

TECHNICAL REPORT

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See Conclusive summary in Chapter 1.

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1 CONCLUSIVE SUMMARY

In the installation mode, the drag-in plate anchor can be compared with a fluke anchor, but in the operational mode, after having been installed to the target installation load, the line tension is applied *normal* to the fluke (plate) area. This transition from the installation to the operational mode is termed *triggering*, which can be accomplished in different ways.

The report presents a procedure for design of drag-in plate anchors, taking into account the close relationship between the available pullout resistance in the operational mode and the applied installation load. The moderating effects of cyclic loading are also considered in the proposed design procedure.

The pullout resistance, which one can count on to resist operational and extreme loads, is expressed as a performance ratio P , times the horizontal component of the installation load T_{dip} . In the assessment of T_{dip} the proposed design procedure outlines how to account for

- the *increase* in the anchor resistance due to the difference in loading rate between the rate of wave loading and the standard rate of loading a clay specimen to failure in the *static* consolidated undrained triaxial test, and
- the *decrease* in the anchor resistance due to cyclic strength degradation during a storm.

The *cyclic loading effect* is the combined effect of both loading rate and cyclic degradation.

For practical reasons it is convenient to refer the effect of cyclic loading to a *static* pullout resistance R_S , which may be obtained by subtracting the estimated loading rate effect, which normally is included in the pullout resistance measured in an offshore test. It is proposed that R_S be used as a reference pullout resistance when adding the cyclic loading effect ΔR_{cy} . The proposed method for assessment of ΔR_{cy} follows the principles of the strain accumulation procedure used to determine the cyclic shear strength $\tau_{f,cy}$ of clay.

The design procedure is based on the *limit state method*. The design line tension is split into one mean component and one dynamic component, each component with their respective partial safety factor. The design anchor resistance R_d is the sum of R_S and ΔR_{cy} , each divided by their respective partial safety factor.

The proposed partial safety factors are *tentative* until they have been calibrated based on reliability analysis.

The total creep of an anchor during the operational period at the actual location should be evaluated as an *extra check* as part of the anchor design, and a procedure for doing this check is proposed. The intention is to establish a creep pullout resistance $R_{p,cr}$, such that mean line tensions below this load level do not lead to harmful creep of the anchor.



2 INTRODUCTION

2.1 About the Project

2.1.1 Participants

The project is organised as a joint industry project (JIP) with financial funding from the following twelve participants, which is gratefully acknowledged:

STATOIL, Norway
Saga Petroleum a.s, Norway
Det Norske Veritas, Norway
Health & Safety Executive, UK
Minerals Management Service, USA
Petrobras UK
Norsk Hydro ASA, Norway
Norske Conoco AS, Norway
BP Exploration Operating Company Limited, UK
Bruce Anchor Limited, UK
SOFEC, Inc., USA (only Part 1)
Shell Internationale Petroleum Maatschappij B.V., The Netherlands (only Part 1)

2.1.2 Brief Description of Project

The project is divided in three parts, and the objectives of the respective part-project are briefly summarised in the following.

Part 1, which was executed between August 1995 and February 1997 had the following main objectives:

- Development of a design procedure for fluke anchors in clay, utilising the results from fluke anchor tests compiled from different accessible sources and the offshore industry's general knowledge about fluke anchor performance in clay.
- Follow-up and compilation of data from drag-in plate anchor tests and identification of important design considerations and necessary further work to improve such anchors for deep-water application.
- Writing a DNV Classification Note on fluke anchors based on the work on such anchors in Part 1 (after formal completion of Part 1).

Deliverables from Part 1 comprised a total of nine Interim Reports and seven Technical Reports, plus an executable version of the computer programme DIGIN.

Part 2, duration March 1997 - 1998, focuses further on deep-water anchors in clay with the following main objectives:

- Further improvements to the DIGIN programme, e.g. better equilibrium solutions, and update of the fluke anchor back-fitting analyses from Part 1.
- Compilation of more drag-in plate anchor test data, e.g. from the DeepStar Project and Petrobras (through a confidentiality agreement).



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- Back-fitting analysis of drag-in plate anchor tests to improve our understanding of this type of anchors both during installation and pullout.
- Development of a design procedure for drag-in plate anchors.
- Specification and execution of a pilot reliability analysis of fluke anchors using the PROBAN system, with DIGIN providing the anchor-soil behaviour input and the DEEPMOOR project providing the extreme distribution of the line tension during storm.

Part 3 will comprise a full scope reliability analysis of a fluke anchor in clay with the objectives

- to develop a reliability-based design procedure for fluke anchor foundations and
- to perform a formal code calibration.

Only tentative plans have been presented to the Steering Committee, awaiting the conclusions from the pilot reliability analysis in Part 2.

2.1.3 Project Organisation

In DNV the project team consists of Rune Dahlberg (*Project Manager*), Pål J. Strøm, Trond Eklund (until 30.06.97), Jan Mathisen, Espen H. Cramer, Torfinn Hørte and Knut Olav Ronold with Knut Arnesen and Gudfinnur Sigurdsson as *Verifiers*, Øistein Hagen as *QA Responsible* and Arne E. Løken as *Project Responsible*.

The *Steering Committee*, composed of one representative from each participant with Asle Eide from Statoil as *Chairman*, contributes to a validation of the final products from the project by approving plans and reviewing and commenting on the Draft Final Reports.

2.2 The Present Report

This technical report, "Design procedure for drag-in plate anchors", is the final result of the work covered by activity 230 of the joint industry project on "Design Procedures for Deep Water Anchors, Part 2: Further Work on Anchors in Clay." Based on the design procedure presented herein DNV will develop a Recommended Practice for design of drag-in plate anchors as a post-project activity (covered by sub-activity 233).

The motivation for introducing the drag-in plate anchor concept has been that taut mooring systems (TMS), as opposed to conventional catenary mooring systems, transmit significant vertical load components to the anchors in addition to the horizontal components. A TMS will occupy much less area on the seabed than a conventional catenary system, since the mooring lines typically have angles with the horizontal between 30° and 45°, which may be slightly reduced close to the seabed by adding a chain segment. This means that the mooring lines intersect the seabed under a relatively large uplift angle, which requires anchors capable of resisting both vertical and horizontal load components.

In a taut mooring system the mooring lines are made up of synthetic fibre ropes, e.g. polyester. A design procedure for mooring lines of floating offshore structures is provided in the POSMOOR Rules, which currently (1998) are under revision /1/.

Drag-in plate anchors are from an installation point of view comparable with fluke anchors, see DNV Recommended Practice No. RP 601 /2/, see also /3/. They have, however, the additional feature of acting as a plate anchor in their operational mode. The transition from a fluke anchor to a plate anchor function, termed *triggering*, may be accomplished in different ways, but is also anchor type dependent.



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The pullout resistance normal to the plate (fluke), which is the resistance of interest from a design point of view, is related to the horizontal component of the installation load through an anchor performance ratio P_r , which is an important factor in the design of drag-in plate anchors.

An investigation into the effects of uplift on the behaviour of fluke anchors and drag-in plate anchors within this joint industry project has provided a basis for assessment of acceptable uplift angles for installation of drag-in plate anchors.

According to this recommendation the geotechnical design of drag-in plate 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 Limit State (ALS) condition. The design procedure presented herein is primarily applicable to permanently anchored installations.

The material and load factors proposed at this stage are for temporary use only, until a formal calibration of the partial safety factors has been carried out.



3 GLOSSARY AND DEFINITION OF TERMS

The glossary and definition of terms following is purposely somewhat extended, such that it may also serve as a quick reference for the relationship between different terms and safety aspects. Many of the terms are identical to those used in RP 601 for fluke anchors /2/, and others have been added as relevant for drag-in plate anchors. More details about the respective terms are found in the remainder of the report.

	<i>Dip-down point</i>	The point on the seabed, where the anchor line starts to embed.
	<i>Touch-down point</i>	The point at the seabed, where the suspended catenary part of the anchor line first touches the seabed.
<i>R</i>	<i>Anchor resistance</i>	The resistance of the embedded anchor <i>plus</i> the embedded part of the anchor line
<i>R_{ult}</i>	<i>Ultimate anchor installation resistance</i>	The anchor installation resistance at ultimate penetration <i>z_{ult}</i> The anchor does not penetrate any deeper during <i>continuous</i> penetration, but drags at a constant depth without further increase in the installation line tension.
<i>z_{ult}</i>	<i>Ultimate penetration</i>	This penetration is a function of the type and size of the anchor, the soil conditions and the installation uplift angle $z_{ult} = \lambda \cdot F_L$
<i>F_L</i>	<i>Equivalent fluke length</i>	Set equal to square root of fluke area, i.e. $F_L = \sqrt{A_{fluke}}$
λ	<i>Ultimate depth factor</i>	Varies typically between 6 and 12 for soft clays. Should not be set >8 without site specific test data.
<i>R_{dip}</i>	<i>Anchor installation resistance</i>	The horizontal component of the <i>measured</i> anchor installation resistance equal to (or higher than) the target installation load (<i>T_{dip}</i>) in the <i>dip-down</i> point.
<i>U_{cons}</i>	<i>Consolidation factor</i>	Factor, which gives the consolidated anchor (installation) resistance <i>R_{cons}</i> when multiplied with <i>R_{dip}</i>
<i>R_{cons}</i>	<i>Consolidated anchor resistance</i>	Anchor (installation) resistance including the consolidation effect, i.e. $R_{cons} = U_{cons} \cdot R_{dip}$ (<u>to be avoided!</u>)
<i>R_{pu}</i>	<i>Ultimate pullout resistance</i>	The resistance at ultimate depth of penetration <i>z_{ult}</i> . The anchor may be sized to resist <i>R_{pu}</i> .
<i>M</i>	<i>Mobilisation factor</i>	Degree of mobilisation of <i>R_{pu}</i> $R_{pi} = M \cdot R_{pu}$
<i>R_{pi}</i>	<i>Anchor installation</i>	The pullout resistance of the plate (fluke) 'immediately'



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	<i>pullout resistance</i>	after anchor installation in the <i>dip-down</i> point (loading rate or speed dependent)
P_r	<i>Anchor performance ratio</i>	The ratio between R_{pi} and T_{dip} , i.e. $P_r = R_d / T_{dip} \quad (R_d \text{ from geotechnical calculations})$ $P_r = R_{pi} / T_{dip} \quad (R_{pi} \text{ from anchor tests})$
R_S	<i>Static pullout resistance</i>	$R_S = \beta \cdot R_{pi}$ <ul style="list-style-type: none"> • β to be assessed from case to case • currently $\beta = 0.80$ is recommended
$R_{p,cr}$	<i>Creep pullout resistance</i>	$R_{p,cr} = \rho \cdot R_S$ <ul style="list-style-type: none"> • to be assessed from case to case (soil dependent) • currently $\rho = 0.75$ is recommended
ΔR_{cy}	<i>Cyclic loading effect</i>	<i>Predicted</i> contribution to the anchor pullout resistance from the effect of cyclic loading
U_r	<i>Loading rate factor</i>	Used herein also in the meaning of loading (or strain) rate factor
U_{cy}	<i>Cyclic loading factor</i>	Factor, which gives the characteristic pullout resistance R_C when multiplied with R_{pi}
$R_{p,cy}$	<i>Cyclic pullout resistance</i>	Pullout resistance including the effects of cyclic loading
R_A	<i>Approximate pullout resistance</i>	First estimate of the required pullout resistance (Step (2) in design procedure) $R_A = k_A \cdot T_d$
k_A	<i>Approximate safety factor on anchor resistance</i>	Used as an approximation in the first estimate of the required pullout resistance (Step (2) in design procedure)
A_{fluke}	<i>Fluke (plate) area</i>	The projected area of the anchor fluke (or plate).
F_L	<i>Equivalent fluke length</i>	Proportional to square root of fluke area, i.e. $F_L = \kappa \sqrt{A_{fluke}}$ where κ is dependent on anchor type, typically $\kappa \approx 1.25$
R_C	<i>Characteristic pullout resistance</i>	The anchor static pullout resistance R_S <u>plus</u> the <i>predicted</i> cyclic loading effect ΔR_{cy} at the installation depth z_i , i.e.: $R_C(z_i) = R_S(z_i) + \Delta R_{cy}(z_i) = R_S(z_i) \cdot U_{cy} \quad (=R_{p,cy}(z_i))$
R_d	<i>Design pullout resistance</i>	The anchor pullout resistance in the <i>dip-down</i> point with material factor γ_m included: $R_d(z_i) = R_S(z_i) / \gamma_{m1} + \Delta R_{cy}(z_i) / \gamma_{m2}$
z_i	<i>Installation depth of anchor</i>	Depth related to the design pullout resistance coming out of the anchor design process.



