
<td>1.3</td>
</tr>
</tbody>
</table>

5.4 Minimum installation tension

The prescribed installation tension at dip down point $T_i$, see Figure 5-1 and Figure 5-2, will to a great extent determine the geotechnical safety of the anchor as installed.

The partial safety factors used as basis in this recommended practice were determined in a pilot reliability analysis aiming for a target annual probability of failure of $10^{-4}$ for consequence class 1 and $10^{-5}$ for consequence class 2. If aiming for these probabilities of failure, $T_i$ may be assessed from Eq. (7) below for anchor design for no additional drag and Eq. (8) for anchor design relying on additional drag. Otherwise, for the minimum requirements to installation tension, please refer to DNV-OS-E301 /5/.

$$ T_i = T_d - (\Delta R_{\text{fric}} + \Delta R_{\gamma}) \gamma_{\text{m}} $$  \(7\)  

$$ T_i = T_d - (\Delta R_{\text{drag}} + \Delta R_{\gamma}) \gamma_{\text{m}} $$  \(8\)

If the anchor is designed to rely on additional drag to achieve a sufficient resistance, one should in addition select a high enough $T_i$ such that the additional drag will remain within the allowable drag calculated in the mooring analysis i.e. make sure that the additional drag needed to achieve $\Delta R_{\text{drag}}$ does not lead to the breach of the safety factor requirements of any of the mooring lines.

The line length on the seabed during installation $L_{\text{s,i}}$ may be different from the length $L_s$ assumed in the anchor design calculations and used for calculation of $\Delta R_{\text{fric}}$ in Eq. (7) and Eq. (8).

At the stage of anchor installation the prescribed minimum installation load $T_{\text{min}}$ in the touch-down point is intended to ensure that the target installation load $T_i$ in the dip-down point is reached, accounting for the installation seabed friction over the length $L_{\text{s,i}}$. Therefore, the predicted seabed friction is multiplied by a partial safety factor $\gamma_{\text{m,i}}$. Tentatively this factor is set equal to $\gamma_{\text{m}}$ for the predicted anchor resistance, i.e. $\gamma_{\text{m,i}} = 1.3$.

The required minimum installation tension at touch down point becomes:

$$ T_{\text{min}} = T_i + \mu \cdot W_i \cdot L_{\text{s,i}} \cdot \gamma_{\text{m,i}} $$  \(9\)

If the anchor can be installed with an uplift angle or if the anchor can be installed with no line on seabed, the length of line on the seabed will be set to zero (i.e. $L_s = L_{\text{s,i}} = 0$), and $T_{\text{min}}$ can be taken equal to $T_i$.

In practice, $T_i$ will have to be calculated through an iterative process following the step-by-step procedure outlined in Section 5.5. The resulting $T_{\text{min}}$ will then be evaluated and compared with the installation tension that can be achieved with the installation scenarios under considerations, see Appendix C.

Eq. (7) and Eq. (8) assume implicitly that the installation line tension is measured with such accuracy that the partial safety factor on $T_i$ and thus on $R_i$ can be set equal to 1.0. It is therefore imperative for achieving the intended safety level that adequate means for measuring the installation line tension versus time is available on
board the installation vessel. For such purpose the utilization of an anchor tracker system (See Appendix C) will be of significant help. Other means such as subsea beacons installed on the mooring line can also be used.

5.5 Step-by-step description of procedure

The following main steps should be followed in the design of fluke anchors in clay, see flowchart in Figure 5-3.

Step-by-step procedure:

1) Select mooring pattern, line configuration, anchor model and size, installation tension and consequence class (CC1 or CC2).

2) Determine the design line tension \( T_d \) in the touch-down point, see Section 5.3 for consequence classes. Determine (from the mooring analysis) the maximum allowable drag under the intact conditions.

3) Compute the penetration path down to the ultimate depth \( z_{ult} \) for this anchor, see Section 4 and Figure 3-1, 5-1 and 5-2 for guidance. For the installation tension \( T_i \) considered, calculate the design resistance \( R_{d,CC2} \) for CC2 conditions.
   - If \( R_{d,CC2} \) is higher than \( T_{d,intact,CC2} \) and \( T_{d,damaged,CC2} \), there will be no drag and the design can be accepted.
   - Else, operations must be stopped when tensions become higher than \( R_{d,CC2} \). If CC1 is applicable go to step 4, else go to step 6.

4) Compute \( R_{d,CC1} \) and \( R_{ult,CC1} \) (see Section 5.2.2 and Figure 5-2).
   - If \( R_{d,CC1} \) is higher than \( T_{d,intact,CC1} \) and \( R_{ult,CC1} \) is higher than \( T_{d,damaged,CC1} \) there will be no drag and the design can be accepted.
   - Else, go to step 5.

5) Compute the additional drag to resist \( T_{d,intact,CC1} \)
   - If the additional drag required to resist \( T_{d,intact,CC1} \) is within the maximum allowable drag and \( R_{ult,CC1} \) is higher than \( T_{d,damaged,CC1} \), the design can be accepted.
   - Else, return to Step 1 and select another mooring pattern and/or anchor and/or installation tension.

6) Compute the additional drag to resist \( T_{d,intact,CC2} \) and \( R_{ult,CC2} \)
   - If the additional drag required to resist \( T_{d,intact,CC2} \) is within the maximum allowable drag and \( R_{ult,CC2} \) is higher than \( T_{d,damaged,CC2} \), the design can be accepted.
   - Else, return to Step 1 and select another mooring pattern and/or anchor and/or installation tension.

7) Estimate the anchor drop point based on the computed drag length for penetration depth \( z = z_i \), see Figure 5-1.

Note 1:
In case of significant layering or stiff soil reference is made to guidance in Appendix B.

Note 2:
The acceptable uplift angle during design loading will be decided from case to case, see guidance in Appendix F.

Note 3:
The uplift angle and the position of the touch-down point under design load should be computed by mooring line analysis for the design tension, not for the characteristic tension. In this calculation, the reduction of the effective line length from dip down point to fairlead due to the embedded part of the line should be accounted for.

Note 4:
Analytical tools used for prediction of anchor performance during installation and operational conditions should be well documented and validated, see guidance in Section 4.3 and Appendix A.
Figure 5-3
Design procedure - flowchart.
6. Requirements for Soil Investigation

The planning and execution of soil investigations for design of fluke anchors should follow established and recognized offshore industry practice. As a general guidance to achieve this quality of soil investigation reference is made to the NORSOK standard /11/, which makes extensive references to international standards.

Some specific recommendations are given herein for soil investigations for fluke anchors.

For design of fluke anchors the soil investigation should provide information about:

- Seafloor topography and sea bottom features
- Soil stratification and soil classification parameters
- Soil parameters of importance for all significant layers within the depth of interest.

The most important soil parameters for design of fluke anchors in clay are the intact un-drained shear strength ($s_u$), the remoulded un-drained shear strength ($s_{u,r}$), the clay sensitivity ($S_t$), the soil unit weight ($\gamma$), the coefficient of consolidation ($c_v$), and the cyclic shear strength ($\tau_{f,cy}$) for each layer of significance.

As a minimum, the soil investigation should provide the basis for specification of a representative soil profile and the un-drained shear strengths ($s_u$ and $s_{u,r}$) for each significant soil layer between the seabed and the maximum possible depth of anchor penetration. The number of soil borings/in situ tests required to map the soil conditions within the mooring area will be decided from case to case. However, normally one in-situ test for each anchor location is desirable in order to capture the variations in soil conditions.

The ultimate depth of penetration of fluke anchors in clay varies with the size of the anchor and the un-drained shear strength of the clay. It is convenient to account for the size of the anchor by expressing the penetration depth in terms of fluke lengths. In very soft clay the ultimate penetration may be up to 8 to 10 fluke lengths decreasing to only 1 to 2 fluke lengths in strong, over consolidated clays. However, an anchor is never (or seldom) designed for full utilisation of the ultimate anchor resistance $R_{ult}$, because of the associated large drag distance.

In general, the necessary depth for soil investigation will be the expected anchor penetration depth, plus some reserve depth in order to cover for any uncertainty in the design (change in anchor size, model…). The necessary investigation depth will therefore have to be decided on a case by case basis. In soft soils, and depending on the size and model of the anchor, the investigation depth might range from 10 to 15 m to more than 30 m while in stiff soil the investigation depth might be lower than 10 m.

The upper few metres of the soil profile are of particular interest for the critical initial penetration of the anchor, and for assessment of the penetration resistance and the inverse catenary of the embedded part of the anchor line.

General requirements to the soil investigation for fluke anchor foundations, in addition to the recommendations in /11/ are provided in Appendix G.

Due to the large spread of the anchors in a mooring pattern and the relatively large uncertainty on the exact positions of the anchors it is important to have a good description of the lateral variations in soil layering, bathymetry and seabed features. The use of a geophysical site survey will therefore be an important input to the design of fluke anchors. Such surveys should provide data on stratigraphy such that it can be correlated to CPT and sampling results and seabed features such as plough marks, boulders or possible erosion channels.

A site survey usually consists of high resolution seismic data, multi beam echo-sounding, side scan sonar and allow producing maps of e.g. bathymetry or soil layer thickness.

