

RECOMMENDED PRACTICE
DNV-RP-E302

DESIGN AND INSTALLATION OF
PLATE ANCHORS IN CLAY

DECEMBER 2002

DET NORSKE VERITAS

FOREWORD

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ACKNOWLEDGEMENTS

This Recommended Practice is based upon a design procedure developed within the Joint Industry Projects "Design Procedures for Deep Water Anchors" /1/, /2/ and "Reliability Analysis of Deepwater Plate Anchors" /3/. The following companies sponsored these JIPs:

BP Exploration Operating Company Ltd. (/1/ and /2/); BP America Production Company (/3/); Statoil ASA; Norske Conoco AS; Petrobras; Norsk Hydro ASA (only /1/ and /2/); Shell International Exploration and Production (only /1/); Det Norske Veritas; Health & Safety Executive; Minerals Management Service; Norwegian Petroleum Directorate (only /3/); Bruce Anchor Ltd. (only /1/ and /2/); and SOFEC Inc. (only /1/).

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THIS REVISION

This revision of RP-E302 is made to honour the significant contributions from the most recent JIP on plate anchors, which has resulted in a more general design code for plate anchors in clay with reliability-based calibration of the partial safety factors. The number of partial safety factors on the anchor resistance has in this new revision been reduced from two to one, which makes the safety format more transparent and practical.

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CONTENTS

1.	General	1
1.1	Introduction	1
1.2	Objective.....	1
1.3	Scope of application	1
1.4	Structure of the RP.....	1
1.5	Abbreviations.....	2
1.6	Symbols and explanation of terms.....	2
2.	Design principles	5
2.1	Limit state method of design	5
2.2	Limit states	5
2.3	Consequence classes	5
3.	Design code for plate anchors	6
3.1	Characteristic and design line tension.....	6
3.2	Characteristic anchor resistance, R_C	6
3.3	Design anchor resistance, R_d	10
4.	Reliability analysis	11
4.2	General.....	11
4.3	Target annual probabilities	11
5.	Installation effects on penetration depth	12
5.1	General.....	12
5.2	Anchor keying and rotation	12
5.3	Verification of as-installed penetration depth	12
6.	Requirements to soil investigation	12
7.	References	13

Appendix A: Characteristics and behaviour of drag-in plate anchors

Appendix B: Examples of drag-in plate anchors

Appendix C: Uplift angle at the seabed

Appendix D: Drag-in plate anchors in layered clay

Appendix E: Installation and testing of drag-in plate anchors

Appendix F: Consolidation effects

Appendix G: Cyclic loading effects

Appendix H: Analysis tool DIGIN

Appendix I: Prediction of target penetration depth

Appendix J: General requirements to soil investigations

1. General

1.1 Introduction

The design code for plate anchors outlined in this Recommended Practice is based on the results from the following two joint industry projects:

- *Design procedures for deep water anchors* /1/ and /2/
- *Reliability analysis of deepwater plate anchors* /3/

The recommendations with respect to analysis of the installation phase of drag-in plate anchors are based on the work on fluke anchors in the first JIP, see RP-E301 /4/. An overview of this JIP is given in /5/.

The main objective with the second JIP /3/ was to calibrate and quantify the partial safety factors of a design code for plate anchors based on results from structural reliability analysis. As a result of this JIP, RP-E302, has been extended to cover all types of plate anchors in clay, not only drag-in plate anchors, following a significant revision of the design code.

Anchor specific installation effects, of importance for the as-installed anchor resistance, are addressed separately and guidance for accounting for such effects is given in Chapter 5 and in Appendix I.

In this revised RP-E302 a distinction is made between drag-in plate anchors and push-in type plate anchors.

- A drag-in plate anchor is installed as a conventional fluke anchor, see /3/, and through an anchor-specific triggering method the anchor is converted to take high vertical loads.
- The driving force required to install a push-in plate anchor may be either gravity, hydraulic or suction. Once the vertically oriented plate has reached the target penetration depth the anchor is rotated into a position that creates a high resistance against pullout.

1.2 Objective

The objective is that this Recommended Practice shall

- give good designs
- be convenient in use
- impose a known reliability level, and
- allow comparison of different designs for consistent reliability

1.3 Scope of application

This Recommended Practice applies to the geotechnical design and installation of plate anchors in clay for taut, semi-taut and catenary mooring systems. The design code is applicable to anchors for both production platforms and drilling platforms. The recommended method for calculating the resistance of a plate anchor is applicable to both drag-in and push-in type plate anchors. To make the description of the design code as concise and transparent as possible, anchor specific text has been moved to the

appendices. Some of the appendices originally written for drag-in plate anchors are, to some extent, also applicable to push-in type plate anchors.