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γ_{m1}	<i>Material factor</i>	<p>Accounts for the uncertainty in</p> <ul style="list-style-type: none"> • $s_u(z_i)$ and $s_{u,r}(z_i)$, U_r and reference strain rate v_{ref} • the prediction method and the analytical model
γ_{m2}	<i>Material factor</i>	<p>Accounts for the uncertainty in</p> <ul style="list-style-type: none"> • $s_u(z_i)$ • the cyclic test data used and U_{cy} • the prediction method and the analytical model
	<i>Soil consolidation</i>	<p>A time dependent process, which leads to an increase in the anchor resistance as the undrained shear strength gradually regains its <u>intact</u> strength after having been remoulded. The maximum possible increase is a function of the <u>soil sensitivity</u> (S_t) and the anchor geometry.</p> <p>N.B.: The consolidation effect on the pullout resistance is set to zero</p>
	<i>Cyclic loading</i>	<p>Affects the static undrained shear strength (s_u) in <i>two ways</i>:</p> <ul style="list-style-type: none"> • During a storm, the rise time from mean to peak load may be about 3 - 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. The higher loading rate leads to an <i>increase</i> in the undrained shear strength • As a result of repeated cyclic loading during a storm, the undrained shear strength will <i>decrease</i>, and the degradation effect increases with the overconsolidation ratio (<i>OCR</i>) of the clay.
$\tau_{f,cy}$	<i>Cyclic shear strength</i>	<p>Accounts for both the loading rate effect and the cyclic degradation effect and is the <u>preferred</u> characteristic soil strength for use in the design of <i>drag-in plate</i> anchors.</p> <p>$\tau_{f,cy}$ is calculated according to the strain accumulation method, which utilises so-called strain-contour diagrams to describe the response of clay to various types, intensities and duration of cyclic loading.</p> <ul style="list-style-type: none"> • <u>Determination of $\tau_{f,cy}$:</u> A clay specimen with a certain s_u and <i>OCR</i> is subjected to a load history defined in terms of a sea state and a storm duration. The intensity of that load history is gradually increased until the soil fails in cyclic loading. • <u>Line loads in a mooring system:</u> 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 (stress reversal). 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 $\tau_{f,cy}$. • <u>Representative load history:</u> The load history for use in the calculations should account for the



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		combination of wave-frequency load cycles superimposed on low-frequency, slowly varying, load cycles and mean tension, particularly the amplitude of cyclic loads relative to a defined average (or mean) load level during the storm.
<i>OCR</i>	<i>Overconsolidation ratio</i>	The ratio between the maximum <u>past</u> effective vertical stress on a soil element and the <u>present</u> effective vertical stress acting on the same soil element. <ul style="list-style-type: none"> The higher the <i>OCR</i> is, the more strength degradation due to cyclic loading and the less strength increase due to an increase in loading rate. For a normally consolidated (<i>NC</i>) clay the <i>OCR</i> = 1
<i>s_u</i>	<i>Intact strength</i>	The static undrained shear strength, which is the best measure of the <i>in situ</i> undisturbed (intact) soil strength.
<i>s_{u,av}</i>	<i>Average intact strength</i>	Undrained shear strength which accounts for strength anisotropy, often set equal to $s_{u,av} = (s_{u,E} + s_{u,D} + s_{u,C})/3$ where <ul style="list-style-type: none"> <i>s_{u,E}</i> = consol. undrained triax. extension strength <i>s_{u,D}</i> = consol. constant volume DSS strength <i>s_{u,C}</i> = consol. undrained triax. compression strength
<i>s_{u,D}</i>	<i>DSS intact strength (used in procedure)</i>	In many cases the effect of <i>strength anisotropy</i> may be accounted for simply by setting <i>s_{u,av}</i> = <i>s_{u,D}</i> , the direct simple shear (DSS) strength, as done herein. The justification of this should be evaluated from case to case. (In the procedure <i>s_{u,D}</i> has been shortened to <i>s_u</i>.)
<i>s_{u,UU}</i>	<i>UU intact strength</i>	Undrained shear strength measured in an unconsolidated undrained (UU) triaxial test.
<i>s_{u,r}</i>	<i>Remoulded shear strength</i>	The undrained shear strength measured e.g. in a UU triaxial test after having remoulded the clay completely.
<i>S_t</i>	<i>Soil sensitivity</i>	The ratio between <i>s_u</i> and <i>s_{u,r}</i> , as determined e.g. by UU triaxial tests (fall-cone tests may be an alternative).
<i>α_{soil}</i>	<i>Adhesion factor</i>	
<i>α_{min}</i>	<i>Minimum adhesion factor</i>	Set equal to 1/ <i>S_t</i>
<i>η</i>	<i>Empirical factor</i>	Accounts for the effects on <i>R_d</i> of soil remoulding and inclined/excentric anchor loads (default value <i>η</i> =0.73). See discussion of this factor in step (2b) of the design procedure in Section 5.7.2.
<i>β</i>	<i>Strain rate factor</i>	Adjustment for strain rate of the pullout resistance <i>R_{pi}</i> measured in an offshore test when calculating <i>R_S</i> (default value <i>β</i> = 0.8 based on current test data base)
<i>N_c</i>	<i>Bearing capacity factor</i>	Theoretically <i>N_c</i> = 12.5 for an infinitely long plate.



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s_c	<i>Shape factor</i>	For a typical drag-in plate anchor in clay $s_c \approx 1.1$.
T_{C-mean}	<i>Characteristic mean line tension</i>	The calculated mean line tension at the <i>touch-down</i> point for the limit state under consideration *
T_{C-dyn}	<i>Characteristic dynamic line tension</i>	The calculated dynamic line tension at the <i>touch-down</i> point for the limit state under consideration
T_C	<i>Characteristic line tension</i>	The combined line tension at the <i>touch-down</i> point for the limit state under consideration $T_C = T_{C-mean} + T_{C-dyn}$
T_d	<i>Design line tension</i>	$T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn}$
γ_{mean}	<i>Partial safety factor on mean line tension</i>	Accounts for the uncertainty in mean line tension
γ_{dyn}	<i>Partial safety factor on dynamic line tension</i>	Accounts for the uncertainty in dynamic line tension
T_{dip}	<i>Target installation load</i>	The horizontal component of the line tension at the <i>dip-down</i> point during anchor installation.
T_{touch}	<i>Minimum installation load</i>	The target installation load T_{dip} plus the factored seabed friction over length L_s of the anchor line on the seabed $(\mu \cdot W' \cdot L_s) \cdot \gamma_{m,i}$ at installation The T_{touch} is to be maintained for a period of 20-30 minutes and documented by measurements. If the load fluctuates due to movements of the installation vessel, the T_{touch} shall be the minimum load level during these fluctuations. Any uncertainty in the load measuring system to be accounted for.
$\gamma_{m,i}$	<i>Material factor</i>	Accounts for the uncertainty in the predicted seabed friction during anchor installation

* The line tension model applied in this document corresponds to a revised version of DNV's rules for Position Mooring (POSMOOR) that is currently under preparation.



4 DESIGN CONSIDERATIONS

4.1 General.

Design considerations related to drag-in plate anchors are concerned with:

- anchor installation resistance, penetration and drag
- target installation load T_{dip} and anchor performance ratio P_r .
- installation scenarios and procedures
- effect of loading rate and cyclic degradation (cyclic loading)
- analytical tools used for prediction of anchor behaviour.

In the following, these aspects will be discussed followed by a description of the recommended design procedure. Reference is made to the nomenclature in Chapter 3 for glossary and definition of terms in connection with design and installation of drag-in plate anchors.

The main components of a drag-in plate anchor are (Figure 1):

- the shank (rigid or wire system)
- the fluke (plate), and
- the shackle

Although it would be more appropriate to use the word *plate* rather than *fluke* when drag-in plate anchors are discussed, the words *fluke* and *fluke angle* are maintained, since a drag-in plate anchor is basically a fluke anchor as far as installation is concerned.

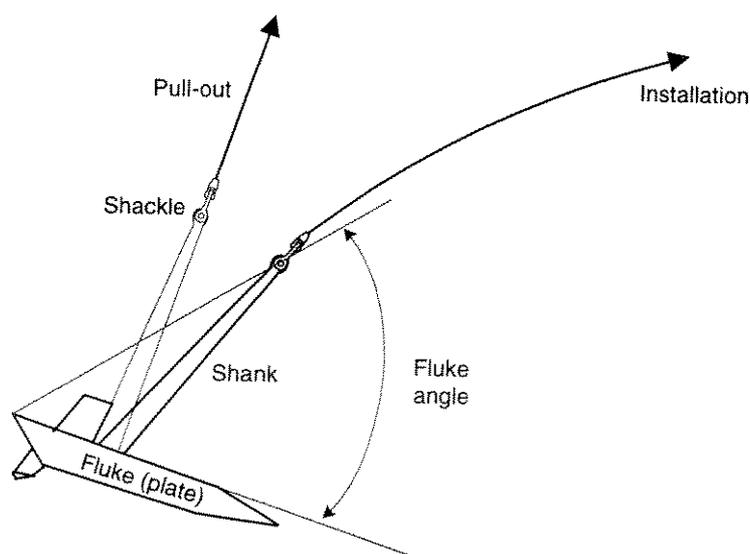


Figure 1 Main components of a drag-in plate anchor.

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. Other definitions exist, and if one of these are used it should be clearly stated how the angle is defined.



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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 is significantly affected by the type (wire or chain) and size of the forerunner.

The *inverse catenary* of the anchor line is the curvature of the embedded part of the anchor line.

4.2 Anchor resistance, penetration and drag.

4.2.1 Anchors in clay without significant layering

The resistance of an anchor depends on the ability of the anchor to penetrate and to reach the required target installation load T_{dip} .

The penetration path and ultimate depth of penetration is a function of

- the soil conditions (soil layering, variation in intact and remoulded undrained shear strength)
- the type and size of anchor,
- the anchor's fluke angle,
- the type and size of the anchor forerunner (wire or chain), and
- the line uplift angle at the seabed level.

It should be mentioned that the predictability of the new drag-in plate anchors may be much improved by doing site-specific tests with instrumented anchors, see Section 5.10.3. The predicted ultimate penetration z_{ult} of the anchor is crucial for sizing the anchor given T_{dip} and the shear strength profile.

A drag-in plate anchor is normally penetrating along a path, where the ratio between incremental penetration and drag decreases with depth, see example in Figure 2.

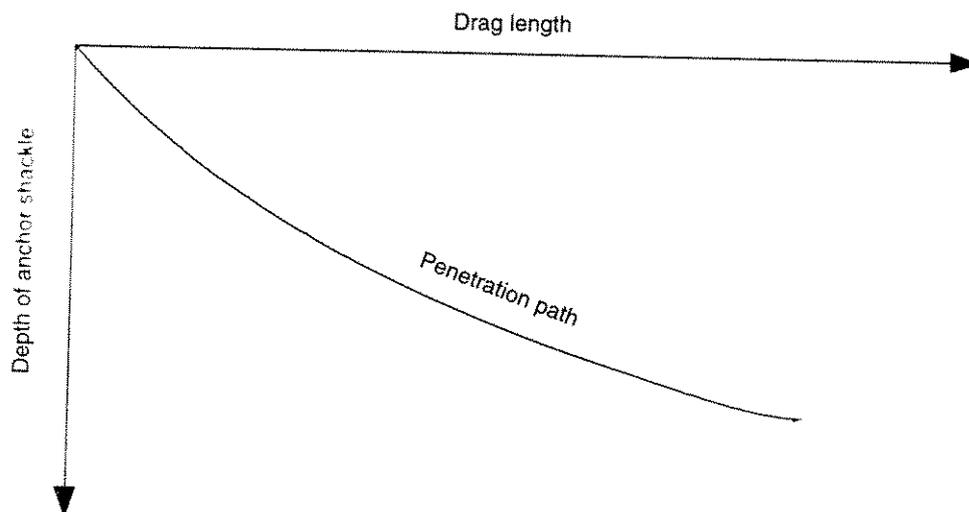


Figure 2 Typical drag-penetration relationship for a drag-in plate anchor.

At a certain depth, the ultimate depth, the anchor is not penetrating any further. The anchor is "dragging" with a horizontal (or near horizontal) fluke, and the tension in the line is constant. The ultimate depth z_{ult} varies with the consistency (undrained shear strength) of the clay. At this depth the anchor reaches its ultimate penetration resistance R_{ult} , see illustration in Figure 3.

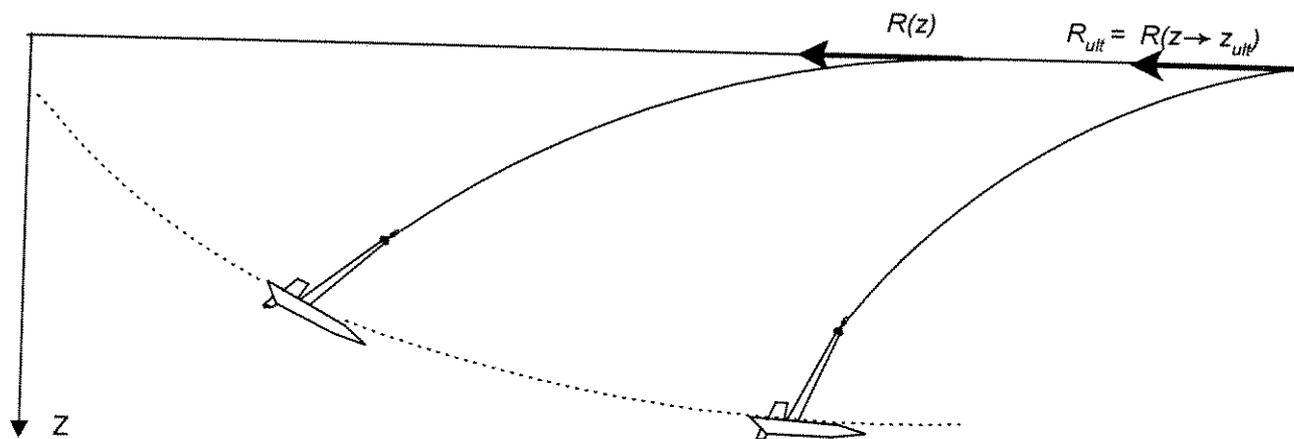


Figure 3 Definition of ultimate anchor resistance R_{ult} .

It is important not to overestimate z_{ult} . In the worst case the target installation load T_{dip} will not be reached before the anchor starts dragging without further increase in the anchor resistance. To avoid this the design (sizing) of the anchor should *not* rely on full mobilisation M of the ultimate anchor penetration resistance. On the other hand the anchor should reach a penetration of minimum 3 fluke widths to ensure that the boundary conditions for assuming deep failure are satisfied in the computation of the anchor pullout resistance. A degree of mobilisation in the range $M = 0.40$ to 0.80 is recommended with 0.75 as a tentative default value.

It is important to have a clear definition (although arbitrarily) of how the fluke angle is to be measured. With the definition given in Figure 1 the fluke angle is normally varied between 30° and 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.