It is referred to the Guidelines for Conduct of Mobile Drilling Rig Site Surveys /15/ and the Guidelines for the Conduct of Offshore Drilling Hazard Site Survey /G-4/ for more guidance on such surveys.
7. References


/12/ ISO 19901-7 (2005), Station keeping systems for floating offshore structures and mobile offshore units, 1st Ed., dated 2005-12-01 (2nd edition expected to be issued in 2012).


APPENDIX A
ANALYSIS TOOL FOR FLUKE ANCHOR DESIGN

A.1 General

An analytical tool for fluke anchor design should be able to calculate anchor line catenary in soil as well as the fluke anchor equilibrium itself. In addition, it should be able to predict the catenary part of the line such that full interaction between the mooring line and the anchor can be analysed. Further, the analytical tool should be able to assess the effect of consolidation as being an important design issue in soft to stiff clay. The following section describes in brief the principles for such an analytical tool developed by DNV /A-1/.

Since there are neither well-established theories nor numerical tools to predict anchor behaviour in other soils than soft clay as per today, the theory presented below is only valid for soft to stiff cohesive soils. For other soils, other methods such as compiling results from anchor tests in comparable soils or using higher test tension will have to be used. This is discussed in Section 5.2 and Appendix B. More research and testing will be necessary in soils other than soft cohesive soils in order to be able to develop reliable prediction tools. It is worth noting that some research on anchor behaviour in sand has been published in /A-4/ and /A-5/ and may serve as basis for further development while /A-6/ presents a study for fluke anchors in carbonate soils.

The equations presented for the line equilibrium are however for general purpose and can be extended to non-cohesive soils provided that the bearing capacity and friction factors are adapted.

A.2 A2 Anchor line seabed friction

The resistance due to seabed friction $\Delta R_{\text{fric}}$ in Eq. (1) is expressed as follows:

$$\Delta R_{\text{fric}} = f \cdot L_s = \mu \cdot W_l' \cdot L_s \quad (A-1)$$

where

- $f$ = unit friction (also of cohesive nature)
- $L_s$ = line length on seabed for the characteristic line tension $T_C$
- $\mu$ = coefficient of seabed friction
- $W_l'$ = submerged weight of the anchor line per unit length

**Guidance note:**

Based on the back-fitting analysis of data from measurements on chain segments reported in /A-2/ and estimated values for wire, the following coefficients of seabed friction are recommended for clay

<table>
<thead>
<tr>
<th>Coefficient of seabed friction</th>
<th>Lower bound</th>
<th>Default value</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wire</td>
<td>0.1</td>
<td>0.2</td>
<td>0.3</td>
</tr>
<tr>
<td>Chain</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
</tr>
</tbody>
</table>

1) The unit friction $f$ along the embedded part of the anchor line as required for calculation of anchor line contribution to the anchor resistance $R_i$ is given by Eq. (A-5).

For non-cohesive soils, reference is made to /A-8/ and /A-9/ for guidance on the $\mu$ values.

---e-n-d---of---G-u-i-d-a-n-c-e-n-o-t-e---
A.3 Equilibrium equations of embedded anchor line

The equilibrium of the embedded part of the anchor line can be solved approximately by closed form equations or exactly in any soil strength profiles by iterations /7/. The normal stress $q$ and the unit soil friction $f$, which act on an anchor line element in the soil are shown schematically in Figure A-1.

![Figure A-1](image)

Soil stresses at an anchor line segment in soil

The loss in line tension $dT$ over one element length $ds$ is calculated from the following formula (Simplified equation only valid for small $ds$ and $θ$):

$$
\frac{dT}{ds} = -f \cdot AS - W' \cdot \sin(θ) \tag{A-2}
$$

where

$T$ = anchor line tension

$θ$ = orientation of anchor line element ($θ = 0$ for a horizontal element)

$AS$ = effective surface of anchor line per unit length of line

$ds$ = element length

The angular advance from one anchor line element to the next is then solved by iterations from the following formula (Simplified equation only valid for small $ds$ and $θ$):

$$
\frac{dθ}{ds} = \frac{q \cdot AB - W' \cdot \cos(θ)}{T} \tag{A-3}
$$

where

$q$ = normal stress

$AB$ = effective bearing area of anchor line per unit length of line

**Guidance note:**

The following default values are suggested for the effective surface area $AS$ and the effective bearing area $AB$:

<table>
<thead>
<tr>
<th>Type of forerunner</th>
<th>$AS$</th>
<th>$AB$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chain</td>
<td>$11.3 \cdot d$</td>
<td>$2.5 \cdot d$</td>
</tr>
<tr>
<td>Wire or rope</td>
<td>$\pi \cdot d$</td>
<td>$d$</td>
</tr>
</tbody>
</table>

where

$d$ = nominal diameter of the chain and actual diameter of the wire or rope.

---end---of---G-u-i-d-a-n-c-e---n-o-t-e---

The normal stress $q$ on the anchor line in cohesive soils is calculated from the following equation:

$$
q = N_c \cdot s_u \tag{A-4}
$$

where

$N_c$ = bearing capacity factor

$s_u$ = undrained shear strength (direct simple shear strength $s_{ud}$ is recommended)
Effect of embedment and inclination on the bearing capacity factor should be included.
For non-cohesive soils, it is referred to DNV Class Notes CN30.4 /A-7/ for general guidance on bearing capacity formulas.

Guidance note:
Based on the back-fitting analysis reported in /A-2/, the following bearing capacity factors are recommended for the embedded part of the anchor line in clay

<table>
<thead>
<tr>
<th>Wire / Chain</th>
<th>Lower bound</th>
<th>Default value</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nc</td>
<td>9</td>
<td>11.5</td>
<td>14</td>
</tr>
</tbody>
</table>

1) See Guidance Note above for values of the effective bearing area \( AB \), which is a pre-requisite for use of the bearing capacity factors given here.

Effect of shape, orientation and embedment of the various resistance members on the anchor should be included as relevant.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---

The unit friction \( f \) along the anchor line in cohesive soils can be calculated from the following formula:

\[
 f = \alpha_{soil} \cdot s
\]  

(A-5)

where

\( \alpha_{soil} \) = adhesion factor for anchor line

For non-cohesive soils, it is referred to DNV Class Notes CN30.4 /A-7/ for general guidance on bearing capacity formulas.

Guidance note:
Based on the back-fitting analysis of data from measurements on chain segments reported in /A-2/, and estimated values for wire, the following coefficients of seabed friction are recommended for the embedded part of the anchor line clay

<table>
<thead>
<tr>
<th>Wire / Chain</th>
<th>Lower bound</th>
<th>Default value</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_{soil} )</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>Chain</td>
<td>Lower bound</td>
<td>Default value</td>
<td>Upper bound</td>
</tr>
<tr>
<td>( \alpha_{soil} )</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
</tr>
</tbody>
</table>

1) See Guidance Note above for values of the effective surface area \( AS \), which is a pre-requisite for use of the adhesion factor given here.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---

A.4 Equilibrium equations for fluke anchor

Moment equilibrium and force equilibrium can be solved for the fluke anchor for two different failure modes. One mode leading to further anchor penetration in a direction close to the fluke penetration direction, and a second mode leading to reduced or no further penetration. In principle, the soil resistance contributions are the same for the two failure modes, but in the first failure mode the soil resistance normal to the fluke may not take on the ultimate value. Using the symbols shown in Figure A-2 the necessary equilibrium equations are defined and explained in the following.

Figure A-2
Principal soil reaction forces on a fluke (anchor penetration direction coincides with fluke penetration direction).
For the range of possible penetration directions, the horizontal and vertical equilibrium should satisfy the following equations:

Horizontal equilibrium:

\[ T \cdot \cos(\theta) = \sum_{i=1}^{N} R_{ai} \cdot \cos(\beta) + R_{FS} \cdot \cos(\beta) + R_{TIP} \cdot \cos(\beta) + R_{FN} \cdot \sin(\beta) \]  \hspace{1cm} (A-6)

Vertical equilibrium

\[ T \cdot \sin(\theta) = R_{FN} \cdot \cos(\beta) + W_a' - \left( \sum_{i=1}^{N} R_{ai} \cdot \sin(\beta) + R_{FS} \cdot \sin(\beta) + R_{TIP} \cdot \sin(\beta) \right) \] \hspace{1cm} (A-7)

where

- \( T, \theta \) = tension and corresponding orientation of anchor line at the shackle
- \( R_{FN} \) = soil normal resistance at the fluke
- \( R_{FS} \) = soil sliding resistance at the fluke
- \( R_{TIP} \) = tip resistance at the fluke
- \( R_{ai} \) = soil resistance at the remaining components of the anchor (separated through anchor geometry)
- \( W_a' \) = submerged anchor weight
- \( \beta \) = penetration direction of fluke

The normal resistance will be the normal stress times the bearing area of the anchor part being considered, and may need to be decomposed in the three orthogonal directions defined (one vertical and two horizontal). The normal stress can be calculated from the following formula:

\[ q = N_c \cdot s_u \] \hspace{1cm} (A-8)

where

- \( N_c \) = bearing capacity factor

Sliding resistance will be the unit friction times the adhesion area of the anchor part being considered. The unit friction \( f \) along the anchor part can be calculated from the following formula:

\[ f = \alpha \cdot s_u \] \hspace{1cm} (A-9)

where

- \( \alpha \) = adhesion factor for anchor

The bearing and adhesion areas should in this case be modelled with due consideration of the actual geometry of the anchor.