The design code outlined is a recipe for how plate anchors in clay can be designed to satisfy the requirements by DNV.

The partial safety factors presented for use in connection with this design code have been calibrated based on reliability analysis /3/. The scope of the calibration covers plate anchors in normally consolidated and layered clay, subjected to extreme line tensions representative of semisubmersibles and ship operating in a range of water depths and environmental conditions. A general description of the calibration procedure adopted is given in /6/.

Tentative recommendations are given for determination of the target installation depth of plate anchors that are designed to resist vertical loads.

1.4 Structure of the RP

The recommended design code for plate anchors in clay is presented in Chapter 3, and the calculated penetration depth of the anchor following this design code corresponds to the as-installed penetration depth. The target penetration depth has to be set so that the possible loss of penetration depth due to anchor keying and rotation will be accounted for, see guidance in Chapter 5 and Appendix I. The design code assumes that the as-installed penetration depth be verified by measurements.

The intention has been to make the design code presented herein as concise as possible, but still detailed enough to avoid misinterpretation or misuse. For transparency, details related to the various design aspects are therefore given in the appendices.

The main characteristics of drag-in plate anchors are described in Appendix A and brief descriptions of the various types of plate anchors are given in Appendix B.

The uplift angle of the installation line at the seabed for drag-in plate anchors is discussed in Appendix C, and considerations associated with use of plate anchors in layered clay are discussed in Appendix D.

Guidance for installation and testing of drag-in plate anchors are provided in Appendix E, and the effects of consolidation and cyclic loading on plate anchors are discussed in Appendix F and Appendix G, respectively.

Guidance for modelling of the anchor line, the anchor and the soil in connection with analysis of drag-in plate anchors and fluke anchors are presented in Appendix H based on the experience with the DIGIN programme, see /1/, /2/ and /5/.

General requirements to soil investigations are given in Chapter 6 and Appendix J.

A number of **Guidance notes** have been included as an aid in the modelling and analysis of plate anchors.

Symbol	Term	Explanation of term	Symbol	Term	Explanation of term
$(N_c)_{shallow}$	Bearing capacity factor	N_c -factor for the shallow zone, $z_i \leq 4.5W_F$, which accounts for the depth effect	R_{ai}	Sum of soil resistance at anchor components	Excluding soil resistance at the fluke
N_{eqv}	Equivalent number of cycles to failure	The number of cycles at the constant cyclic shear stress that will give the same effect as the actual cyclic load history	R_{FN}	Soil normal resistance	At the fluke
OCR	Overconsolidation ratio	Ratio between maximum past and present effective vertical stress on a soil element	R_{FS}	Soil sliding resistance	At the fluke
PI	Plasticity Index	Difference $w_L - w_P$ (used in Table G-1)	R_{TIP}	Tip resistance	At anchor members
P_r	Performance ratio	$P_r = R_{p,i}(z_{test})/T_i(z_{test})$	Rm_{ai}	Moment contribution	From R_{ai}
q	Normal stress	Used in equilibrium equations for anchor and embedded anchor line	Rm_{FS}	Moment contribution	From R_{FS}
θ	Orientation of anchor line element	$\theta = 0$ for a horizontal element	Rm_{TIP}	Moment contribution	From R_{TIP}
Q_1, Q_2	Pile resistance	Pile resistance at loading rates v_1 and v_2 , respectively	ρ	Creep factor	Related to $R_{p,cr}$
R	Anchor resistance	Resistance in the line direction in the line dip-down point with reference to the anchor penetration depth z	$\sigma_{v,0}'$	Effective overburden stress	In relation to OCR (Eq. (3.14))
R_C	Characteristic anchor pullout resistance	$R_C(z_i) = R_S(z_i) \cdot U_{cy} = R_{cy}(z_i)$	s_c	Shape factor	$= 1 + 0.2(W_F/L_F)$, related to N_c
R_{cy}	Cyclic anchor resistance	$R_{cy}(z_i) = R_S(z_i) \cdot U_{cy} = R_C(z_i)$ Static resistance, superimposed cyclic loading effects	S_t	Soil sensitivity	The ratio between s_u and $s_{u,r}$, as determined e.g. by fall-cone or UU triaxial tests
R_d	Design anchor resistance	$R_d(z_i) = R_C(z_i)/\gamma_m = R_{cy}(z_i)/\gamma_m$	s_u	Intact strength	For plate anchor analysis the DSS strength $s_{u,D}$ (or the UU triaxial strength) is assumed to be representative
ΔR_{cons}	Consolidation effect	At restart after stoppage during anchor installation	$s_{u,D}$	Intact strength	Undrained shear strength from DSS test
$R_{p,cr}$	Creep pullout resistance	Threshold for development of significant creep, which depends on actual line tension history, duration of operation, soil characteristics, etc.	$s_{u,0}$	Intact strength	Seabed undrained shear strength intercept
$R_{p,i}$	Anchor installation pullout resistance	The pullout resistance 'immediately' after anchor installation, triggering and rotation in the dip-down point	$s_{u,r}$	Remoulded shear strength	The undrained shear strength measured e.g. in a fall-cone or a UU triaxial test after having remoulded the clay completely
R_{ult}	Ultimate anchor resistance	The resistance reached at $z = z_{ult}$, the anchor drags without further increase in the resistance during continuous pulling	Δs_u	Change in s_u at $z_i = z_l$	Step change in s_u at layer boundary in a 2-layer profile
			$s_{u,mean}$	Mean undrained shear strength	Accounts for variation in s_u across a layer boundary and within the volume of soil affecting the anchor resistance, depth interval $3W_F$, with centre at $z_i = z_{plate}$
			$s_{u,I}$, $s_{u,III}$	Average undrained shear strength	For spherical volume of soil above and below the depth of the plate (z_{plate}) involved in the calculation of $s_{u,mean}$
			τ_a	Average shear stress	↑ Used in connection with cyclic DSS tests
			$\tau_a/s_{u,D}$	Average shear stress level	↓
			$\tau_{f,cy}$	Cyclic shear strength	Accounts for both loading rate and cyclic degradation effects