If the soft clay is overlain by a sand or a stiffer clay the 50° fluke angle may have to be combined with a smaller angle, for example 30° , to ensure initial penetration of the anchor into and through the top layer. By designing the shear pin controlling the 30° fluke angle such that it breaks for a load corresponding to a fluke position well into this top layer, the fluke angle will then open to 50° as suitable for the underlying soft clay. See more about anchors in layered clay in Section 4.2.2.

The cutting resistance of a chain forerunner will be greater than the resistance of a steel wire, with the result that the inverse catenary for a chain forerunner will be much steeper than for a wire forerunner. The consequence is that a drag-in plate anchor with a chain forerunner will penetrate less than one with a wire forerunner, and mobilise less resistance for a certain drag distance. As a consequence the pullout resistance for any given drag will be less than for a drag-in plate anchor with a wire forerunner.

In translating the results from the actual anchor installation, proper adjustments will have to be done if the measured installation load includes seabed friction, including effect of possible



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misalignment of the anchor installation line. The target installation load T_{dip} refers to the dip-down point and any extra resistance, which needs to be overcome up to that point has to be added to the installation load, see further about anchor installation in Section 5.9.

The anchor resistance R is defined as the mobilised resistance against the anchor *plus* the resistance along the embedded part of the anchor line, i.e. up to the *dip-down* point. However, drag-in plate anchors in deep water may normally be installed with an uplift angle in the final stage of the installation, in which case there will be no line on the seabed.

Although drag-in plate anchors are designed to resist loads with significant vertical components, the uplift angle during installation should be close to zero until a certain depth of penetration has been reached, whereupon a gradual increase in the uplift angle can be accepted. If the installation angle becomes too large the anchor penetration path will, however, be shallower giving less anchor resistance compared to a situation with zero uplift, see more about uplift in Section 5.6.

4.2.2 Anchors in layered clay

Drag-in plate anchors are particularly suitable for soft normally consolidated clays, but experience has shown that they often penetrate through an overlying layer of sand or stiffer clay as long as the thickness of this layer is less than 30 to 50 % of the fluke length of the actual anchor.

In a *soft-stiff* layer sequence the anchor should normally stay in the soft layer and avoid partly penetration into the stiff layer. Since the pullout resistance will be governed by the undrained shear strength of the soft overlying clay, a target installation load related to the penetration resistance of the stiffer clay will be misleading. If predictions or anchor tests show that there is a risk that the target installation load cannot be reached without penetration into the stiffer layer, changing to another type and/or size of anchor may improve the situation. If drag-in plate anchors at all should be used is dependent on the thickness of the soft layer and the loads, which have to be resisted.

A *stiff-soft-stiff* layer sequence will in most circumstances involve extra complications in that penetration through the upper stiff layer may require a smaller fluke angle than desirable for penetration through the locked-in soft layer. Again, the drag-in plate anchor should be designed to stay within the soft layer and avoid partial penetration into the underlying stiff layer. If the strength of the locked-in soft layer is smaller than assumed in designing the anchor, the target installation load may not be reached, visualised by continuous drag at constant load. Designing the anchor for less than ultimate penetration as discussed in Section 4.2.1 may reduce this risk. 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. In many cases it may be possible to find an optimal, non-standard, combination between anchor size and fluke angle, which accounts both for the overlying and the underlying stiff layer and ensures that the anchor stays within the soft clay layer in between. For considering drag-in plate anchors at all in layered soil the target clay layer must be reachable and have a strength and thickness, which confidently can be utilised to provide a safe pullout resistance.

From the above it is evident that layer thickness, and depth to boundaries between layers, need to be documented for proper design of a drag-in plate anchors and to avoid unexpected behaviour



of the anchor during the installation phase, see further about requirements to soil investigation in Section 5.10.

4.3 Installation and testing of drag-in plate anchors.

The database for drag-in plate anchors loaded to their ultimate resistance R_{ult} is unfortunately limited to rather small anchors. The largest anchors tested in connection with offshore projects have normally not reached the R_{ult} , but for the future it would be fruitful for the industry if the most significant parameters (tension force, drag and penetration) are recorded during all installations. 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. Since the design pullout resistance of a drag-in plate anchor is made dependent on the measured and documented target installation load, it is essential that the installation measurements are as reliable as possible, and on the conservative side. If the anchor installation load is reported to be higher than it actually is, the resulting pullout resistance of the anchor will be smaller than assumed in the design. By prescribing a minimum installation load T_{touch} , see Section 5.8.2, the intention is to ensure that the design assumptions are fulfilled during anchor installation.

The design curves published by the American Petroleum Institute in /4/, which are based on work by the Naval Civil Engineering Laboratory (NCEL), give the ultimate anchor resistance R_{ult} of the respective anchors. These diagrams, which include no curves for drag-in plate anchors, 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 fluke anchors and corresponds to a safety factor of 1.0. 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 mobilisation. After such correction the resulting anchor resistance may be comparable with the installation anchor resistance R_{dip} 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.

Most of the anchor tests in the database for fluke anchors are with a chain forerunner, whereas all drag-in plate anchor tests performed so far have been with a wire forerunner. The choice of forerunner has a significant effect on the ultimate depth penetration and needs to be addressed in the anchor design. There are many limitations in a design method relying on the Power Law Method, which justifies using a design procedure based on *geotechnical* principles.

4.4 Analysis tools for drag-in plate anchor design

4.4.1 General

An analytical tool for drag-in plate anchor design should be able to calculate anchor line catenary in soil as well as the drag-in plate anchor equilibrium itself, both during installation and pullout. Further, the analytical tool should be able to assess the effect of consolidation as being an important design issue in soft clay. This section describes in brief the principles of such an analytical tool.



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4.4.2 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 /10/.

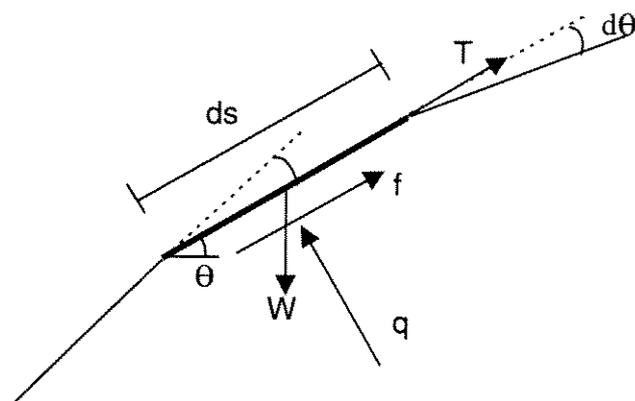


Figure 4 Soil stresses at an anchor line segment in soil.

The normal resistance to the anchor line is calculated from the following equation:

$$q = N_c \cdot s_u \quad (1)$$

where

- N_c = bearing capacity factor
- s_u = undrained shear strength

Effect from embedment on the bearing capacity factor should be included.

The unit friction r_{fric} for the anchor line can be calculated from the following formula:

$$f = \alpha_{soil} \cdot s_u \quad (2)$$

where

- α_{soil} = adhesion factor
- s_u = undrained shear strength

The loss in line tension over one element length is calculated from the following formula:

$$\frac{dT}{ds} = -f \cdot d - W' \cdot \sin(\theta) \quad (3)$$

where

- T = anchor line tension
- θ = orientation of anchor line element ($\theta = 0$ for a horizontal element)
- W' = submerged weight of the anchor line per unit length
- f = unit friction
- d = effective surface of anchor line
- ds = element length



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The angular advance from one anchor line element to the next is then solved by iterations from the following formula:

$$\frac{d\theta}{ds} = \frac{q \cdot B - W' \cdot \cos(\theta)}{T} \quad (4)$$

where

- q = normal resistance
 B = effective bearing area of anchor line

4.4.3 Equilibrium equation for drag-in plate anchor

Moment equilibrium and force equilibrium can be solved for the drag-in plate anchor for two different failure modes. One for which the anchor will penetrate in the same direction as the fluke orientation in the soil, and a second where the penetration direction deviates from the fluke orientation. The principle with respect to soil resistance contributions is similar, however in the first mode the soil resistance normal to the fluke may not take on the ultimate value.

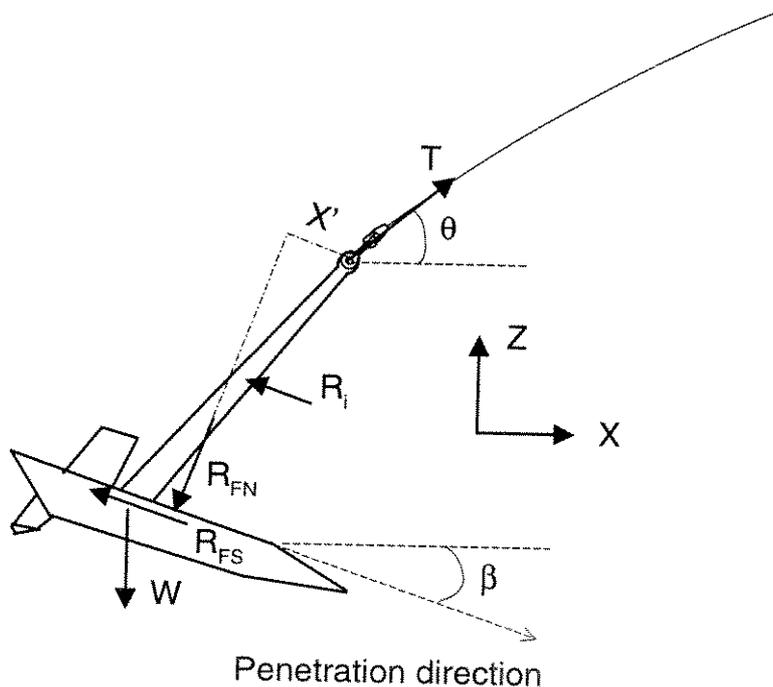


Figure 5 Principal soil reaction forces on a drag-in plate anchor (orientation coincides with anchor penetration direction).

For the range of penetration directions, horizontal and vertical equilibrium should satisfy the following equations:

Horizontal equilibrium:



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$$T \cdot \cos(\theta) = \sum_{i=1}^N R_i \cdot \cos(\beta) + R_{FS} \cdot \cos(\beta) - R_{FN} \cdot \sin(\beta) \quad (5)$$

Vertical equilibrium

$$T \cdot \sin(\theta) = \sum_{i=1}^N R_i \cdot \sin(\beta) + R_{FS} \cdot \sin(\beta) - W - R_{FN} \cdot \cos(\beta) \quad (6)$$

where

- T, θ = 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_i = soil resistance at the remaining components of the anchor
(separated through anchor geometry)
- W = anchor weight
- β = penetration direction of anchor

The magnitude of the various resistance contributions can in principle be calculated by the same equations as presented for stresses normal and tangential to the anchor line, Eq. (1) and Eq. (2).

Horizontal and vertical equilibrium for a certain penetration direction can now be achieved for a number of anchor orientations and 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):

$$\sum_{i=1}^N Rm_i + Rm_{FS} - Wm - R_{FN} \cdot X' = 0 \quad (7)$$

where

- Rm_{FS} = moment contribution from soil sliding resistance at the fluke
- Wm = moment contribution from anchor weight
- R_{FN} = soil normal resistance at the fluke
- X' = distance from shackle to centre of normal resistance at the fluke
- Rm_i = moment contribution from soil resistance at the remaining components
of the anchor (separated through anchor geometry)

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

When several solutions are found, the one giving the lowest tension should be selected.

In Figure 6 an example of a back-fitting analysis with the DIGIN program /11/ is shown. In this case the anchor installation records included measurements of line tension at the fairlead, drag, the final depth of anchor penetration, the anchor line configuration and undrained shear strength



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profile. Through an iteration process the measured and predicted anchor behaviour is gradually improved until a satisfactory match is found as shown in Figure 6. The calibration of the program is based on a number of such back-fitting analyses /12/.

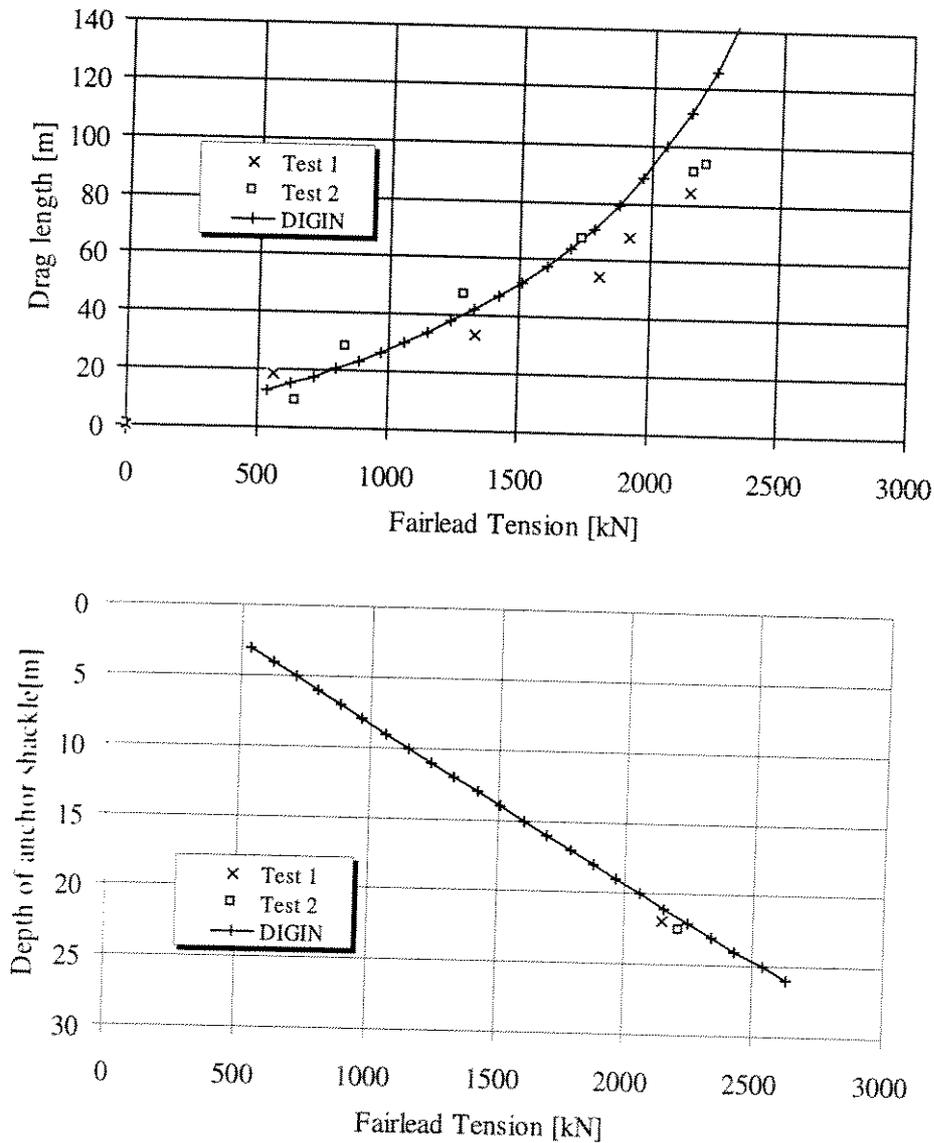


Figure 6 Example of DIGIN back-fitting analysis.