**Guidance note:**
Based on the back-fitting analysis reported in /A-1/, /A-2/ and /A-3/ the following values are tentatively recommended for the resistance towards the various anchor members in clay:

**Table A-5 Bearing capacity and adhesion factor**

<table>
<thead>
<tr>
<th>Bearing capacity factor ( N_c ) for: ( R_{FN} )</th>
<th>Adhesion factor ( \alpha ) for: ( R_{ai} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( R_{FN} ) = 12.5 (^1) \hspace{1cm} ( R_{ai} ) = 12.5 \hspace{1cm} ( R_{TIP} ) = 12.5 \hspace{1cm} ( R_{FS} ) = 1 / ( S_t )</td>
<td></td>
</tr>
</tbody>
</table>

1) Effect of shape, orientation and embedment of the various resistance members on the anchor should be included as relevant.
2) Actual degree of mobilisation of this value as required satisfying moment equilibrium.

Due consideration should be given to the difference in adhesion for continuous penetration and inception of anchor drag (failure event). For the latter, an adhesion factor compatible with time available for consolidation should be assessed, see Appendix D.

Horizontal and vertical equilibrium for a certain fluke penetration direction can be achieved for a number of
fluke orientations and line tensions at the shackle. In order to determine the correct penetration direction and the corresponding line tension, moment equilibrium must be satisfied (here taken with respect to the shackle point):

\[ \sum_{i=1}^{N} R_{m_{ai}} + R_{m_{FS}} + R_{m_{TIP}} -(W_m + R_{EN} \cdot e) = 0 \]  \hspace{1cm} (A-10)

where

- \( R_{m_{FS}} \) = moment contribution from soil sliding resistance at the fluke
- \( R_{m_{TIP}} \) = moment contribution from tip resistance at the fluke
- \( W_m \) = moment contribution from anchor weight
- \( R_{EN} \) = soil normal resistance at the fluke
- \( e \) = lever arm between shackle and the line of action of the normal resistance at the fluke
- \( R_{m_{ai}} \) = moment contribution from soil resistance at the remaining components of the anchor (separated through anchor geometry)

When the anchor penetrates in the same direction as the fluke, any possible lever arm (\( e \)) and normal resistance that can be replaced by a realistic stress distribution at the fluke should be considered. When the anchor penetrates in another direction than the fluke, the centre of normal resistance on the fluke should act in the centre of the fluke area.

### A.5 References


APPENDIX B
ANCHORS IN LAYERED CLAY OR STIFFER SOIL

B.1 General
As stated in Section 5.2.3 the specification of the target anchor installation resistance must take into account the site specific soil conditions and the consequences of potential anchor drag during operation. Three soil classes are defined and used as basis for discussing the need for adapting the strategy for design and installation of fluke anchors to account for the site-specific soil conditions. The following three soil classes are defined in Section 5.2.3:

<table>
<thead>
<tr>
<th>Class</th>
<th>Soil properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>1)</td>
<td>Hard or dense soils e.g. hard clays, dense sands, cemented soils, rock…</td>
</tr>
<tr>
<td>2)</td>
<td>Layered soil e.g. soft clay over hard clay, sand inter-beds in clay soil, cemented layers in sands…</td>
</tr>
<tr>
<td>3)</td>
<td>Soft to medium stiff clays, possibly layered</td>
</tr>
</tbody>
</table>

The procedures for design and installation of fluke anchors described in this recommendation have been developed for Class 3 soil, since their behaviour both during the installation and the design life can be predicted with reasonable confidence. The safety requirements in Section 5.3 state that the anchors in an offshore mooring system shall be able to hold the lower end of a mooring line within an acceptable (if any) additional drag under all environmental conditions. If the extreme line tension causes the anchor to drag continuously or drag more than an acceptable distance, then the anchor has failed to fulfil its intended function.

In order to fulfil these requirements in Class 1 (and, in some cases, in Class 2 soils), the reliability of the anchor points will often need to be ensured by specification of an adequate anchor installation tension. This is achieved by setting target anchor installation tension $T_i$, high enough to avoid any anchor drag during the lifetime of the platform. It is not recommended to let the capacity of the installation equipment decide the upper limit of the anchor installation tension.

The same conservative strategy needs to be pursued also when anchors are designed for installation in carbonate sands, where the soil conditions may vary significantly from one anchor point to another within the mooring pattern, both in the lateral and vertical direction. Carbonate sands may have a high sensitivity, a high degree of compressibility and a variable degree of cementation, which needs to be taken into account when the target anchor installation tension is decided. Due to these soil characteristics the measured anchor installation resistance may be reduced due to cyclic loading effects during extreme environmental events, which affects particularly the normal resistance against the anchor. This normal resistance is generated mainly by the strength of the soil volume in front of the anchor, above the depth of the fluke tip. If the strength of this volume of soil is reduced during a storm the anchor force and moment equilibrium may not be possible to maintain during the extreme events. In the worst case the anchor may then lose its grip at the fluke tip level and start to drag through the loosened top layer (see /A-6/).

In Section 3 and Figure 3-1 the focus is on a fluke anchor in normally consolidated clay without layering. A discussion of the possible penetrability and long-term resistance of fluke anchors in layered soils is given in the following. It should however be kept in mind that depending on the soil properties, the relative strength of each layers, the anchor model, size and fluke angle the anchor behaviour can vary significantly.

In a stiff-soft layer sequence, experience has shown that a fluke anchor may penetrate through a surface layer of sand or relatively stiffer clay into an underlying softer clay layer, provided that the thickness of this surface layer is less than 30 to 50% of the fluke length of the actual anchor. Although this cannot be taken for granted, it can be used as guidance when various anchor alternatives are being evaluated. The prevailing soil conditions and possible past experience with fluke anchor installation in the actual area should always be evaluated before the choice of anchor is made.

In a soft-stiff layer sequence, the ability of an anchor to pick up the resistance of the underlying stiffer layer depends on the difference in soil strength between the two layers, the depth to the stiffer layer and the angle of the fluke plane when it meets the stiffer layer. If this ‘attack’ angle is too small the anchor may drag on top of the stiff layer at constant load. If this angle is too large at a shallow penetration, the anchor may rotate and break out of the soil rather than continue along the initial penetration path. In both these cases the target installation load will not be reached. Changing the fluke angle or choosing another type and/or size of anchor may improve the situation.

In a stiff-soft-stiff layer sequence, involves the extra complication that penetration through the upper stiff layer may require a smaller fluke angle than desirable for the anchor to penetrate through the locked-in soft layer and then continue to penetrate securely also into the second stiff layer. Again, the anchor should meet the deeper stiff layer at an angle, which ensures a grip and penetration into that layer. Furthermore, the line angle $\theta$ increases due to the stiff overlying soil which makes the anchor rotating backwards and thus reducing the anchor ultimate penetration. If the thickness of the two first layers is such that the anchor approaches the deeper layer at an angle, which is too small, the anchor will just drag along the surface of that layer. This may be
visualised by the fact that the drag becomes excessive, or non-tolerable, and the target anchor installation tension is never reached. In most cases, predictions may show that the penetration path improves in that respect, and becomes steeper for a given depth and a given fluke angle, if the anchor is increased in size. It may also be possible to find more optimal, non-standard, combinations between anchor size and fluke angle, which account both for the overlying and the underlying stiff layer.

From the above it is evident that layer thickness, depth to boundaries between layers, and soil strength need to be documented for proper design of a fluke anchor foundation and to avoid unexpected behaviour of the anchor during the installation phase, see Section 6 and Appendix G for requirements to soil investigation.
APPENDIX C
INSTALLATION AND TESTING OF FLUKE ANCHORS

C.1 General
Fluke anchor design is by tradition empirical as illustrated by the design charts published by e.g. the American Petroleum Institute /6/. The anchor tests being the basis for such design charts are of variable quality, and typically there are gaps in the test data, which makes it difficult to fully understand and rely on the test results. All reasonable efforts should therefore be made to ensure that the measurements are reliable and include the most crucial test data for maximum usefulness of the results and improvement of the database. This should be fully appreciated when installing both test anchors and prototype anchors.

C.2 Minimum installation tension
The anchor installation should follow procedures, which have been presented and agreed to by all parties well ahead of the installation. By prescribing a minimum installation tension $T_{\text{min}}$, see Section 5, the intention is to ensure that the design assumptions are fulfilled during anchor installation. In other words, if the anchor is installed to $T_{\text{min}}$ the design anchor resistance $R_d$ has implicitly also been verified, or the additional drag needed to reach $R_d$ is kept within acceptable limits.

Part of the $T_{\text{min}}$ is made of the necessary $T_i$ to be achieved at dip down point while the rest depends on line friction on seabed. For assessment of the necessary $T_{\text{min}}$, the line scope should therefore be known.

The target tension should be reached stepwise, insuring a slow enough penetration speed of the anchor, thus reducing the anchor penetration resistance. The target tension level should be then held for a specified holding period, which period may be soil dependent. Any relaxation (drag) during this period should be compensated for, such that the required line tension is maintained as constant as possible. The anchor installation and testing log should document the events and the measurements taken from start to end of the installation.

C.3 Monitoring of fluke anchor installations

C.3.1 General
When installation of prototype, or test anchors, is being planned it is imperative that the most essential boundary conditions for the installation are taken into consideration. Well ahead of the installation such background information should be compiled and documented.