Symbol	Term	Explanation of term	Symbol	Term	Explanation of term
τ_{cr}	Creep shear strength	on s_u . From a triaxial test (compare with $R_{p,cr}$)	v_2	Loading (or strain) rate	Reference rate at the end of installation
t_{cy}	Time to failure	Rise time of line tension from mean to peak level during the design storm (= 1/4 load cycle period)	v_{ref}	Loading (or strain) rate	Reference rate for pullout resistance
t_{hold}	Installation tension holding period	Period of holding T_{min} at the end of anchor installation	W_F	Equivalent plate width	Equal to $\kappa^*(A_{plate})^{0.5}$, ($\kappa \leq 1$, anchor plate specific)
t_{su}	Time to failure	Time to failure in a laboratory test for determination of the intact undrained shear strength (typically 0.5 – 2 hours)	W'	Submerged anchor weight	Taken as $0.87 \cdot$ anchor weight in air W
T	Line tension	From mooring analysis	Wm	Moment contribution	From anchor weight W'
T_v, T_h	Components of line tension at the shackle	Vertical and horizontal component of the line tension at the anchor shackle for the actual anchor and forerunner	W_l'	Submerged weight of anchor line	Per unit length of actual line segment
T_C	Characteristic line tension	Split into a mean and dynamic component, T_{C-mean} and T_{C-dyn} following recipe in /7/	w_p, w_L	Plasticity limits	Lower and upper limit, respectively, of water content w for calculation of plasticity index PI .
T_{C-mean}	Characteristic mean line tension	Due to pretension and the effect of mean environmental loads in the environmental state	ψ	Inverse loading rate factor	Adjustment for loading rate effect in the pullout resistance $R_{p,i}$ measured in an anchor test, $R_S = \psi \cdot R_{p,i}$ (with $\psi=1/U_r$)
T_{C-dyn}	Characteristic dynamic line tension	The increase in tension due to oscillatory low-frequency and wave-frequency effects	z	Penetration depth	Depth below seabed of the plate centre of area.
T_d	Design line tension	With specified partial safety factors included	z_{calc}	Calculated penetration depth	$= z_{plate}$
T_{d-mean}	Design mean line tension	$= T_{C-mean} \cdot \gamma_{mean}$	z_i	Installation penetration depth	At end of penetration, and after rotation of anchor into position for normal loading, z_i refers to centre of plate area ($= z_{plate}$).
T_f	Time to failure	Related to creep (Figure G-2)	$z_{lab, test}$	Depth reflected by lab. tests	Refers to cyclic soil test data given in Appendix G
$(T_f)_{ref}$	Reference time to failure	$(T_f)_{ref} = 60$ minutes (Figure G-2)	z_{min}	Minimum penetration depth	To ensure deep embedment (deep failure), $z_{plate} > 4.5W_F$ below seabed and $z_{plate} > 1.5W_F$ below layer boundary
T_i	Target installation tension	Specified installation tension at the dip-down point, see Figure E-1	z_{plate}	Depth of plate	Reference depth for calculation of $s_{u,I}$ and $s_{u,III}$ ($z_i = z_{plate}$)
T_{min}	Minimum installation tension	Specified installation tension at the touch-down point (if $L_{s,i} > 0$), for $L_{s,i} = 0$, $T_{min} = T_i$	z_t	Target penetration depth	Accounts for loss of penetration depth Δz_k and Δz_f
U_{cy}	Cyclic loading factor	$U_{cy} = \tau_{f,cy}/s_u$	Δz_k	Keying distance	Loss in penetration depth due to anchor keying and rotation
U_r	Loading rate factor	$U_r = (v_i/v_2)^n$ $U_r = (v/v_{ref})^n$	Δz_f	Failure displacement	Loss in penetration depth due to anchor failure displacement
v	Loading (or strain) rate	Actual pullout rate	z_{test}	Penetration depth	After keying/triggering and rotation (before pullout test)
v_I	Loading (or strain) rate	Actual installation rate	z_{ult}	Ultimate penetration depth	Relates to R_{ult} , and is expressed in number of fluke widths, $\lambda \cdot W_F$