5 DESIGN PROCEDURE FOR DRAG-IN PLATE ANCHORS

5.1 General.

The procedure for design of drag-in plate anchors recommended herein is based on the limit state method of design. In an actual design situation the designer would benefit from having an adequate analytical tool at hand for parametric studies.

The analytical tool should account for the interaction between the anchor, the soil and the applied line tension and provide relationships between anchor drag, penetration and resistance for the actual type and size (anchor weight and fluke area) of anchor and soil strength profile. Since the anchor resistance is dependent on both anchor orientation within the soil and penetration direction, it is essential that the analytical tool is able to calculate the force and moment equilibrium of the anchor when subjected to a given line tension force.

The analytical tool should be based on geotechnical principles and calibrated against high quality anchor tests. The development and validation of such a tool should make use of results from tests with instrumented full-scale anchors in a well-documented soil. Guidance for analysis of anchor behaviour is given in Section 4.4.

The anchor line influences the anchor behaviour and should be incorporated as an integral part of the anchor analysis. The size of the anchor line affects the maximum depth of penetration and consequently also the ultimate anchor resistance.

In normally consolidated clays, where the undrained shear strength increases with depth, the analyses may show that the anchor mobilises stronger soils the deeper it penetrates, which is not reflected in a simple power formula approach or log-log design diagrams as included in /4/.

Sound engineering judgement should always be exercised in the assessment of the characteristic resistance of the chosen anchor, giving due consideration to the reliability of the analytical tool and the uncertainty in the design parameters provided for the site. A drag-in plate anchor, in its intended operational mode, orients itself such that the fluke plane (plate) is normal to the direction of loading, which means that the soil disturbance due to penetration of the anchor in a direction parallel to the fluke plane has only marginal effect on the pullout resistance. It is therefore logical to disregard the consolidation effect on the pullout resistance.

The effect of cyclic loading may, however, contribute to the pullout resistance, although the effect may be difficult to document in practice, see further in Section 5.4.

The database for drag-in plate anchor tests is still limited, but some well-instrumented tests have provided valuable data and good insight into the behaviour of drag-in plate anchors. Offshore tests do not give sufficient information about all relevant parameters from a back-fitting analysis point of view. In most cases there are uncertainties attached to the reported installation data, e.g. soil stratigraphy, soil strengths, anchor installation load, contribution from sliding resistance along the anchor line segment on the seabed, depth of anchor penetration, possible effect of anchor roll or pitch during penetration, pullout resistance, pull-in and pullout speed, etc.

It is therefore of general interest that future drag-in plate anchor testing, and monitoring of commercial anchor installations, be carefully planned and executed, such that the test database gradually improves. Extrapolation from small to medium size anchor tests to prototype size



anchors should be made with due consideration of possible scale effects, preferably by use of a suitable analytical tool as discussed in Section 4.4.

5.2 Basic nomenclature and contributions to anchor resistance

The nomenclature used in the design procedure for drag-in plate anchors is basically the same as that used in /2/ for fluke anchors, see Figure 7.

The anchor installation resistance R_{dip} refers to the *dip-down* point and is the *horizontal* component of the anchor resistance in that point. The mobilisation of this resistance is verified during anchor installation by reaching the specified target installation load T_{dip} in the same point, which load is maintained during a specified period of time, see further Section 5.9. T_{dip} may be derived from the *measured* minimum installation load T_{touch} in the touchdown point. If some length of the anchor line is lying on the seabed when T_{touch} is reached the resulting seabed friction must be calculated and subtracted to get T_{dip} , see Eq. (8).

$$T_{touch} = T_{dip} + (\mu \cdot W' \cdot L_s) \cdot \gamma_{m,i} \quad (8)$$

where

L_s = length of anchor line on the seabed when the horizontal component of the line tension in the dip-down point equals T_{dip}

W' = submerged weight per metre of the anchor line segment on the seabed.

μ = friction coefficient applicable for the type of forerunner and seabed soil

$\gamma_{m,i}$ = material factor on the predicted seabed friction to be overcome by the installation load.

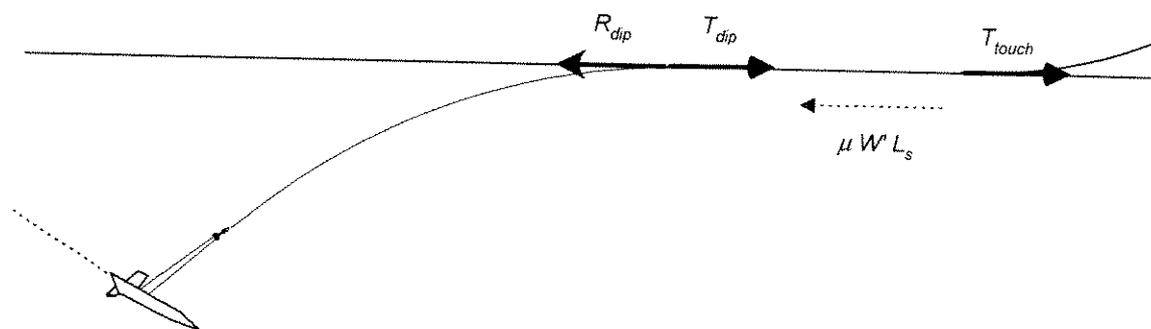


Figure 7 Nomenclature related to anchor installation.

If the anchor installation is performed with an uplift angle at the seabed towards the end of the installation the seabed friction term may of course be set to zero, and a situation as shown in Figure 8 applies. The anchor installation resistance R_{dip} shall be established by applying an installation load with a horizontal component in the dip-down point equal to the target installation load T_{dip} .

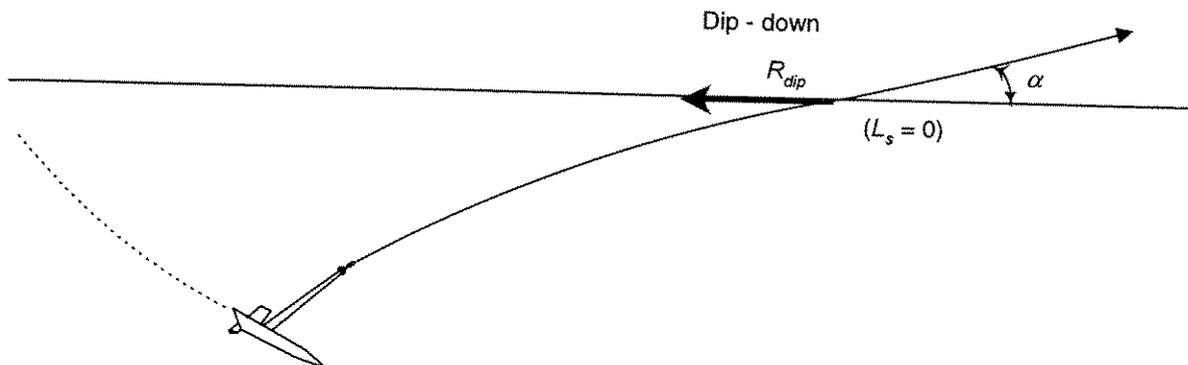


Figure 8 Installation of a drag-in plate anchor with an uplift angle at the seabed.

The assessment of the target installation load T_{dip} is a crucial design decision, which to a large extent is governed by the anchor performance ratio P_r , as shown in Figure 9.

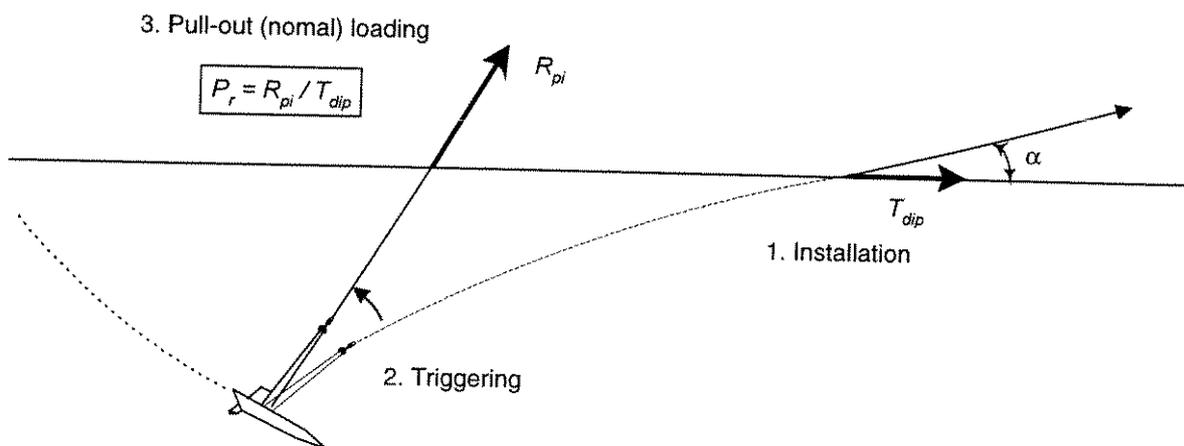


Figure 9 Considerations in the design of a drag-in plate anchor.

After installation of the anchor to satisfy the target installation load T_{dip} the anchor is *triggered*, which means that the anchor is prepared to resist the operational and extreme loads as an embedded plate oriented such that the loads are being applied normal to the plate. This triggering step can be accomplished by breaking the shear pin, which controls the fluke angle, as for the *Denla* anchor from Bruce Anchor sketched in Figure 9. Another alternative is to attach both the installation line and the mooring line to the anchor shackle through a triplate arrangement. The installation line then controls the fluke angle and the normal loading (triggered) position of the anchor is achieved simply by pulling in the mooring line. This scenario has been proposed for the *Stevmanta* anchor from Vryhof Ankers, which has an 'imaginary' shank consisting of four wires attached to the corners of the fluke and coupled together at the triplate (angle adjuster). A similar scenario can be obtained with the *Denla* anchor sketched in Figure 9.



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The triggered anchor will have a pullout resistance immediately after installation of the anchor, R_{pi} , which normally is set equal to the target installation load T_{dip} times the performance ratio P_r , as shown in Figure 9, i.e.

$$R_{pi} = P_r \cdot T_{dip} \quad (9)$$

It should, however, be noted that the installation pullout resistance R_{pi} varies with the rate of pulling the anchor to failure. As will be further discussed in Section 5.4.2 it is practical to define a *static* pullout resistance R_S equal to R_{pi} divided by a loading rate factor U_r , i.e.

$$R_S = (1/U_r) \cdot R_{pi} = \beta \cdot R_{pi} \quad (10)$$

where β will appear in some of the expressions in the design procedure in Section 5.7.2. The performance ratio P_r in Eq. (2) therefore refers to a situation where the anchor is pulled to failure at a rate similar to that used in an offshore test. Based on back-fitting analysis of field test data a typical loading rate effect may be represented by $U_r = 1.25$, giving $\beta = 0.80$.

Both the offshore and the onshore testing of drag-in plate anchors have focussed a great deal on the performance ratio. The *Denla* and *Stevmanta* anchors tested under controlled onshore conditions have given performance ratios in the range $P_r = 1.8 - 2.3$, the higher values obtained for the larger of the two sizes tested. It is desirable to continue testing of these and other plate anchors, since the database is rather thin, although many tests are good.

One parameter of particular importance for assessment (and verification) of the pullout resistance is the ultimate depth of penetration z_{ult} as indicated in Figure 9. This depth has a direct bearing on the penetration trajectory assumed for the anchor and thus on the undrained shear strength that will be assumed in the back-calculation of the pullout resistance resulting from the simple design equation in Eq. (9). The tests carried out so far are not conclusive in this respect. It would be of particular interest to carry out a few well-controlled and instrumented offshore tests with anchors small enough to be installed to their ultimate resistance R_{ult} with the vessel(s) that can be made available for such tests.

Immediately after installation and triggering of the anchor, the design procedure assumes that the anchor has a pullout resistance, which is equal to the installation pullout resistance R_{pi} . This resistance is then corrected for the loading rate effect as discussed above to obtain the static pullout resistance R_S . At this point the cyclic loading effects are calculated and added to R_S as discussed in Section 5.4.2.

The cyclic loading effect consists of two parts, one is the *loading rate* (or speed) effect, and the other is the *cyclic degradation* effect on the undrained shear strength of the clay. These two effects are linked together and may be expressed through the cyclic shear strength $\tau_{f,cy}$, see Section 5.4.2 for details.