It is recommended to check the position and orientation of the anchor, as well as the alignment, straightness and length on the seabed of the as-laid anchor line, before start of tensioning. This will often require ROV assistance during anchor installation. Significant misalignment of the installation anchor line will require extra line tension to reach the specified target anchor installation tension $T_i$, which should be estimated and accounted for.

During the anchor installation a number of parameters need to be measured to serve as a documentation of the installation. The more information that is recorded beyond the minimum documentation requirements, the more useful the installation data will become in the end.

Monitoring of the anchor installation should, as a minimum, provide data on

- line tension
- line (pitch) angle at the stern roller
- anchor drag
- anchor penetration (if possible)

These parameters should be measured as a function of time from start to end of the installation. A calibrated transducer, being a segment of the installation line or mounted on the anchor shackle, should preferably be used to measure the line tension.

If manual measurements are taken intermittently, see checklist below, they should also be time-stamped and stored together with the other installation data.

The final installation measurements should at least document that the minimum anchor installation tension $T_{\text{min}}$ has been achieved and maintained during the specified holding time.

The checklist below indicates the type of information that should be focussed on before and during the installation and testing of fluke anchors. This checklist can be used as a guidance both for installation of both prototype and test anchors.

C.3.2 Checklist

1) Before the installation.
   
a) Assessment of the most likely soil stratigraphy at the anchor location and the soil strength of significant layers (from soil investigation report), see Section 6 for guidance.
b) Specification of the anchor and the installation line configuration.

c) Specification of the fluke angle(s) to be used, and how this angle is defined, see Section 2 and Figure 2-1 for guidance.

d) Estimate of friction resistance at the stern roller.

e) Equipment and procedures for anchor installation, e.g. type and tensioning system of the vessel, method of laying and tensioning the anchors, availability of ROV, etc.

f) Type of measurements to be undertaken, and procedures to be applied, from check list below.

2) During the installation.

a) Line tension (horizontal component measured at deck level)\(^1\)

b) Drag (method of measurement, reference point)

c) Penetration depth (method of measurement, at least the final depth)

d) Line angle with the horizontal outside the stern roller (at least for the final line tension)

e) Pull-in speed (vessel speed, drag and line angle at stern roller versus time)

3) Final installation measurements

a) Maintaining \(T_{\text{min}}\) (during specified holding time \(t_{\text{hold}} = 15\) to 30 minutes)

b) Measure tension vs time during holding time (mean tension \(\geq T_{\text{min}}\))

c) Drag (corresponding to final penetration depth)

d) Penetration depth (best estimate of final depth, for example with help of an anchor tracking device)

e) Unexpected anchor behavior in relation to available soil info should be reported.

\(^1\) It is recommended to measure the installation tension by an anchor tracker, see Section C5, or by means of the DNV Tentune method /10/.

The database for fluke anchors loaded to their ultimate resistance \(R_{\text{ult}}\) is unfortunately limited to rather small anchors. The largest anchors tested in connection with offshore projects have normally not reached the \(R_{\text{ult}}\), but for the future it would be fruitful for the industry if the most significant parameters (line tension, drag and final penetration depth) are recorded during all installations, at least in a few locations out of many.

In this connection it is important that all reasonable efforts are made to make the recorded data as reliable as possible, since the assessment of the safety of the anchoring system depends on such installation data.

C.4 Anchor installation vessels

The bollard pull of the most powerful new generation anchor handling vessels is in the range 2.5 to 3.5 MN (250mT to 350mT). Depending on the required minimum installation tension \(T_{\text{min}}\) at the touch-down point, one or two \(\text{AHV}\)'s may be required. As an alternative to using \(\text{AHV}\)'s the anchor tensioning can be done from a special tensioning vessel/barge or from the floater itself. If two opposite anchors are tensioned simultaneously line tensions up to 5 MN to 10 MN (500mT to 1000mT) can be reached.

The chosen scenario for anchor installation shall ensure that the specified minimum installation tension \(T_{\text{min}}\) can be reached. The bollard pull, winch capacity and minimum breaking load (MBL) of the installation wire on the actual vessel(s) will have to be assessed on this basis. If \(T_{\text{min}}\) cannot be reached due to pulling limitations set by the vessel(s), the design anchor resistance \(R_d\) will not be achieved.

It is essential that all parties involved in the decisions related to the anchor design appreciate the relationship between anchor resistance and installation tension. In deep waters, unless lightweight anchor lines are used, the weight and sea bed friction of the anchor lines limit the net line tension that can be used for anchor penetration, which must be considered when the requirements for the installation vessel are specified.

C.5 Anchor trackers

The current anchor test data base includes results from both onshore and offshore tests with both fluke anchors and drag-in plate anchors. In most cases the quality of offshore test data used in the calibration of the anchor models in the DIGIN data base is not sufficient for a reliable back-fitting analysis. Among the short-comings are the uncertainty attached to the load measurements. The ongoing development of anchor tracker systems for on-line measurement of anchor behaviour, including load, for offshore application will improve the quality of the offshore anchor tests significantly.

An anchor tracker was used already in 1997 to monitor the installation of drag-in plate anchors offshore Brazil on behalf of Petrobras. In the following years this tracker system continued to be used both in connection with offshore and onshore anchor tests. In the onshore anchor tests in 1998 at Onsøy in Norway this tracker was used to monitor both the Denla and the Stevmanta drag-in plate anchors in real-time, for details see /C-1/.

The main anchor manufacturers are developing their own anchor tracking systems for commercial use with the objective to have a real time measurement of the anchor load, drag, orientation and penetration depth.
However, for a meaningful interpretation of the results from anchor tracker measurements at an offshore or onshore location the site-specific soil conditions should be well documented, see requirements to soil investigations in Appendix G.

C.6 References

APPENDIX D
SETUP EFFECT ON ANCHOR FRICTION RESISTANCE

D.1 General

The setup and shear strength increase in the soil at the wall of a fluke anchor occur due to dissipation of excess pore pressure, increased horizontal normal effective stress and thixotropy. During continuous penetration of a fluke anchor in clay, the friction resistance between the fluke anchor walls and the surrounding soil will be governed by the remoulded shear strength in a narrow zone close to the anchor. Although it is common to assume that the clay will be fully remoulded during installation of fluke anchors it may be argued that the clay will not be fully remoulded during this process. It is therefore recommended to account for this possibility by introducing a parameter called Disturbance Ratio, \( DR \), as an alternative to the sensitivity, \( S_t \), see Section D2.

After an anchor has been installed to a certain installation tension (and depth), the clay will gradually reconsolidate from the partly, or fully, disturbed state to its fully reconsolidated shear strength. A corresponding gradual increase in the anchor friction resistance can be expected, which is referred to as setup. The setup effect due to consolidation is discussed in Section D3.

Setup may also be caused by thixotropy effects, which can lead to strength increase along the walls of a fluke anchor of more than 100% after remoulding, even before setup due to pore pressure dissipation (consolidation) occurs. Setup due to thixotropy effects is discussed in Section D4.

D.2 Disturbance Ratio vs. Sensitivity

In an analytical model the friction resistance of the fluke anchor walls during anchor installation may be accounted for through the adhesion factor, \( \alpha \), see Eq. (A-9). In the pile and suction anchor design it is common industry practice to set this factor equal to the inverse of the sensitivity, \( S_t \), which implies that the \( \alpha \)-factor is set equal to its minimum value \( \alpha_{\text{min}} \), see Eqs. (D-1) and (D-2) below.

\[
S_t = \frac{s_u}{s_{u,r}} \quad \text{(D-1)}
\]
\[
\alpha_{\text{min}} = \frac{1}{S_t} \quad \text{(D-2)}
\]

Since the remoulded shear strength depends on the rate of strain, it is important to take rate effect into account and use a remoulded strength representative for the rate and duration of the design load, just as for intact strength /D-1/. If the remoulded shear strength is determined based on intact strength and sensitivity through \( s_{u,r} = s_u / S_t \), the rate effect should be included in the intact shear strength and the rate independent sensitivity can be used.

It has also been the default assumption in this recommendation that the clay will be fully remoulded during installation of a fluke anchor and to use Eq (D-2) when predicting the installation friction resistance of fluke anchors, in combination with use of the DSS un-drained shear strength, i.e. \( s_u = s_{u,DSS} \), in Eq. (D-1).

This design practice may be conservative, and thus acceptable, for prediction of the installation resistance of piles and suction anchors, but questionable for use in the prediction of the penetration path and installation resistance of fluke anchors. The reason is that the actual penetration path of a fluke anchor is closely linked to the requirement that anchor force and moment equilibrium must be satisfied throughout the penetration of the anchor along this path. If the soil strength parameters are assumed based on conservative, or default, assumptions that do not agree with the actual in-situ soil conditions, neither the predicted anchor penetration path will agree with the actual path followed by the anchor.

If the assumed unit friction on the anchor walls in a particular design case is set equal to the remoulded shear strength of the clay, \( s_{u,r} \), corresponding to 100% remoulding, \( s_{u,r} = s_{u,DSS} / S_t \), and the actual degree of remoulding is only 50%, the predicted friction resistance during installation of the anchor will be underestimated by a factor 2. By using the higher friction value the design prediction may show that the anchor penetrates deeper and will be accepted rather than rejected, since this also leads to less predicted drag for the target anchor installation tension.