As mentioned before the effects of *consolidation* should normally be disregarded in the assessment of the pullout resistance of drag-in plate anchors, but it should be mentioned that there are long term effects, which may lead to an increase in the pullout resistance. At this stage the basis for prediction of such long-term effects is, however, insufficient.

The anchor should be installed by continuous pulling until the target installation load T_{dip} has been reached. Stoppage of the installation at a smaller load is not permitted, since there is a risk



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that the remoulded clay around the anchor regain its strength during the stoppage period, which may lead to sufficient increase in the penetration resistance to reach the target installation load without further penetration. Of course, this will not lead to the correct conclusions with respect to the installation pullout resistance R_{pi} , but rather to a situation where the real safety of the anchors is less than reflected by the measurements. Measures should be taken to avoid this situation in the planning and execution of the anchor installation.

The basis for calculation of the effects of consolidation, cyclic loading and uplift at the seabed are discussed in Sections 5.3 through 5.6, respectively, and the complete design procedure is presented step-by-step in Section 5.7. Tentative safety requirements are given in Section 5.8. Since there is a close relationship between the actual anchor installation load and the resulting design anchor resistance, the design procedure integrates these items through an iterative process. The assessment of the minimum installation load resulting from this process is addressed in Section 5.8.2. Finally the requirements to soil investigation are given in Section 5.10.

5.3 Consolidation effects

During continuous penetration of the anchor, the friction resistance will be governed by the remoulded shear strength, s_{ur} , in a narrow zone close to the anchor. In an analytical model this may be accounted for through the adhesion factor, α , which will depend on the soil sensitivity, S_t , i.e. the ratio between the intact (in situ) undrained shear strength, s_u , and the remoulded undrained shear strength

$$S_t = s_u / s_{ur} \quad (11)$$

The α -value may as a lower limit be set equal to the inverse of the sensitivity

$$\alpha_{min} = 1 / S_t \quad (12)$$

After an anchor has been installed to a certain installation load (and depth), the remoulded shear strength will gradually reconsolidate and regain its intact value. As a result the resistance against further penetration will be increased. This effect is in the literature referred to as soaking, set-up or consolidation of the anchor and anchor line. Since a drag-in plate anchor is considered to have reached its required depth of penetration when measurements show that the prescribed target installation load has been reached, consolidation effects must be avoided. In other words the anchor penetration must continue without stoppage until the target installation load has been reached.

The effect of soil consolidation is that the installation anchor resistance R_{dip} will increase as a function of the time elapsed since the anchor installation was stopped t_{cons} . The maximum effect of soil consolidation depends on the soil sensitivity S_t . For a particular anchor and depth of penetration the increase in penetration resistance may be described through a factor U_{cons} , i.e.

$$U_{cons} = f(t_{cons}, S_t, \text{ and geometry, depth and orientation of the anchor}) \quad (13)$$

This may be expressed as

$$R_{cons} = R_{dip} \cdot U_{cons} = R_{dip} + \Delta R_{cons} \quad (14)$$



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By using the consolidated anchor resistance R_{cons} instead of R_{dip} the installation pullout resistance R_{pi} will be over-predicted by a factor equal to the actual consolidation factor $U_{cons} = R_{cons}/R_{dip}$.

5.4 Cyclic loading effects

5.4.1 Background

In order to understand how cyclic loading may affect the resistance of drag-in plate anchors a parallel may be drawn between piles and drag-in plate anchors. Important work on the effect of loading rate on axial pile capacity has been published by Bea and Audibert /5/, followed by Kraft et al /6/, and later by Briaud and Garland /7/. Fundamental work on the effects of cyclic loading on the undrained shear strength of clay and the cyclic response of gravity base foundations has been published by Andersen and Lauritzen /8/.

Cyclic loading affects the static undrained shear strength (s_u) in *two ways*:

- 1) During a storm, the rise time from mean to peak load may be about 3 - 5 seconds (1/4 of a wave frequency load cycle), as compared to 1 to 2 hours in a static consolidated undrained triaxial test (somewhat less in a direct simple shear test), and this higher *loading rate* leads to an *increase* in the undrained shear strength
- 2) As a result of repeated cyclic loading during a storm, the undrained shear strength will *decrease*, the degradation effect increasing with the overconsolidation ratio (OCR) of the clay.

The following relationship is suggested in /7/ for description of the effect of the loading rate, v , on pile capacity, Q

$$(Q_1/Q_2) = (v_1/v_2)^n \quad (15)$$

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

5.4.2 Application to drag-in plate anchor design

If the rate of loading on the anchor were higher during wave loading than during the installation phase, the resistance of the anchor would increase as a function of the relative increase in rate of loading, see Eq. (15). A loading rate factor U_r , equal to the relationship between pile capacity and loading rate in Eq. (15) may be introduced, which expresses the loading rate effect on the anchor resistance, i.e.

$$U_r = (v_1/v_2)^n \quad (16)$$

One practical problem with Eq. (16) 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 . In addition, Eq. (16) does not account for the strength degradation due to cyclic loading. Based on the results from high quality onshore instrumented drag-in plate anchor tests at Onsøy in Norway /13/ a relationship according to Eq. (16) was established for the actual test conditions. It was shown that a static pullout resistance R_S could be defined, which is linked to a reference strain rate v_{ref} (in % per hour) comparable to that used in a static triaxial compression test giving the undrained shear strength $s_{u,C}$. By setting $v_2 = v_{ref}$ in Eq. (16) and using an exponent $n = 0.50$ it was possible to back-calculate R_S from the pullout rates used in the tests. The n -value was obtained by combining results from triaxial tests and anchor tests performed at different strain rates. In the



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lack of other similar anchor tests the experience from /13/ has been used as a reference for assessment of R_S in the procedure outlined in Section 5.7.2. It should be borne in mind, however, that the value of the exponent n varies with the characteristics of the clay, e.g. with the plasticity index I_p . The clay in the referenced onshore tests had a plasticity index $I_p = 25-30$ and a sensitivity $S_t = 6-10$.

The most direct approach to account for both the loading rate effect and the cyclic loading effect is to determine the cyclic shear strength $\tau_{f,cy}$ of the clay, following the strain accumulation procedure described in /8/. $\tau_{f,cy}$ is the preferred characteristic soil strength for use in the design of drag-in plate anchors. As stated above the undrained shear strength $s_{u,D}$ from a DSS test is considered to account reasonably well for the strength anisotropy effects, and is the preferred strength for use in the procedure. (In the design equations this strength is expressed without the subscript D , simply s_u .)

The strain accumulation method utilises so-called strain-contour diagrams to describe the response of clay to various types, intensities and duration of cyclic loading:

- Given a clay specimen with a certain s_u and OCR , which is subjected to a load history defined in terms of a sea state and a storm duration, the intensity of that storm is gradually increased until calculations according to the strain accumulation method show that the soil fails in cyclic loading.
- In a catenary 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 (stress reversal). 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 $\tau_{f,cy}$.
- The load history for use in the calculations should account for the combination of wave-frequency load cycles superimposed on low-frequency, slowly varying, load cycles, particularly the amplitude of cyclic loads relative to the average (or mean) load level.

If cyclic soil data, applicable for the actual site, are available, the cyclic strength $\tau_{f,cy}$ may be determined according to the procedure outlined in /8/. The cyclic strength $\tau_{f,cy}$ as defined in /8/ incorporates effects of both loading rate and cyclic degradation, provided that the cyclic load period is representative for the variation in line tension with time at the anchoring point. This would lead to a combined loading rate and cyclic degradation factor, or simply a cyclic loading factor U_{cy} as shown in Eq. (17) below.

$$U_{cy} = (\tau_{f,cy}/s_{u(REF)}) = f [t_{REF}/t_{cy}, \text{ soil data, load history, etc}] \quad (17)$$

where

$\tau_{f,cy}$ = cyclic shear strength with time to failure $t_{cy} = (1/4) \cdot (\text{load period})$

$s_{u(REF)}$ = reference undrained shear strength based on time to failure ($t_{REF} = 1$ hour)

Setting $s_{u(REF)}$ = the intact undrained shear strength s_u , and $s_u \sim R_S$ the following expression for the contribution due to cyclic loading ΔR_{cy} to the pullout resistance of a drag-in plate anchor is obtained



$$\Delta R_{cy} = R_s \cdot (U_{cy} - 1) \quad (18)$$

If no relevant cyclic soil data exist for the site, and experience from better documented sites with similar soil conditions cannot be drawn upon, a conservative assessment of $\tau_{f,cy}$ may be made based on Eq. (16) corrected for the effect of cyclic strength degradation. In order to account for the possible strength degradation due to one-way cyclic loading, the resulting loading rate factor from Eq. (16) should therefore be multiplied by a cyclic degradation factor k_c . The expression for U_{cy} then becomes:

$$U_{cy} = 1 + k_c \cdot (U_r - 1) = 1 + k_c \cdot \{(v_1/v_2)^n - 1\} \quad (19)$$

k_c is a function of the line tension load history through a storm and the characteristics of the clay. The load history varies with water depth, type of rig and mooring line configuration. Therefore the value of k_c should be assessed from case to case. As experience with calculation of the cyclic shear strength will accumulate with time it will also be possible to give more precise recommendations for assessment of the cyclic degradation factor k_c .

5.5 Creep versus loading (or strain) rate

Anchors for deepwater mooring in taut mooring system will be subjected to significant permanent (and mean) line tension due to pre-tensioning and mean tension during severe weather conditions. This makes anchor creep a design issue, which needs to be addressed. It should, however, be mentioned that for a plate being embedded to some 20 to 30 m depth, creep should not represent a serious threat to the safety of the mooring system, if the anchors are design to satisfy the ULS and ALS requirements according to this procedure.

In the following, an approach for assessment of a threshold line tension accounting for the strain rate, the operational period (lifetime) of the floater and the accumulated duration of various sea states (sustained loading) during the lifetime is presented. The experience from triaxial laboratory tests carried out at different strain rates combined with the results from onshore drag-in plate anchor tests at Onsøy, as reported in /13/, have been used.

The majority of the anchor tests at Onsøy were performed at the same speed, being somewhat lower than the offshore loading rate associated with storm loading, but certainly above the reference speed for a static test in the laboratory. Since loading speed was found to have a noticeable impact on the penetration resistance, an effect of this could also be expected for the pullout test. The test equipment did not allow for running the tests at a set speed, but in one of the tests the speed was reduced significantly (test 12-S-4). Comparing this test with the previous one in the same trench and with the same anchor (12-S-3), one might expect that the deviation in bearing capacity factor is due to speed alone since both tests were performed beyond the depth for maximum bearing capacity factor. The calculated bearing capacity factor in the two tests are 9.06 and 9.91, for the 12-S-4 and 12-S-3, respectively. The deviation in loading rate (or strain rate) for the two tests is a factor of 6.05, giving an increase of 9.4%. Compared to increase pr. log-cycle this represent 12%.

The effect of strain rate on the Onsøy clay has not been investigated, but it has been possible to establish a relationship between the results from the anchor tests and the results from extensive laboratory tests on Drammen clay and Troll clay. The following approach was used:



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- 1) Find results from anisotropically consolidated undrained compression (CAU) triaxial tests on clays with an overconsolidation ratio $OCR = 1$.
- 2) Relate the static undrained shear strength $s_{u,C}$ to the applied strain rate (in % per hour).
- 3) Find for comparison the peak cyclic shear stress in the first cycle ($N_{eq} = 1$), which gives a permanent axial strain of 10 % (failure strain), using results from a test with 1-way cyclic loading. With a cycle period of 10 seconds, this gives a strain rate of 14,400 %/hour for comparison with 3 %/hour in the static test (failure strain set to 3 %, reached after one hour).
- 4) Compare the measured relationship between anchor pullout resistance and strain rate in the Onsøy tests with the behaviour observed in the triaxial tests.

The result based on this approach is shown in Figure 10.

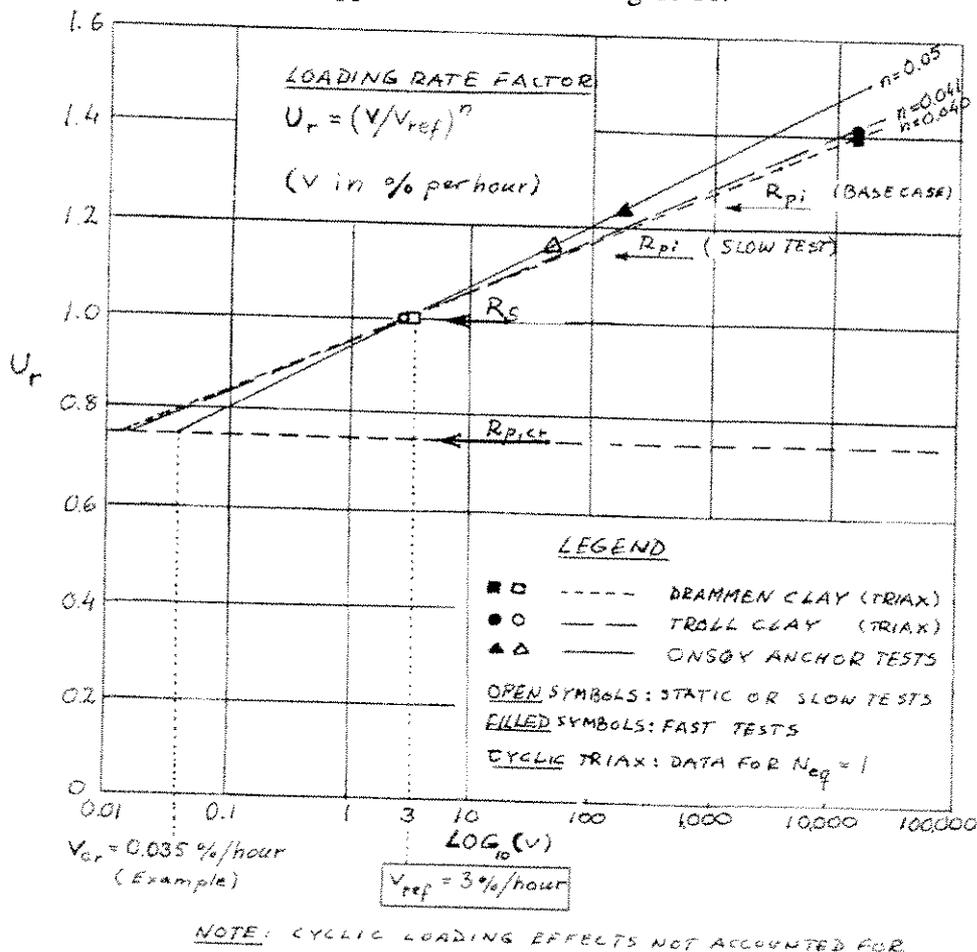
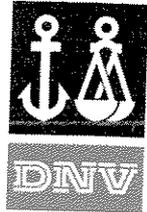


Figure 10 Loading rate factor versus strain rate v , with $v_{ref} = 3\%/\text{hour}$.

As can be seen an increase in the strain rate by one log-cycle increases the shearing resistance by 9.6 % for the Drammen clay, which corresponds to a loading rate factor $U_r = 1.096$ referred to the static shear strength. Since test data suggest that the effect of an increase in strain rate on the static undrained shear strength is nearly linear, it is convenient to express it in terms of a strain rate (or loading rate) factor



$$U_r = (v / v_{ref})^n$$

(20)

where

v = actual strain rate (%/hour)

v_{ref} = reference strain rate, set to 3 %/hour

n = exponent, which is dependent on type of soil and method of testing

In the example in Figure 10, $n = 0.040$ was found for the Drammen clay and $n = 0.041$ for the Troll clay. Combining the results from the anchor tests at Onsøy with the criterion that the line must intersect the static resistance line at a strain rate level of 3 %/hour, an exponent $n = 0.05$ is found for the anchor tests, assuming the failure strain to be 5 %.

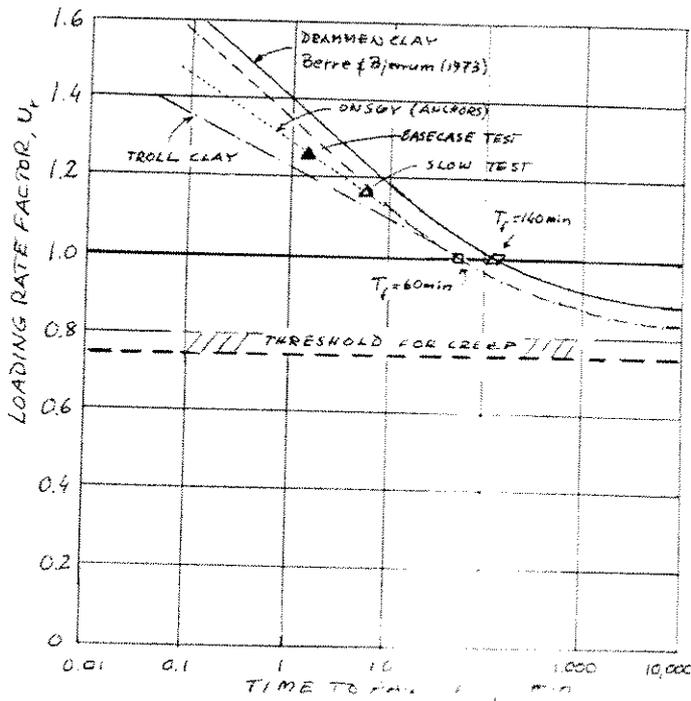
This means that the base case strain rate gives an *installation* pullout resistance R_{pi} , which is 25 % higher than the *static* pullout resistance R_S referred to a strain rate of 3 %/hour. Alternatively, it may be said that the pullout test was run at a strain (loading) rate, which was 62 times higher than the rate corresponding to a static test.

If it can be assumed that this observation is representative for all tests, the measured values of R_{pi} need to be multiplied by a factor 0.80 to get the static pullout resistance R_S . Assuming further that the lines can be extrapolated downwards towards strain rates less than 3 %/hour, a basis may be obtained for assessment of the threshold strain rate level, which would give only negligible creep of the anchor under the sustained load associated with this strain rate.

An idea about how far down in strain rate one needs to go in order to reach a threshold value may be found by presenting the results as a function of time to failure as shown in Figure 11. In a comprehensive paper by Berre and Bjerrum /14/, the experience from tests on Drammen clay was presented, and a curve from that paper has been included in Figure 11. The curve has been corrected roughly to fit a time to failure of $T_f = 60$ minutes instead of 140 minutes as used in /14/. It may also be seen that the Troll data fall much below the Drammen clay data, which differ from the good agreement shown in Figure 10 between Drammen clay and Troll clay. Using the times to failure for the two anchor tests and the loading rate factor derived in Figure 10 a straight-line slope representing the anchor tests has been plotted in Figure 11. The curved shape for $T_f > 60$ minutes is roughly taken from the Berre and Bjerrum curve. It appears that the threshold strain rate level, at which creep might start to become important for the design is where the sustained line tension exceeds a load of 0.75 times the static pullout resistance. Looking again at Figure 10, this would correspond to a strain rate of about 0.035 %/hour.



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NOTE: • REFERENCE TIME TO FAILURE $(T_f)_{ref}$ WAS SET TO 140 min. (Berre & Bjerrum (1973))
 • $(T_f)_{ref} = 60$ min. (slow test)

Figure 11 Loading rate factor U_L versus time to failure T_f , with $(T_f)_{ref} = 60$ minutes.

The above approach for assessment of a threshold strain rate level and a comparable creep pullout resistance $R_{p,cr}$ may be followed up by using the strain accumulation method to determine the cyclic pullout resistance $R_{p,cy}$. To do this the triaxial or DSS cyclic test data are used but the anchor pullout resistance, with reference to Figure 12, replaces the undrained shear strength.

$$\begin{aligned}
 s_{u,D} &= R_S &= \beta \cdot R \\
 \tau_{crit} &= R_{p,cr} &= \rho \cdot K \\
 \tau_{f,cy} &= R_{p,cy} &= U_{cy} \cdot R_S \quad (=R_C, \text{ since consolidation effects are disregarded})
 \end{aligned}$$

The factors β and ρ need to be established on a case by case basis until sufficient understanding of these relationships has been developed in the industry. The cyclic loading factor U_{cy} is computed according to the procedure outlined in Section 5.4.2, but with R_S replacing $s_{u,D}$ and $R_{p,cy}$ replacing $\tau_{f,cy}$.

Currently $\beta=0.80$ is suggested as a default value for determination of R_S from the pullout resistance measured under offshore testing conditions, see above.

The creep factor ρ should be determined based on the given operational time of the floater on the actual location, and the intensity and duration of the various sea states and the maximum acceptable total creep over the operational lifetime. Currently a default value of $\rho=0.75$ is

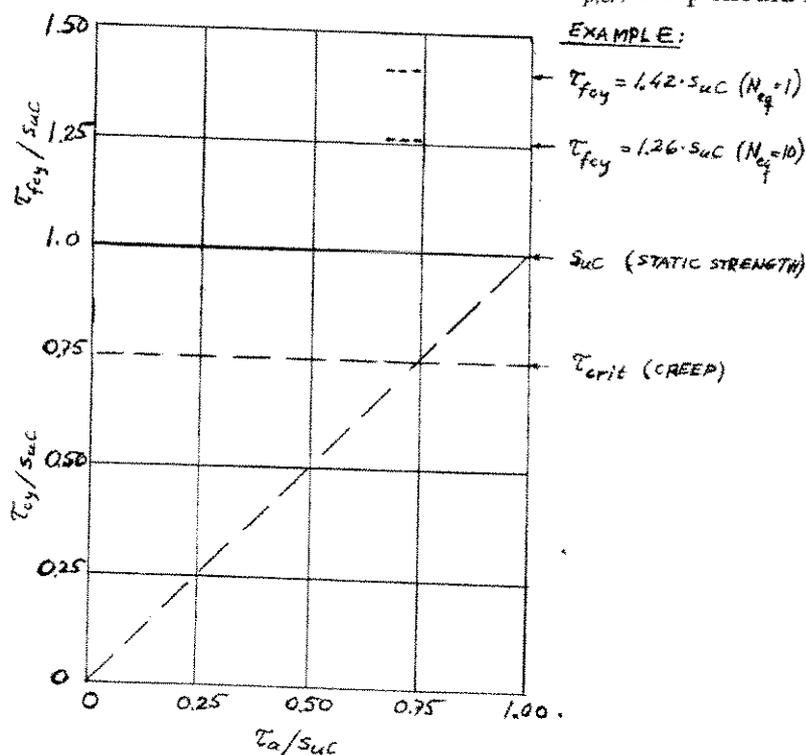


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suggested, but this is dependent on the type of clay and its characteristic properties as well as the duration of the operation.

To calculate the accumulated creep over the operational period the strain rates (in % per hour) associated with the mobilisation level due to a certain storm intensity needs to be estimated based on a plot like that shown in Figure 10. If a relationship giving the marginal distribution of significant wave height for the actual area has been developed, the accumulated duration (in hours) of a wave height of certain amplitude over the operational period can be computed. By multiplying this number with the strain rate determined for that load level, the contribution to creep from that wave height can be computed. By repeating this for a number of wave heights and adding the contribution from each load level, an estimate of the total creep can be obtained. This total creep should be compared with the tolerable creep specified in the actual case.

If the design mean line tension is less than $R_{p,cr}$, creep should not represent a problem.



PARALLEL TO ANCHOR DESIGN:

$$R_S \approx s_{uC} \quad (= \beta \cdot R_{pi})$$

$$R_{p,cr} \approx \tau_{crit} \quad (= \beta \cdot R_S)$$

$$R_C \approx \tau_{fcy} \quad (= R_S \cdot U_{cy})$$

Figure 12 Assessment of the cyclic pullout resistance R_{cy} ($= R_C$).



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5.6 Uplift angle at seabed

It may be cost-effective to install drag-in plate anchors with acceptance of an uplift angle at the seabed. From a calculation point of view it is illustrative to split the anchor installation line into three parts, one part embedded in the soil, a second part resting on the seabed, and a third part suspended in water.

Stretching out the installation line, either by increasing the line tension (bollard pull) or decreasing the distance between anchor and installation vessel (winch operation), will result in a reduction of the seabed part of the anchor line and giving less curvature to the embedded and suspended parts. At some point the length of the seabed part becomes zero ($L_s = 0$), and a further increase in load or decrease in distance will result in a situation where the anchor line intersects the seabed under an uplift angle (α), see Figure 8. The target installation load T_{dip} should then ensure that the installation anchor resistance R_{dip} (without consolidation effect included) is reached.

There will be a potential for significant cost savings if drag-in plate anchors can be installed with an uplift angle. In the following, recommendations will be given for how to assess a safe uplift angle in reasonably normally consolidated clay.

Uplift angles during installation typically occur due to an increased bollard pull or indirectly through pull-in of line using a winch.

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 reasonable compromise to avoid initial penetration problems and to minimise the penalty of reduced final penetration, uplift should not be applied before the anchor fluke has reached a depth corresponding to 2.5 fluke lengths. A final uplift angle exceeding 10° should not be expected during installation of a drag-in plate anchor according to this procedure, even if the anchor approaches its ultimate depth. If higher angles are used the effect on the penetration depth should be evaluated and documented.

The penetration path is only slightly affected by controlling the uplift angles according to 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 percent 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 can be taken as negligible. By accepting uplift from a shallower depth both the final uplift angle and the ultimate depth penalty will increase.

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. 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.



5.7 Recommended design procedure.

5.7.1 General

Two alternative procedures may be considered:

- (1) primarily based on geotechnical calculations (see Section 5.7.2)
- (2) primarily based on anchor tests at the actual site (see Section 5.7.3)

5.7.2 Recommended procedure

Drag-in plate anchors should be designed based on geotechnical calculations using a suitable analytical tool, as discussed in Section 4.4, to do the calculations. The recommended procedure is described in this section.

For simplicity, we assume that there is no need to correct for friction between the sea bed and mooring line lying on the sea bed, implying that there will be a positive uplift angle or only a short length of line on the sea bed during the final stage of installation. If this assumption is invalidated, then the procedure should be corrected to include these friction forces, see Eq. (23).

In this situation, it is convenient to relate both the applied line tension T and the anchor resistance R to the dip-down point, where the mooring line enters down into the seabed. The anchor resistance will be dependent on the installation depth of the anchor z , on the soil conditions, and on the extent to which the loading is applied statically or cyclically.

Step (1): Design tension

- (a) The design tension is computed from the characteristic mean tension T_{C-mean} and dynamic tension T_{C-dyn} , with respective partial safety factors $\gamma_{mean}, \gamma_{dyn}$, as described in DNV's POSMOOR rules (including revision from DEEPMOOR project)

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

Step (2): Approximate pullout resistance and initial anchor size

- (a) Assume that the approximate anchor resistance R_A at the installation depth z_i can initially be set equal to

$$R_A(z_i) = k_A \cdot T_d$$

where the factor k_A is an approximation of the safety factor on the pullout resistance. For the initial sizing of the anchor k_A may be set equal to 1.3. Then the approximate anchor resistance at the ultimate depth z_{ult} can be obtained as

$$R_A(z_{ult}) = R_A(z_i) / M$$

where the mobilisation factor M indicates the proportion of the ultimate resistance that is intended to be utilised. M should preferably be in the range 0.4-0.8, and a value of 0.75 is a reasonable choice, which leaves some margin for the installation process.