To address the scenario outlined above a parameter called Disturbance Ratio, \( DR \), is introduced, which is defined as:

\[
DR = \frac{s_u}{s_{u,di}} \quad \text{(D-3)}
\]

where \( s_{u,di} \) is the disturbed un-drained shear strength (\( > s_{u,r} \)). In the case described above \( s_{u,di} = 2 \cdot s_{u,r} \) and \( DR = 0.5 \cdot S_t \).

The actual disturbance ratio \( DR \) depends probably on many factors in addition to the size and geometry of the anchor, e.g. the clay characteristics (plasticity, liquidity, clay content, etc.), but no systematic research has been published on how the degree of remoulding depends on the disturbance energy applied. Limited data from
published tests of different types where the amount of disturbance energy is controlled indicate, however, that the derived \( DR \) values may be significantly smaller than the reported clay sensitivity, especially when the reported sensitivity is high.

The local unit skin friction at the shaft of a pile has been shown to degrade as a function of the number of meter pile length that passes the actual point, which indicates that the length of the pile affects the degraded average skin friction along the whole pile. Due to the short length of an anchor fluke, the length effect may be much smaller for a fluke anchor than for a pile and the resulting average friction resistance correspondingly higher.

On the basis of this length effect it may be of particular importance, when interpreting the results from field tests with scaled-down fluke anchors (and drag-in plate anchors), to account for the size of small test anchor.

Based on the observations described above it is tentatively concluded that the predicted installation trajectory of a fluke anchor depends on the degree of remoulding assumed. The loading rate effect (anchor penetration speed) will also affect this trajectory, probably in a same way as a decrease of the disturbance ratio \( DR \).

It seems likely that anchor tracking systems will soon be offered for use during installation of anchors, which will measure the actual penetration trajectory of the anchors as well as the installation load applied at the anchor. It is recommended to use this information in the model calibration point of view if the site-specific soil data are of poor quality, which is often the case at MODU locations. Therefore, any “scientific” anchor tests, which are aimed at giving more insight into the anchor behaviour during installation, should be carried out at a suitable and well-documented (preferably onshore) location, where all parameters of interest can be measured.

**D.3 Setup due to consolidation**

From a geotechnical point of view there should be no major difference between fluke anchors and for example piles or the skirts of suction anchors or gravity structures, when the effects of installation and subsequent reconsolidation on the clay un-drained shear strength are considered.

The installation anchor resistance \( R_i \) increases with time \( t \) after installation due to reconsolidation (setup) of the remoulded clay surrounding the anchor. This increase in resistance represents the increase in friction resistance along the anchor walls and is termed \( \Delta R_{\text{setup}} \) herein.

The setup effect on the installation anchor resistance \( R_i \) may also be expressed as a setup factor \( U_{\text{setup}} \), which will increase with time \( t \) after installation. The resulting setup anchor resistance \( R_{\text{setup}} \) after consolidation will then be

\[
R_{\text{setup}} = R_i \cdot U_{\text{setup}} = R_i \cdot \left(1 + \frac{\Delta R_{\text{setup}}}{R_i}\right)
\]

where

\[
U_{\text{setup}} = f(t, S_t \text{ or } DR, \text{ geometry, depth and orientation of the anchor})
\]

The maximum setup, after 100% consolidation, is governed by the clay sensitivity \( S_t \), or the disturbance ratio \( DR \), depending on whether the clay became fully remoulded or only partly remoulded during anchor installation, and the anchor geometry, depth and orientation.

The setup due to consolidation can be assessed by considering the drainage characteristic in the volume of clay adjacent to the anchor, which has been influenced (remoulded) during the anchor installation. The extent of this zone depends on the anchor geometry and the actual soil characteristics. Guidance for modelling and calculation of this setup effect can be obtained using the experience from e.g. tests on piles.

The setup factor \( U_{\text{setup}} \) related to the total (static) anchor installation resistance \( R_i \) will be much smaller than reflected by the value of the sensitivity \( S_t \) or the disturbance ratio of the clay, since the friction resistance only contributes to part of \( R_i \). The relation between setup factor \( U_{\text{setup}} \) and the increase in the friction resistance depend on the geometry of the anchor, and its final depth of penetration into the soil during the installation phase. A reliable quantification of this effect can only be obtained by site-specific relevant full-scale tests or by use of adequate analytical tools. The analytical tools should be able to predict both the penetration part and the subsequent consolidation phase. It is essential that the analytical tool accounts for full force and moment equilibrium, compatible with the failure modes in question, see Appendix A.

Caution is recommended in the assessment of the possible consolidation effect when the likely failure mode, following upon such consolidation, may either reduce or prevent further penetration. Overloading will in this case initiate anchor movement in the direction of the line tension, before the full effect from consolidation is utilised. When such movement has been initiated, the soil adjacent to the anchor walls will lose the setup effect of consolidation, which will increase the anchor drag. This can in particular be expected when the anchor has reached only a shallow depth of penetration, where the resistance in the direction of the line tension is limited. It may also be relevant if the fluke tip has penetrated partly into a stiffer layer underlying a soft layer.

In practice, the setup factor \( U_{\text{setup}} \) must be assessed on a case by case basis.
D.4 Setup due to thixotropy

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 /D-2/.

The thixotropy strength ratio, which is defined as the ratio between the shear strength after a period of time with thixotropic strength gain and the shear strength just after remoulding, is herein termed setup factor $U_{\text{setup}}$ using the same terminology as in Section D3 for setup effects due to consolidation.

Published test data show that thixotropy can result in significant strength increase shortly after installation, which means that it is possible to rely on shear strength along the walls of a fluke anchor, which is higher than the remoulded shear strength, even before pore water pressure dissipation occurs. The measurements show that the thixotropy effect gives a very rapid increase in strength in the first few days, or even hours, and that strength continues to increase even after 2 months.

Plots of the thixotropy strength ratio after 1, 10 and 60 to 90 days for a number of natural clays as function of the plasticity index presented in /D-3/ confirm that the thixotropy has a significant effect on the time-dependent gain in shear strength after remoulding of clays. Testing of different type of clay has also shown that the thixotropy of natural clays is likely to depend strongly on their mineral composition. This was confirmed in /D-4/ when plotting the thixotropy strength ratio against the inverse of the activity of the soil, the activity defined as $\%\text{clay}/I_p$, which plot was extended further in /D-1/ by inclusion of more data points.

However, there is a considerable scatter both in the plasticity and inverse activity plots. Due to this uncertainty it is recommended to select a low value of the thixotropy strength ratio, $\alpha_{\text{th}}$, within the scatter of measured values, when a low strength is conservative for the design, like in capacity analyses. If a high strength is conservative, like in retrieval of fluke anchors for MODUs, a high value should be selected. If the thixotropy strength ratio turns out to be important for the design, thixotropy tests on site specific soil should be considered.

The unit friction $f$ along the anchor part due to the thixotropy effect can never be higher than the full consolidation effect, $(1/\alpha)$, according to Eq. (A-9), and the consolidation and thixotropy effects are not additive.

D.5 References


APPENDIX E
EFFECT OF CYCLIC LOADING

E.1 Background
Cyclic loading affects the static undrained shear strength ($s_u$) in two ways:

— **Effect of loading rate**
  During a storm, the rise time from mean to peak load may be about 3 to 5 seconds (1/4 of a wave frequency
  load cycle), as compared to 0.5 to 2 hours in a static consolidated undrained triaxial test, and this higher
  load rate leads to an increase in the undrained shear strength.

— **Effect of repeated cyclic loading**
  As a result of repeated cyclic loading during a storm, the undrained shear strength will decrease, the
  degradation effect increasing with the over consolidation ratio ($OCR$) of the clay.

The most direct, and preferred, approach to account for both the loading rate effect and the cyclic degradation
effect is to determine the cyclic shear strength $\tau_{f,cy}$ of the clay, following the strain accumulation procedure
described in /E-4/.

The calculation of the cyclic shear strength, $\tau_{f,cy}$, should be based on anisotropic, stress path dependent
undrained shear strengths. The strengths should be monotonic or cyclic, depending on the loading situation. The
shear strengths are normally determined for triaxial compression, triaxial extension and DSS. The shear
strengths for intermediate stress paths can be interpolated from these three based on the inclination of the
potential failure surface (limiting equilibrium) or the direction of the major principal stress (finite element
method). For strain softening clays, strain compatibility should be taken into account when the anisotropic
monotonic shear strengths are established. Numerical analyses with a strain softening monotonic soil model
are recommended when the strain softening is significant.

In cases where the monotonic load acts over a long period of time, it is important to consider the potential
reduction in undrained shear strength due to creep effects. It is also important to evaluate whether drainage
may occur and what the effect of this drainage may have on the anchor resistance.

In this recommendation it is assumed that the loads are cyclic in nature and that the anchor resistance is
 calculated by use of the cyclic shear strength, $\tau_{f,cy}$, which is discussed in Section E2 below.

E.2 Cyclic shear strength
The cyclic shear strength $\tau_{f,cy}$ is dependent on the load history, i.e. the composition of the line tension
amplitudes (number and magnitude) in the storm in question. The characteristic value of the cyclic shear
strength for calculation of the characteristic anchor resistance $R_C$ shall be determined as the cyclic shear
strength associated with the characteristic storm. The characteristic storm is the stationary sea state of specified
duration with a return period of 100 years. In practice, for determination of the characteristic cyclic shear
strength, the characteristic storm shall be taken as that particular sea state along a 100-year environmental
contour in the ($H_S$, $T_P$) space which produces the smallest cyclic shear strength. Here, $H_S$ and $T_P$ denote
significant wave height and peak period, respectively.