- (b) Select a suitable type of drag-in-plate anchor and obtain an initial approximation for the fluke area A_{fluke} of the anchor assuming that it is installed to the ultimate depth z_{ult} and capable of providing the ultimate pullout resistance

$$A_{fluke} = \frac{R_A(z_{ult})}{N_c \cdot s_c \cdot \eta \cdot s_u(z_{ult})}$$

where N_c is the bearing capacity factor (theoretically $N_c = 12.5$ for an infinitely long plate), s_c is the shape factor (typically $s_c = 1.1$ for the drag-in plate anchors currently used), η is an empirical correction factor (default value $\eta = 0.73$), and $s_u(z_{ult})$ is the static, undrained, DSS shear strength of the soil at the ultimate depth z_{ult} .

- The ultimate depth z_{ult} is typically expressed in number of fluke lengths, $z_{ult} = \lambda \cdot F_L$, where the factor λ may range from 6 to 12 for anchors in normally consolidated clays, increasing with decreasing strength of the clay. The λ -factor should not be set higher than 8, unless higher values can be documented for the actual site. For simplicity it is recommended that the fluke length is set equal to the square root of the fluke (plate) area times an anchor specific factor κ , i.e. $F_L = \kappa \cdot \sqrt{A_{fluke}}$. As an average κ may be set equal to 1.25.
- The factor η accounts for the violation of the ideal conditions assumed in the theoretical assumptions, i.e gradual strain softening of the clay when the plate is loaded to failure, remoulding of the clay caused by the anchor installation and subsequent anchor rotation to prepare for hook-up. It also accounts for the effect of possible inclined/excentric loading on the anchor due to inaccurate orientation of the anchor relative to the direction of acting line tension (this is a reality both during anchor testing and operational conditions). To correct for the latter effect the anchor has to rotate further in the soil, which can be expected to occur only during extreme weather conditions. This would then lead to an increased resistance against pullout. If the operational period of the mooring system is long enough for the clay to reconsolidate completely a further gain in pullout resistance may be expected, but at present no allowance for such time dependent increase in the pullout resistance is accounted for in the procedure.

Step (3): Penetration trajectory

- (a) The penetration trajectory for the anchor is computed to the ultimate depth z_{ult} using a suitable program based on geotechnical principles. This gives the installation tension $T_{dip}(z_i)$ and the drag length $x(z_i)$ as a function of the depth to which the anchor is installed z_i .
- (b) The variation of the static intact and remoulded soil shear strength with depth is taken into account. The trajectory is also dependent on the anchor geometry, the forerunner of the mooring line, and the uplift angles applied under installation.

Step (4): Design resistance

- (a) The static pullout resistance $R_S(z_i)$ should be computed for a number of points along the trajectory, using the same program as in Step (3). In the assessment of the approximate pullout resistance $R_A(z_i)$ in Step (2) it has been accounted for the fact that part of the line tension is dynamic, see Step (1), which implies that $R_A(z_i)$ implicitly incorporates a cyclic



loading effect. In order to quantify the cyclic loading effect it is practical to split the anchor pullout resistance into two components, one static resistance and one cyclic loading component. The static component at depth z_i is set equal to

$$R_S(z_i) = \beta \cdot [N_c \cdot s_c \cdot \eta \cdot s_u(z_i) \cdot A_{fluke}]$$

The factor β is a correction of $s_u(z_i)$ for strain rate when comparing the pullout speed normally adopted in offshore anchor tests with the 'would be' speed in a static pullout test. This 'would be' static pullout speed is estimated based on results from triaxial tests run at different strain rates, see discussion in Section 5.5. Tentatively, a value of $\beta = 0.8$ is recommended as a default value for drag-in plate anchors in soft clay.

The undrained shear strength in normally consolidated clay is often seen to increase linearly with depth, i.e.

$$s_u(z_i) = k \cdot z_i = M \cdot s_u(z_{ult}) = M \cdot k \cdot \lambda \cdot \kappa \cdot \sqrt{A_{fluke}}$$

which gives the following expression for $R_S(z_i)$

$$R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}}$$

- (b) The contribution due to cyclic loading ΔR_{cy} is computed as described in Section 5.4.2. The design resistance is then given by

$$R(z_i) = \frac{R_S(z_i)}{\gamma_{m,1}} + \frac{\Delta R_{cy}(z_i)}{\gamma_{m,2}} = R_S(z_i) \left[\frac{1}{\gamma_{m,1}} + \left(\frac{U_{cy} - 1}{\gamma_{m,2}} \right) \right]$$

Step (5): ULS check

- (a) The required installation depth z_i is then determined such that the ultimate limit state is satisfied

$$R_d(z_i) \geq T_d$$

- (b) The depth z_i should preferably be between 40 and 80% of the ultimate depth z_{ult} from step (3) and (4), to leave a margin for the installation process. In a normally consolidated clay with the shear strength increasing linearly with depth this mobilisation factor is equal to M from step (2), which as a default value may be set to 0.75. If the ULS requirement cannot be satisfied, then return to Step (1) and select another mooring pattern, or to step (3) and select another anchor.

Step (6): Determine required installation tension

- (a) Determine the required installation tension $T_{dip}(z_i)$ from the trajectory in step (3), for the selected installation depth. The computed anchor performance ratio P_r at the installation depth is then

$$P_r = R_C(z_i) / T_{dip}(z_i)$$

where $R_C(z_i)$ is the characteristic anchor resistance at the installation depth z_i

$$R_C(z_i) = R_S(z_i) + \Delta R_{cy}(z_i)$$

If a suitable computer is not available for calculation of $T_{dip}(z_i)$, then the performance ratio P_r has to be estimated, either from site specific anchor tests or from tests in similar clay

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formations. $T_{dip}(z_i)$ is the obtained from

$$T_{dip}(z_i) = R_C(z_i)/P_r$$

Step (7): Installation tension check

- (a) Check that the installation tension from step (6) is feasible with respect to the cost and availability of installation equipment. Return to step (1) or (3) if the installation tension is excessive.

Step (8): Check margin against anchor creep

- (a) A check of the margin against anchor creep can be made according to the recommendations in Section 5.5.

Step (9): Estimate anchor drop point

- (a) The anchor drop point is estimated based on drag length computed in step (4) and the selected installation depth.

The iteration process is continued until a suitable anchor is found, while also taking account the combined costs of purchase of equipment, installation, and retrieval.

Note 1. In case of significant layering reference is made to guidance in Section 4.2.2.

Note 2. The acceptable uplift angle during installation of a drag-in plate anchor may be evaluated based on the guidance in Section 5.6.

Note 3. The proposed partial safety factors for design of drag-in plate anchors, see Section 5.8, are tentative until the design rule proposed herein has been calibrated based on reliability analysis.

Note 4. Analytical tools used for prediction of anchor performance during installation and operational conditions should be well documented and validated.

A calculation example following the recommended procedure is included in Appendix A.

5.7.3 Procedure primarily based on anchor tests

If anchor tests are being planned, and the results are intended to become the basis for designing anchors for installation in the same area, the following parameters should be measured.

During anchor installation of the anchor:

- ✓ Line tension (e.g. running line tensiometer and/or instrumented anchor shackle)
- ✓ Fairlead line angle
- ✓ Pull-in speed
- ✓ Pitch and roll of anchor

All the above parameters should be measured versus time. At the end of the anchor installation the following measurements would be particularly useful:



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- ✓ Depth of penetration (final depth)
- ✓ Dragging distance (related to final depth)

Before/during pullout test:

- ✓ Evidence of anchor triggering
- ✓ Line tension (running line tensiometer and/or instrumented anchor shackle)
- ✓ Embedded length of pullout line after triggering, but before reaching the pullout failure load. The failure load may be reached at an anchor displacement of about 1/4 to 1/2 fluke widths (fluke width set equal to $(1/F_L) \cdot \sqrt{A_{\text{fluke}}}$).
- ✓ Pullout speed

High quality anchor tests should continue to be performed, both to provide a basis for design of anchors following the procedure in Section 5.7.2 or the procedure described in Section 5.7.3. This is vital for the further development of the anchor design procedures.

Properly executed and interpreted site specific anchor tests will provide installation load T_i and pullout resistance R_{pi} versus installation depth (z_i), which partly may replace the use of a computer program to predict the penetration trajectory versus line tension, see Step (3) of the recommended procedure in Section 5.7.2. However, before using the anchor test results as a basis for anchor design, corrections for the effect of pullout speed on the pullout resistance need to be made, see discussion in Sections 5.4 and 5.5. The method of extrapolating the measured behaviour of the test anchor to the prototype anchor size, and the assessment of the required installation tension in Step (6) are examples of important design issues that need to be discussed and documented.

The partial safety factors proposed in Section 5.8 are for the recommended design procedure, but they may have to be increased if the anchor design is based primarily on anchor tests, without having a suitable analytical tool for interpretation and extrapolation of the results. The need to increase the partial safety factors should be decided on a case-by-case basis.

5.8 Tentative safety requirements.

5.8.1 General

Safety requirements for use together with the procedure for geotechnical design of drag-in plate anchors as described above are for temporary use until a formal calibration of the partial safety factors has been carried out.

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) and
- the Accidental Limit State (ALS)

For structural design of the anchor also the Fatigue Limit State (FLS) needs to be satisfied.

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} , T_{C-dyn} at that point multiplied by their respective partial safety factors γ_{mean} , γ_{dyn} , i.e.



$$T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn} \quad (21)$$

The characteristic tensions may be computed as specified in DNV's rules for Position Mooring (POSMOOR) (see footnote to tension in Glossary).

The design anchor resistance (R_d) is defined as

$$R_d = R_S / \gamma_{m,1} + \Delta R_{cy} / \gamma_{m,2} \quad (22)$$

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 having a one-line failure.

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 leads to unacceptable creep, or pullout of the anchor, then the anchor has failed to fulfil its intended function. The acceptable creep shall be assessed on a case by case basis. Limited creep of an anchor, during a storm or accumulated over the duration of the operation at the actual location, is normally acceptable for drag-in plate anchors.

The failure criterion for a drag-in plate anchor in its operational, triggered mode, is defined as the event when the design line tension T_d exceeds the design anchor pullout resistance R_d . This is the limit state definition used in the ULS.

Target reliability levels have to be defined as part of a calibration of the design equations and the corresponding partial safety factors have to be evaluated. These levels will be chosen when more experience is available from a detailed reliability analysis.

For calibration and quantification of the partial safety factors for ULS and ALS design, probabilistic analyses will be necessary. Such studies are presently being carried out by DNV through the Deepmoor Project with respect to catenary moorings, which work may be extended to taut moorings and synthetic fibre ropes.

The partial safety factors proposed at this stage are therefore only tentative awaiting a formal calibration of them.

5.8.2 Partial Safety Factor for Anchor Resistance in ULS Case

With an intact mooring system, the anchors are designed to avoid the development of failure displacements during a storm or accumulated during the period of operation at the actual location, and the following material factors are tentatively suggested:

Partial safety factors on anchor pullout resistance for ULS		
Type of analysis of wave-frequency motion	On R_S , $\gamma_{m,1}$	On ΔR_{cy} , $\gamma_{m,2}$
Dynamic	1.2	1.5
Quasi-static	1.2	1.5



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The material factor γ_m shall account also for the uncertainty in the calculation method and analysis model, and the intact undrained shear strength, as far as it affects the calculation of the mentioned contributions to R_C .

The material factors are intended for use with anchor resistance calculated by geotechnical analysis as described in Section 5.7.

5.8.3 Partial Safety Factor for Anchor Resistance in ALS Case

Two consequence classes are introduced in the ALS, where mooring system failure:

- (1) 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) may well lead to unacceptable consequences of these types.

An initial mooring line failure in severe weather is expected to lead to a more uneven distribution of line tensions in the remaining lines. The definition of the failure event for the ULS case is retained for consequence class 2 of the ALS case. For consequence class 1, failure is defined as an anchor displacement, which would have a noticeable, but tolerable, effect on the load distribution between the lines. Detailed analysis of the ALS has not been carried out yet, but some reduction of the material factor applied to the ULS seems appropriate for consequence class 1. The following factors are tentatively suggested when the same characteristic anchor resistance is used as in the ULS:

Partial safety factors on anchor pullout resistance for ALS		
Consequence Class	On R_S , $\gamma_{m,1}$	On ΔR_{cy} , $\gamma_{m,2}$
1	1.0	1.15
2	1.0	1.3

The maximum acceptable displacement of the anchor during the period in which consequence class 1 conditions prevail needs to be estimated, and the consequences assessed and accepted from an anchor load-displacement behaviour point of view during the actual period.

5.8.4 Partial Safety Factors on Line Tension in ULS Case

For the ULS case; i.e. anchors in an *intact system*, the following partial safety factors are tentatively suggested, based on DNV's rules for Position Mooring (see footnote to tension in Glossary):

Partial safety factors on line tension for ULS		
Type of analysis of wave-frequency motion	Partial safety factor on mean tension γ_{mean}	Partial safety factor on dynamic tension γ_{dyn}
Dynamic	1.20	1.60
Quasi-static	1.80	1.80



5.8.5 Partial Safety Factors on Line Tension in ALS Case

For the ALS case, i.e. when one mooring line has failed, the following tentative partial safety factors are tentatively suggested, based on DNV's rules for Position Mooring (see footnote to tension in Glossary):

Partial safety factors on line tension for ALS			
Consequence Class	Type of analysis of wave-frequency motion	Partial safety factor on mean tension γ_{mean}	Partial safety factor on dynamic tension γ_{dyn}
1	Dynamic	1.00	1.25
2	Dynamic	1.30	1.80
1	Quasi-static	1.35	1.35
2	Quasi-static	1.90	1.90

5.9 Minimum installation load.

The prescribed minimum installation load T_{touch} will to a great extent determine the geotechnical safety of the anchor as installed. T_{touch} may be assessed from Eq. (23) below, which accounts for the possible length of line lying on the seabed during anchor installation, i.e. for the condition of *no uplift* during the anchor installation phase.

$$T_{touch} = T_{dip} + \mu \cdot W' \cdot L_s \cdot \gamma_{m,i} \quad (23)$$

where $\gamma_{m,i}$ is the partial safety factor for seabed anchor line sliding resistance over length L_s during anchor installation. Tentatively this material factor is set to

$$\gamma_{m,i} = 1.3$$

The material factor $\gamma_{m,i}$ shall account for the uncertainty in the predicted sliding resistance, which has to be overcome to reach the target installation load in the dip-down point. The friction coefficient μ will vary with the seabed soil composition and the type of forerunner, chain or wire. For deepwater installations it is, however, likely that the drag-in plate anchors are installed with an uplift angle at the seabed, in which case the horizontal component of the line tension in the dip-down point T_{dip} is the target installation load to be documented by measurements.