**Guidance note:**
The significant wave height $H_S$ is assumed constant during the stationary sea state that constitutes the characteristic
storm. The duration of the characteristic storm may vary depending on the location and depending on the type of
loading implied by the storm.

For winter storms in the North Sea, leading to wave-frequency loading of the anchor, it is common to consider
stationary sea states with a duration of 3 hours. The characteristic storm for North Sea conditions is thus in principle
a 3-hour sea state with a return period of 100 years. However, in the North Sea it is common practice to apply a 42-
hour storm consisting of an 18-hour build-up phase, a 6-hour duration of the 100-year sea state and an 18-hour decay
phase. This 42-hour idealised storm is conservative and produces a somewhat smaller cyclic shear strength than that
produced by the 3-hour characteristic storm alone.

The cyclic shear strength will also depend on the cyclic load period, and the cyclic laboratory tests should be
run with a load period representative of the cyclic line load.

Three types of cyclic shear strengths may have to be considered in the geotechnical design of foundations
against the effects of cyclic loading:

\[
\begin{align*}
\tau_{f,cy,D} & = \text{DSS cyclic shear strength} \\
\tau_{f,cy,C} & = \text{triaxial compression cyclic shear strength} \\
\tau_{f,cy,E} & = \text{triaxial extension cyclic shear strength} 
\end{align*}
\]

The DSS cyclic shear strength is normally used as the reference shear strength.
E.3 Application to fluke anchor design

E.3.1 General

In a mooring system the loads transmitted to the anchors through the anchor lines will always be in tension (one-way), which has a less degrading effect on the shear strength than two-way cyclic loading, which is characterized by shear stress reversals. The failure criterion for one-way cyclic loading is development of excessive accumulated permanent strains. The maximum shear stress the soil can sustain at that state of failure is equal to the cyclic shear strength $\tau_{f, cy}$.

In the design of fluke anchors according to the procedures described herein, the DSS monotonic un-drained shear strength, $s_{u,D}$, is assumed to represent the average of the DSS, triaxial compression and triaxial extension un-drained shear strengths within the volume of clay contributing to the anchor resistance. Consequently, the cyclic DSS shear strength, denoted $\tau_{f,cy}$ herein, may then be taken as representative for the cyclic loading effects on the un-drained shear strengths of this volume of clay.

Since the triaxial cyclic shear strength are not being considered further herein, the cyclic DSS shear strength will simply be denoted $\tau_{f,cy}$ in the following.

E.3.2 Loading rate factor, $U_r$

In order to understand how the loading rate may affect the resistance of fluke anchors a parallel may be drawn between piles and fluke anchors. Important work on the effect of loading rate on axial pile capacity is published in /E-1/, /E-2/, and /E-3/.

The following relationship is suggested in /E-3/ for description of the effect of the loading rate, $v$, on pile capacity, $Q$

$$Q_1/Q_2 = (v_1/v_2)^n$$

(E-1)

where $Q_1$ and $Q_2$ represent the pile capacity at loading rates $v_1$ and $v_2$, respectively.

The loading rate during wave loading is much higher than during anchor installation, and the anchor resistance increases in relation to this increase in loading rate. Using the experience from pile testing as expressed by Eq. (E-1) a loading rate factor $U_r$ may be introduced, which expresses the loading rate effect on the anchor resistance, i.e.

$$U_r = (v_1/v_2)^n$$

(E-2)

One practical problem with Eq. (E-2) is to determine representative values for the loading rates $v_1$ and $v_2$. Another problem is to assess the value of exponent $n$ in the equation for $U_r$ for guidance see /E-3/.

The load rate effect on the static un-drained shear strength can be expressed either as a function of the rate of shear strain or the time to failure. Results from testing of load rate effects on several clays are published in /E-7/ and /E-8/.

E.3.3 Cyclic loading factor, $U_{cy}$

The relationship between the cyclic and the static un-drained shear strength is expressed herein by means of the cyclic loading factor $U_{cy}$, which is defined as

$$U_{cy} = \tau_{f,cy}/s_{u,D}$$

(E-3)

where $\tau_{f,cy}$ is the cyclic shear strength for the specified storm stress amplitude history at a particular average stress $\tau_a$, and $s_{u,D}$ is the monotonic un-drained DSS shear strength.

The cyclic loading factor in purely two-way cyclic loading is denoted $U_{cy0}$. The equivalent number of cycles to failure $N_{cy}$ is defined as the number of cycles of the maximum cyclic shear stress amplitude that will give the same effect as the actual cyclic storm history, see /E-4/.

The cyclic loading factor $U_{cy}$ depends on the normalised average shear stress $\tau_a/s_{u,D}$, where $\tau_a$ denotes the average shear stress. This is illustrated in Figure (E-1).
Anchors are typically subjected to loading for which $\tau_{cy} < \tau_a$, where $\tau_{cy}$ denotes the cyclic shear stress amplitude. The cyclic loading factor can then be determined from a diagram as illustrated in Figure (E-2) for the relevant $N_{eqv}$ and combination of $\tau_a$ and $\tau_{cy}$. In cases when the loading gives a constant ratio between the cyclic and the average shear stresses, the cyclic loading factor can be determined graphically as the intersection between a line with a slope of $(\tau_a+\tau_{cy})/\tau_a$ and the curve for the relevant $N_{eqv}$.

Cyclic resistance calculations should, if possible, be made according to the graphical procedure proposed in /E-4/. This procedure accounts for the redistribution of average soil stresses that occur during cyclic loading and determines whether the failure mode will be large cyclic displacements, large average displacements, or a combination of the two. The procedure is based on the assumption that the combination of average and cyclic shear strains is the same along the potential failure surface (strain compatibility), and on the condition that the average shear stresses along the potential failure surface are in equilibrium with the average loads.

For prediction of the cyclic loading factor $U_{cy}$ by means of diagrams such as those given in Figure (E-1) to Figure (E-3), the $U_{cy}$ value can then be read at the intersection between the respective curves for the relevant $N_{eqv}$ and a line with the following simplified equation:

$$\frac{\tau_a}{s_{u,D}} = \frac{T_{d-mean}}{T_d} \frac{\tau_{fy}}{s_{u,D}}$$  \hspace{1cm} (E-4)

which is an approximation of the conditions valid in the failure situation considered in design.
As an alternative to the graphical approach, the following mathematical expression is proposed for determination of the cyclic loading factor $U_{cy}$:

$$U_{cy} = a_0 + a_1 \left( \frac{\tau_{f,cy}}{\tau_{u,0}} \right) + a_2 \left( \frac{\tau_{f,cy}}{\tau_{u,0}} \right)^2 + a_3 \left( \frac{\tau_{f,cy}}{\tau_{u,0}} \right)^3$$  \hspace{1cm} (E-5)

The coefficients $a_0$, $a_1$, $a_2$, and $a_3$ depend on the equivalent number of cycles to failure $N_{eqv}$, and are determined from well-documented advanced laboratory tests on clay subjected to one-way and two-way cyclic loading. As an example, for normally consolidated Marlin Clay subjected to a 3-hour wave-frequency storm load history representative for Gulf of Mexico conditions, the following expressions for the four coefficients were found to fit the test data, see also Figure (E-1).

$$a_0 = 0.0090(\ln N_{eqv})^3 - 0.1583\ln N_{eqv} + 1.3163$$
$$a_1 = -0.0079(\ln N_{eqv})^3 + 0.5547\ln N_{eqv} + 0.4953$$
$$a_2 = 0.290(\ln N_{eqv})^3 - 2.0959\ln N_{eqv} + 2.0834$$
$$a_3 = 0.2174(\ln N_{eqv})^3 + 1.6789\ln N_{eqv} - 2.9305$$  \hspace{1cm} (E-6)

Another example is given for normally consolidated Drammen Clay subjected to a 3-hour wave-frequency storm load history representative for North Sea conditions. In this example the following expressions for the four coefficients were found to fit the test data for the DSS cyclic shear strength:

$$a_0 = -0.1401 \cdot \ln N_{eqv} + 1.2415$$
$$a_1 = 0.0995 \ln N_{eqv} + 1.0588$$
$$a_2 = -0.5795 \cdot \ln N_{eqv} + 0.3426$$
$$a_3 = 0.6170 \cdot \ln N_{eqv} - 1.6048$$  \hspace{1cm} (E-7)

**E.4 Simplified assessment of $U_r$ and $U_{cy}$**

In the next sections, the basis for a simplified assessment of the load rate effect $U_r$ and cyclic loading effect $U_{cy}$ is provided. The $U_r$ and $U_{cy}$ values provided in Figure E-3 are based on information from /E-4/, /E-5/ and /E-6/.

The cyclic laboratory tests behind Figure E-1 were carried out on normally consolidated clay (OCR = 1 to 1.5), but the effect of OCR on the cyclic behaviour for so-called one-way cyclic loading (no shear stress reversal), which is a relevant assumption when mooring line tension is considered, is moderate. Typically $U_r$ and $U_{cy}$ will be reduced by up to 5% when OCR increases from 1 to 4, by up to 15% when OCR increases from 1 to 7 and by 20% when OCR increases from 1 to 10.

The cyclic response will also be affected by the frequency of loading, e.g. low-frequency versus wave-frequency tension components. The low-frequency component has typically a period, which is about 10 times longer than the wave-frequency component represented in the test results plotted in Figure E-1. Recognising the effect of loading rate an increase in the load rise time $t_{cy}$ from 2.5 seconds to 25 seconds, i.e. one log-cycle change, will give a reduction in the net cyclic loading effect by about 10%, e.g. a reduction from $U_{cy} = 1.3$ to $U_{cy} = 1.27$.

**E.4.1 Load rate factor**

As outlined above the effect of cyclic loading is two-fold, the loading rate effect and the cyclic degradation effect.

In a cyclic laboratory test on clay the cycle period is often set to 10 seconds, which means that the load rise time $t_{cy}$ from mean level to the first peak load is 2.5 seconds (= $t_{cy}$). If the cycle amplitude is high enough to fail the clay specimen during that first quarter of the first load cycle ($N_{eqv} = 1$), the corresponding cyclic strength $\tau_{f,cy}$ of the clay divided by the static un-drained shear strength $\tau_{uD}$ is a measure of the loading rate factor $U_r$ for the actual clay, i.e.

$$U_r = \frac{\tau_{f,cy}}{\tau_{u,0}} \quad \text{for } N_{eqv} = 1$$

Figure E-3 presents excerpts of published results from cyclic direct simple shear tests on the Drammen clay /E-4/, on the Troll clay /E-5/ and on the Marlin clay /E-6/.

Figure E-3a) shows the loading rate factor $U_r$ as a function of the average shear stress level $\tau_{z}/\tau_{uD}$ during the test. It is noted that the loading rate effect is most pronounced for $\tau_{z}/\tau_{uD}$ in the range 0.5 to 0.7.
Based on the mooring analysis it will be possible to define the mean, low-frequency and wave-frequency components of the characteristic line tension, such that a basis is obtained for assessment of a likely range for the parameter $\tau_a/s_{UD}$. Typically the line tension in a catenary mooring system may generate an average shear stress level $\tau_a/s_{UD}$ in the range 0.6 to 0.8. For this range $U_r = 1.4$ to 1.75 for five examples shown in Figure E-3a. Caution is warranted in the use of experience from testing of non-carbonate clay, if the actual clay contains more than 10% carbonate.

**E.4.2 Cyclic loading factor**

Following the strain accumulation procedure as described in detail in /E-4/, and summarised in this Appendix, the cyclic test data may be used for prediction of the cyclic loading factor $U_{cy}$.

In Figure E-3b) and c) the $U_{cy}$-factor is plotted for $N_{eqv} = 3$ and $N_{eqv} = 10$. In the latter case this means that if the calculations lead to failure in cyclic loading for a given cyclic load history the same effect will be achieved if 10 cycles of the extreme load amplitude in the same load history is applied to the clay.

When looking at the range of $U_r$ and $U_{cy}$ reported for the different clays in Figure E-3 it is evident that experience from testing of one clay will not necessarily be representative of the behaviour of another clay in another geological environment. Some characteristics of these clays are shown in Table E-1. Unless a site specific cyclic testing programme has been designed and executed, empirical data like these and elsewhere in the literature should therefore be used with caution.
E.4.3 The cyclic anchor resistance, $R_{cy}$

If a fluke anchor has been subjected to setup for a period of time $t$ after installation, the reference anchor resistance for assessment of the cyclic loading effects will be the anchor setup resistance $R_{setup}$ in Eq. (D-3). This leads to the following expression for the cyclic anchor resistance $R_{cy}$:

$$R_{cy} = R_{setup} \cdot U_{cy} = R_{setup} + \Delta R_{setup} + \Delta R_{cy} $$  \hspace{1cm} (E-9)

The expression for $U_{cy}$ then becomes:

$$U_{cy} = \left(1 + \frac{\Delta R_{cy}}{R_{setup}} \right) $$  \hspace{1cm} (E-10)

E.5 References

APPENDIX F
UPLIFT ANGLE AT THE SEABED

F.1 General
The anchor line in a mooring system may be split into three parts, one part embedded in the soil, a second part resting on the seabed, and a third part suspended in water.

The length of anchor line lying on the seabed at any time during anchor installation will be a function of at least the following factors

— the configuration of the anchor line
— the actual length of line between the anchor shackle and the pulling source (stern roller)
— the actual line tension
— the anchor line catenary (suspended part)
— the inverse catenary of the line (embedded part)
— the penetration trajectory of the anchor (position of the shackle).

At some point the length of the seabed part becomes zero and a further increase in the line tension or decrease in distance will result in a situation where the anchor line intersects the seabed under an uplift angle ($\alpha$), see Figure F-1. The characteristic anchor resistance is then given by Eq. (1) for $L_s = 0$ (Eq. (2)).

Figure F-1 illustrates two situations after hook-up to the floater. If the seabed uplift angle during design loading approaches the angle $\theta$ at the anchor shackle established during installation (extreme uplift), the anchor force and moment equilibrium from the installation stage may be affected, which may reduce the anchor resistance. This situation must be avoided. Line 2 illustrates a situation, where the uplift angle after hook-up affects the inverse catenary only down to Point A, such that the anchor is not at all affected. An acceptable uplift angle after hook-up should give a seabed uplift angle, which is significantly less than the angle $\theta$ at the anchor shackle. This would affect the installation shape (inverse catenary) of the line only to a limited depth below the seabed, indicated by Point A in Figure F-1. Guidance is given below for assessment of acceptable seabed uplift angle.

Historically both installation and operation of fluke anchors have been based on the requirement of zero uplift angle of the line at the seabed. Likely reasons for this traditional practice are listed below.

— Fluke anchors have traditionally been associated with moorings for ships and mobile drilling rigs, which often are equipped with anchors for a wide range of soil conditions, leading to minimum, or no, requirements for site specific soil investigations.
— In the mooring analyses the anchoring point has been modelled as a fixed point somewhere at the seabed, neglecting the fact that the fluke anchor embeds into the soil.
— The design approach for such anchors has been rather crude, reflecting the uncertainties in the boundary conditions, e.g. the soil data.
— Fluke anchors have been installed based on previous experience and empirical data, often extrapolated from small-scale tests.
— Only a few of the experimental data from installations have included uplift of the anchor line.

Accordingly, it has been difficult to take the step to allow for uplift, although it has been a recognised
understanding for some time that fluke anchors can accept a certain degree of vertical loading. It has, however, not been possible to quantify the effect of uplift on the anchor behaviour.

Both with respect to anchor installation and later operation of a mooring system, there will be a potential for significant cost savings if a safe uplift angle can be documented and agreed upon. In the following, guidelines are given for assessment of a safe uplift angle in normally consolidated to slightly over consolidated clay.

F.2 Assessment of a safe uplift angle

There are two situations to consider with respect to assessment of a safe uplift angle, firstly during anchor installation and secondly during extreme environmental loading after hook-up of the anchors to the floater. Non-zero uplift angles during installation typically occur when anchors are installed using a short scope of wire either by bollard pull (and blocked line) or by winch pull (from a stationary vessel).

An anchor should under no circumstances be set with an anchor line giving an initial non-zero uplift angle from start of the installation. This would reduce the possibility for the anchor to enter the soil. As a minimum, the embedment of the fluke tip should be 2.5 fluke lengths ($L_f$) before uplift is applied. This will also limit the possible maximum uplift angle for all practical means considering the path reaching an ultimate depth. An uplift angle exceeding 10° should not be expected during installation of a fluke anchor according to this procedure, even if the anchor approaches its ultimate depth.

The penetration path is only slightly affected by the uplift angles following upon the adoption of the installation procedure described above. If the anchor was to be installed to the ultimate depth using this procedure, the ultimate depth reached would be reduced only by a few per cent as a result of the increased uplift angle at the seabed. Considering that the anchor resistance is mainly a function of the penetration depth, this means that the change in anchor resistance for most installation cases is negligible.

The anchor line may have either a wire or a chain forerunner, and the effect of using one type of line or the other affects the behaviour of the anchor. An anchor penetrated with a wire will reach a larger ultimate depth than an anchor with a chain, since the soil cutting resistance is less for a wire than for a chain, see sketch in Figure 3-1. The maximum acceptable uplift angle for an anchor installed to the ultimate depth with a wire forerunner therefore becomes larger than with a chain forerunner.

Uplift angles for the permanently moored installation may be larger than those reached during anchor installation, since the installation vessel uses either long lines or a tensioner to maintain a zero, or small, uplift angle at the seabed. The line length used during hook-up to the permanent installation may in certain cases be shorter than during anchor installation, leading to higher uplift angles during storm loading than the anchor has experienced during installation. Provided that the uplift angle ($\alpha$) at the seabed is significantly less than the line angle ($\theta$) at the anchor shackle after installation the anchor resistance will not be adversely affected by this increase in uplift angle. The reason is that the shape (inverse catenary) of the forerunner below Point A in Figure F-1 will not be changed for the situation illustrated.