If the anchor can be installed under an *uplift* angle, the length of line on the sea bed will be set to zero (i.e. $L_s = 0$), which changes Eq. (23) to

$$T_{touch} = T_{dip} \quad (24)$$

In practice, T_{touch} will have to be calculated through an iterative process following the step-by-step procedure outlined in Section 5.7. The resulting T_{touch} will then be evaluated and compared with the installation loads, which can be achieved with the installation scenarios under considerations, see sketch of anchor installation in Figure 13.

The bollard pull of the most powerful new generation anchor handling vessels (AHV) is in the range 2 to 2.5 MN. Depending on the required minimum installation load T_{touch} at the *touchdown*



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point, one or two AHV's may be required. As an alternative to using 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 tension forces up to 5 to 6 MN or even 10 MN can be reached.

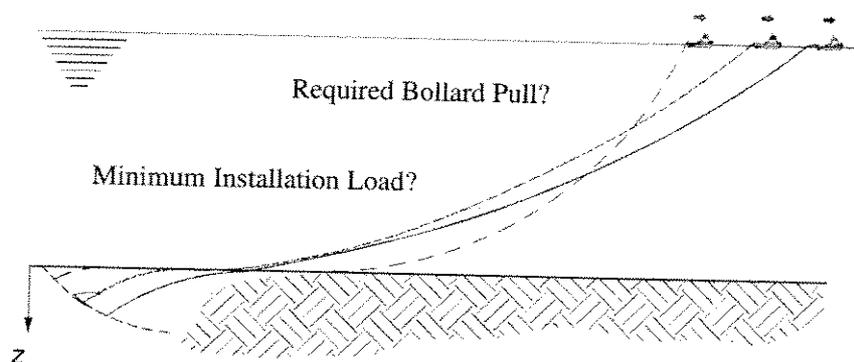


Figure 13 Minimum installation load vs. required bollard pull.

The bollard pull and winch capacity set the limit for the anchor installation load, which again sets a limit on the design anchor resistance in Eq. (22). It is important that all parties involved in the decisions related to the anchor design appreciate the relationship between the anchor resistance and installation load. In deep waters, unless lightweight anchor lines are used, the weight and sea bed friction of the anchor lines limits the net tension force that can be used for anchor penetration, which must be considered when the requirements for the installation vessel are specified.

Since the anchor target installation load T_{dip} sets the limit for the design resistance R_d of the anchors, and consequently determines the number of anchors required, it is important to assess at a relatively early stage the minimum installation load T_{touch} . This load should be held for a specified period of time, in clay normally 20 minutes. 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 should follow procedures, which have been presented and agreed to by all parties well ahead of the installation. Monitoring of the anchor installation load should, as a minimum, provide data on line tension, anchor drag, length of anchor line lying on the seabed and (if practical) line angle at the fairlead, all data presented as a function of time. With ROV assistance during anchor installation it should also be possible to check the position and orientation of the anchor, as well as the alignment and straightness of the as laid anchor line, before start of tensioning. Significant misalignment of the installation anchor line will of course require extra pulling force to reach the specified target installation load T_{dip} . If practical, procedures should also be established for measuring the (final) anchor penetration below seabed.



5.10 Requirements to soil investigation

5.10.1 General

The planning and execution of soil investigations for design of drag-in plate anchors should follow established and recognised offshore industry practice. As a general guidance to achieve this quality of soil investigation reference is made to the NORSOK standard /9/, which makes extensive references to international standards. Some specific recommendations are given herein for soil investigations for drag-in plate anchors.

For design of drag-in plate 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 drag-in plate anchors in clay are the intact undrained shear strength (s_u), the remoulded undrained shear strength (s_{ur}), the clay sensitivity (S_t), the coefficient of compressibility (c_v), and the cyclic shear strength ($\tau_{f,cy}$) for each layer of significance, see Chapter 3 for definition of the respective parameters.

As a minimum, the soil investigation should provide the basis for specification of a representative soil profile and the undrained shear strengths (s_u and s_{ur}) 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.

The ultimate depth of penetration of drag-in plate anchors in clay varies with the size of the anchor and the undrained 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 widths. In very soft clay the *ultimate* penetration may be up to 10-12 fluke widths decreasing to only 1-2 fluke widths in strong, overconsolidated clays. Drag-in plate anchors are primarily of interest for soft clays, such that a reasonable range of ultimate penetrations to consider would be 6-12 fluke widths. However, an anchor is never (or seldom) designed for full utilisation of the ultimate resistance, because of the associated large drag distance. The necessary depth of a soil investigation in a clay without significant layering will be a function of the size of the anchor and the shear strength of the clay. The upper few metres of the soil profile is of interest particular 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 drag-in plate anchor foundations, in addition to the recommendations in /9/, are provided in the following.

5.10.2 Geophysical surveys

The depth of sub-bottom profiling should correspond to the depth of rock or the expected depth of anchor penetration. The seismic profiles should preferably be tied in to geotechnical borings within the mooring area, which will improve the basis for interpretation of the results from the geophysical survey.



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5.10.3 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 5.10.1.

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, water depth, sea floor bathymetry, etc.

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. For the anchor design, most weight should be given to the undrained shear strength derived from direct simple shear (DSS) and unconsolidated undrained (UU) triaxial tests. These types of test are considered to give the most representative estimates of the *intact* undrained shear strength of the clay. Clay sensitivity (S_r) is also a significant soil parameter in the anchor design, which requires companion determinations (on the same soil specimen) of intact and remoulded shear strengths, either by UU triaxial tests or by fall-cone tests.

The challenge to secure soil samples of sufficient quality to determine realistic strength parameters increases with the water depth, and the continuous efforts to improve the existing, and develop new, sampling procedures should be followed. Nevertheless, in situ testing will become increasingly important for mapping of the soil conditions in deep waters. One possibility to acquire soil information relevant for design of drag-in plate anchors is to perform a few instrumented anchor tests at the actual location. The test anchor should be sized such that it can be penetrated to the ultimate depth with the available installation equipment, and it should provide relationships between line tension (preferably at the anchor shackle), depth and drag versus time. Regarding other considerations related to planning and performance of anchor tests reference is made to Section 5.7.3.

For assessment of the post-installation effect due to soil reconsolidation, the consolidation characteristics of the clay, particularly the coefficient of compressibility (c_v) should be gathered as part of the soil investigation.

For calculation of the effect of cyclic loading on the long-term anchor resistance, it is recommended to carry out static and cyclic undrained DSS tests. These tests should be carried out on representative soil samples of high quality, which shall be subjected to stress conditions, which 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 load conditions to be covered by the anchor design, e.g. cyclic tests with a representative combination of average and cyclic shear stresses. The test programme should allow the construction of a strain contour diagram, as required for calculation of the cyclic shear strength ($\tau_{f,cy}$), see /8/ and Chapter 3 for details. If site specific soil data are not provided for assessment of the cyclic loading effect, a conservative assessment of this effect is warranted.



6 REFERENCES

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APPENDIX

A

CALCULATION EXAMPLE



Calculation example:

Design line tension:

1. $T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn}$
2. $T_{C-dyn} = \xi \cdot T_{C-mean}$

Assess the approximate fluke area A_{fluke} :

3. $R_A(z_i) = k_A \cdot T_d$ (Assume $k_A = 1.3$)
4. $R_A(z_{ult}) = R_A(z_i)/M$
5. $A_{fluke} = R_A(z_{ult})/[N_c \cdot s_c \cdot \eta \cdot s_u(z_{ult})]$
6. $z_{ult} = \lambda \cdot F_L$ ($\lambda \leq 8$, unless site specific evidence for use of higher value)
7. $F_L = \kappa \cdot \sqrt{A_{fluke}}$ (approximation, $\kappa=1.25$)
8. $s_u(z) = k \cdot z$

Computer-based parametric study of tension-anchor-soil interaction;

9. $T_{dip}(z)$ for $z = 0 \rightarrow z_{ult}$

Actual design calculations:

10. $R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot \beta \cdot s_u(z_i) \cdot A_{fluke} = N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}}$
11. $\Delta R_{cy}(z_i) = (U_{cy} - 1) \cdot R_S(z_i)$
12. U_{cy} = cyclic loading factor (from cyclic loading analysis)
13. $P_r = R_C(z_i)/T_{dip}(z_i)$
14. $R_C(z_i) = R_S(z_i) + \Delta R_{cy}(z_i)$
15. $R_d(z_i) = R_S(z_i)/\gamma_{m,1} + \Delta R_{cy}(z_i)/\gamma_{m,2}$
 $= (N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}})/\gamma_{m,1}$
 $+ [(U_{cy}-1) \cdot N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}}]/\gamma_{m,2}$
 $= (N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}}) \cdot [(1/\gamma_{m,1}) + (U_{cy}-1)/\gamma_{m,2}]$
16. Check if $R_d(z_i) \geq T_d$
 $(N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot A_{fluke} \cdot \sqrt{A_{fluke}}) \cdot [(1/\gamma_{m,1}) + (U_{cy}-1)/\gamma_{m,2}] \geq T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn}$
 $(N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa \cdot (A_{fluke})^{3/2} \cdot [(1/\gamma_{m,1}) + (U_{cy}-1)/\gamma_{m,2}]) \geq T_{C-mean} \cdot (\gamma_{mean} + \xi \cdot \gamma_{dyn})$
 $A_{fluke} \geq [T_{C-mean} \cdot (\gamma_{mean} + \xi \cdot \gamma_{dyn}) / (N_c \cdot s_c \cdot \eta \cdot \beta \cdot k \cdot M \cdot \lambda \cdot \kappa) \cdot ((1/\gamma_{m,1}) + (U_{cy}-1)/\gamma_{m,2})]^{2/3}$



17. ULS check:

Set

γ_{mean}	= 1.20
γ_{dyn}	= 1.60
$\gamma_{m,1}$	= 1.20
$\gamma_{m,2}$	= 1.50
M	= 0.75
λ	= 8
κ	= 1.25
N_c	= 12.5
s_c	= 1.1
η	= 0.73
k	= 1.35
β	= 0.8
U_{cy}	= 1.18 (assumed)
T_{C-mean}	= 2,560 kN
ξ	= 0.49

then

$$A_{fluke} \geq [T_{C-mean} \cdot (1.2 + 0.49 \cdot 1.6) / (12.5 \cdot 1.1 \cdot 0.73 \cdot 0.8 \cdot 1.35 \cdot 0.75 \cdot 8 \cdot 1.25) \cdot ((1/1.2) + (0.18/1.5))]^{2/3}$$

$$\geq (T_{C-mean} \cdot 1.984 / 77.51)^{2/3} = (T_{C-mean} \cdot 0.03155)^{2/3} = 16.25 \text{ m}^2$$

18. Further checks:

$$T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn} = 2560 \cdot 1.2 + 0.49 \cdot 2560 \cdot 1.6 = 3072 + 2007 = 5079 \text{ kN}$$

$$A_{fluke} = 16.25 \text{ m}^2 \text{ yields}$$

$$s_u(z_{ult}) = k \cdot \lambda \cdot \kappa \cdot \sqrt{A_{fluke}} \text{ kPa} = 1.35 \cdot 8 \cdot 1.25 \cdot \sqrt{16.25} = 54.42 \text{ kPa}$$

$$R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot \beta \cdot M \cdot s_u(z_{ult}) \cdot A_{fluke} = 5326 \text{ kN}$$

$$\Delta R_{cy}(z_i) = (U_{cy} - 1) \cdot R_S(z_i) = 0.18 \cdot 5326 = 959 \text{ kN}$$

$$R_C(z_i) = R_S(z_i) + \Delta R_{cy}(z_i) = 5326 + 959 = 6285 \text{ kN}$$

$$R_d(z_i) = R_S(z_i) / \gamma_{m,1} + R_C(z_i) / \gamma_{m,2} = 5326 / 1.2 + 6285 / 1.5 = 4439 + 640 = 5079 \text{ kN}$$

$$\geq T_d \text{ (OK!)}$$

19. Special check of anchor creep:

- assessment of creep factor ρ

$$R_{p,cr}(z_i) = \rho \cdot R_S(z_i)$$

See Section 5.5 for guidance



20. Evaluate feasibility of required installation load.

Compute $T_{touch}(z_i) = T_{dip}(z_i) + (\mu \cdot W' \cdot L_s) \cdot \gamma_{m,i}$

With a reliable analytical tool at hand - compute P_r :

$P_r = R_C(z_i) / T_{dip}(z_i)$ [$T_{dip}(z_i)$ from the anchor-soil-tension interaction analysis]

Without an analytical tool - estimate P_r (and T_{dip}):

Assume: $P_r = 2.2$ (including loading rate effects present in an offshore pullout test)

Gives: $T_{dip}(z_i) = R_C(z_i) / P_r = 6285 / 2.2 = 2857 \text{ kN}$

T_{touch} (and T_{dip}) acceptable? If not,

- go back to Step 1 (change mooring pattern), or
- go back to Step 2 (change anchor).

21. Estimate the anchor drop point.

- oOo -