Line tension exceeding the available anchor resistance at any time after anchor installation will be experienced by the anchor as a sudden change in uplift angle at the anchor shackle. If the load is high enough to set the anchor in motion, the anchor resistance will drop to $R_1$ plus the loading rate effect representative of the actual overloading situation. The anchor will then, due to the higher uplift angle, follow a more shallow penetration path than during anchor installation. The penetration path becomes shallower the higher the uplift angle at the seabed is after hook-up to the floater. The maximum possible uplift angle ($\alpha_{\text{max}}$) is the angle, which makes the anchor drag at a constant depth, and gradually pulls the anchor out of the soil for higher angles. Tentatively, a safe $\alpha$-angle may be set to 50% of $\alpha_{\text{max}}$, although limited to $\alpha = 10^\circ$. In practice, this can be achieved by limiting the uplift angle to 50% of the angle $\theta$ at the anchor shackle.

In /F-1/ the effect on the anchor resistance of increasing the uplift angle after installation from 0° to $\theta/2$ is assumed to vary linearly according to the following simple expression

$$R_{L,a} = R_{L,a=0}(1 - 2\alpha/\theta) \quad \text{(F-1)}$$

(valid for $\alpha \leq \theta/2$ and $\alpha < 10^\circ$)

where $R_L$ is the contribution to the anchor resistance $R_1$ from the embedded part of the anchor line.

The design of a fluke anchor foundation, including hook-up considerations, should always ensure that extreme loads, which possibly may exceed the installation load will lead to a failure mode, which penetrates the anchor further down into the soil.

F.3 References

/F-1/ Joint Industry Project on Deep Water Anchors, Effect of uplift angle on holding capacity of fluke anchors in clay (Interim Report No. TR 109), DNV Report No. 96-3487 Rev. 01, dated 08.10.96, 18pp
APPENDIX G
GENERAL REQUIREMENTS FOR SOIL INVESTIGATIONS

G.1 Geophysical surveys
In view of fluke anchor design, the aim of a geophysical site survey will be to gather general information on a particular anchoring site. As such the objective should be to determine:

— the types and lateral/horizontal extent of different soil types/soil units/formations.
— Seabed bathymetry
— Seabed features (such as pockmarks, plough marks, erosion channels, boulders, cemented formations…)

Generally a geophysical site survey will consist in a desktop study of the available data (general geology, previous bore-holes) for the area and the geophysical survey itself. The latter usually consisting of high resolution seismic data, multi beam echo-sounding, side scan sonar. This data allows producing maps of e.g. bathymetry or soil layer thickness.

The output of such study will not provide any quantitative data of the soil strength or other geotechnical properties. A site survey can therefore not be substituted to a proper geotechnical soil investigation, but will serve as a support for interpretation of the data from such investigation and derivation of representative soil profiles for each anchor location.

Further guidance on the conduct of geophysical surveys is given in /15/ and /G-4/

G.2 Geotechnical surveys
The soil investigation should be planned and executed in such a way that the soil stratigraphy can be described in sufficient detail for both the anchor and the anchor line analysis. The required depth coverage will vary from case to case, see Section 6.

The extent of the soil investigation, sampling frequency and depth of sampling/testing, will depend on a number of project specific factors, e.g. the number of anchor locations, soil stratigraphy and variability in soil conditions with depth and between the potential anchoring points, as highlighted by the results of the geophysical survey, water depth, sea floor bathymetry, etc.

The challenge to secure soil samples of sufficient quality to determine realistic strength parameters increases with the water depth, and the efforts to improve the existing, and develop new, sampling procedures should continue. Nevertheless, in situ testing will become increasingly important for mapping of the soil conditions in deep waters.

Piezocone penetration testing (PCPT) normally provides valuable and useful information about soil stratigraphy, but the un-drained shear strength derived from such tests will be uncertain if the PCPT results are not calibrated against laboratory strength tests on recovered soil samples. If generally adopted correlation factors are used the un-drained shear strength derived will be affected by the uncertainty in this correlation factor.

One should, however, be aware of the increasing effect of sample disturbance as the water depth increases, which may lead to conservative, or non-conservative, laboratory determinations of the $s_u$-values depending on the laboratory testing procedures adopted.

If soil layering is such that the layer sequence and the variation of thickness and layer boundaries will become an important anchor design and installation consideration, it may be necessary to document the soil layer sequence at each anchor location. The thickness of all significant layers, and the thickness variation between the anchoring locations, should be known with reasonable accuracy prior to the design of the anchor foundation.

Full flow penetrometers, like the T-bar, can also be used for determination of the in situ un-drained shear strength of soft clay. The T-bar measurements are less influenced by the water depth at the test site and have the potential to measure low shear strengths at large water depth more accurately than the PCPT. However, the T-bar method is still quite new and should be used with caution in commercial projects. Some empirical factors to translate T-bar resistance to shear strength are given in /G-1/.

The number of borings and in situ testing that should be considered depends on the soil variability across the mooring pattern, which may be established by means of sub-bottom profiling. Typically, one boring and/or PCPT should be taken at each anchor location in cases where lateral variation in the soil properties are expected, or at least within each anchor cluster provided that the sub-bottom profiling shows little variation in soil properties across the mooring pattern. The soil parameters for design of fluke anchors according to the recommendations in Section 5 will require high quality push sampling in combination with advanced laboratory testing and correlations with PCPT results. If other methods for soil sampling are chosen the effects on the quality of soil samples should be considered in the design.

The soil investigation should also consider that during the detailed design process, the anchor distances and mooring leg headings may change due to changes in field layout, platform properties and mooring leg properties.
The soil investigation and soil properties interpretation should ideally provide the following information needed for the reliable design of fluke anchors for permanent mooring systems:

- definition of soil characteristics, such as general soil description, layering, etc.
- upper and lower bound un-drained direct simple shear (DSS) shear strength properties
- submerged unit weight
- soil stress history and over-consolidation ratio (OCR)
- soil consolidation, unloading and reconsolidation data (compressibility and permeability) in case of long term loading or if site specific “set-up” analyses shall be performed
- soil sensitivity, e.g. by laboratory fall cone or in situ vane tests
- cyclic shear strength under combined average and cyclic loads for DSS stress paths
- creep data to define loss of strength under sustained load (in cases where large sustained loads, e.g. loop currents, are important). As for above, cyclic stresses should be superimposed on the sustained stresses if relevant for the actual load conditions.

In case site specific “set-up” analyses are to be performed, one will also need:

- remoulded soil consolidation characteristics (compressibility and permeability)
- reconsolidated remoulded soil strength characteristics
- Thixotropy.

The setup effect on the installation anchor resistance, $R_i$, is due to the post-installation increase of the friction resistance along the walls of the fluke anchor caused by reconsolidation of the remoulded clay or thixotropy. If the setup effect along the fluke anchor walls is important for design, see Appendix D, It should be considered to perform laboratory tests on soil from the actual site.

The thixotropy effect is based on the lower bound of the database. Higher shear strength factors may be justified by running thixotropy tests on site specific clay. The laboratory tests could include DSS and oedometer tests on remoulded and intact clay and thixotropy tests. The laboratory test results can then be used together with diagrams in /D-1/, /D-3/ and /D-4/ to determine a more accurate site specific shear strength factor, $\alpha$.

For calculation of the effect of cyclic loading on the long term anchor resistance, it is recommended to carry out a series of static and cyclic un-drained DSS and triaxial tests. These tests should be carried out on representative soil samples of high quality, which shall be subjected to stress conditions that simulate the in situ conditions as closely as possible. A combined static/cyclic test programme should allow determination of the strength of the soil under the range of loading conditions that are expected to act on the anchor during a storm. Such a test programme will normally be defined so that the cyclic tests cover a representative combination of average and cyclic shear stresses. A mooring line will be subjected only to tensile loads, i.e. no compression loads, which mean that the soil surrounding the anchor will be subjected to essentially a one-way type of cyclic loading.

The cyclic laboratory tests should be run with a load period representative of the line load period, as cyclic degradation will increase with increasing load period (/G-3/).

When planning the cyclic test programme it is recommended to have in mind the subsequent use of the results, namely the construction of a strain contour diagram, as required for calculation of the cyclic shear strength ($\tau_{cy}$). The scope and content of the cyclic test programme will always have to be tailored to the actual project, the need for site specific cyclic test data versus the project budget, etc.

In general, the average shear stress level, $\tau_{av}$, representative of the design mean line tension over the design tension in a storm $T_{d,mean}/T_d$ will lie in the range 0.5 to 0.8, which implies that the cyclic testing should concentrate on acquiring test data for this range of average shear stress levels. One may also consider that the failure path, starting from a cyclic/average shear stress ratio $\tau_{cy}/\tau_{av} = 0$ in a $\tau_{cy}/\tau_{av}$ vs. $\tau_{cy}/\tau_{av}$ plot will develop along a line sloping more or less along a 45° path, either towards failure in compression, extension or DSS. It would be efficient from a testing point of view to locate the test cases so that the majority of the tests fall along this path.

Besides the cyclic tests, it will be desirable to carry out a few reference static tests, both triaxial compression, triaxial extension and DSS tests.

If site specific soil data are not provided for assessment of the cyclic loading effect, a conservative assessment of this effect is warranted. For guidance in the planning and interpretation of a cyclic test programme, and in the assessment of the effects of cyclic loading, existing data (e.g. /G-2/, /G-3/ and /E-6/) may be utilised. It is important not to underestimate the effect of cyclic loading in the absence of site specific test data.

G.3 References


/G-4/ International Association of Oil & Gas Producers (OGP), *Guidelines for the conduct of offshore drilling hazard site surveys*, Report No. 373-18-1, April 2011