2. Design principles

2.1 Limit state method of design

In the design code for plate anchors outlined in Chapter 3 the safety requirements are based on the limit state method of design.

The design criterion to be satisfied is

$$R_d(z_i) - T_d \geq 0 \quad (2.1)$$

where $R_d(z_i)$ is the design anchor resistance at depth z_i , and T_d is the design line tension.

The design line tension T_d is obtained by multiplying the characteristic mean line tension component T_{C-mean} and dynamic line tension component T_{C-dyn} with their respective partial safety factor, γ_{mean} and γ_{dyn} .

$$T_d = T_{C-mean} \cdot \gamma_{mean} + T_{C-dyn} \cdot \gamma_{dyn} \quad (2.2)$$

where

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

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

A recipe for calculation of the characteristic line tension components is given in /7/.

The design anchor resistance $R_d(z_i)$ at depth z_i is defined as

$$R_d(z_i) = \frac{R_C(z_i)}{\gamma_m} = \frac{R_S(z_i) \cdot U_{cy}}{\gamma_m} = \frac{R_{cy}(z_i)}{\gamma_m} \quad (2.3)$$

where

R_C Characteristic anchor resistance

R_S Static anchor resistance

$U_{cy} = \frac{\tau_{f,cy}}{s_u}$ Cyclic loading factor

$\tau_{f,cy}$ Cyclic shear strength

s_u Intact undrained shear strength

R_{cy} Cyclic anchor resistance

γ_m Partial safety factor on $R_{cy}(z_i)$

A recipe for calculation of the characteristic anchor resistance is given in Chapter 3.

As stated in Section 1.1, anchor specific installation effects, of importance for the as-installed anchor resistance, are dealt with in Chapter 5, and tentative recommendations for accounting for such effects are given.

The partial safety factors for use in combination with this design code are presented in Table 2-1.

Table 2-1 Partial safety factor for line tension and anchor resistance

Limit State:	ULS		ALS	
	1	2	1	2
Consequence Class:				
Partial safety factor				
γ_{mean}	1.10	1.40	1.00	1.00
γ_{dyn}	1.50	2.10	1.10	1.25
γ_m	1.40	1.40	1.00	1.30

The limit states and consequence classes in Table 2-1 are described in the next two sections.

2.2 Limit states

The primary function of an anchor, in an offshore mooring system, is to hold the lower end of a mooring line in place, under all environmental conditions. Since extreme environmental conditions give rise to the highest mooring line tensions, the designer must focus attention on these conditions. If the extreme line tension causes the anchor to move beyond its failure displacement, then the anchor has failed to fulfil its intended function.

The mooring system shall be analysed according to design criteria formulated in terms of two limit state equations:

- An ultimate limit state (ULS) to ensure that the individual mooring lines have adequate strength to withstand the load effects imposed by extreme environmental actions.
- An accidental damage limit state (ALS) to ensure that the mooring system has adequate resistance to withstand the failure of one mooring line, failure of one thruster, or one failure in the thruster system for unknown reasons.

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.

2.3 Consequence classes

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

- Class 1 Failure is unlikely to lead to unacceptable consequences such as loss of life, collision with an adjacent platform, uncontrolled outflow of oil or gas, capsizing or sinking.
- Class 2 Failure may well lead to unacceptable consequences of these types.

Guidance Note

A considerable body of experience exists for the design of drilling platforms with acceptable service performance. This design experience is taken to be well represented by the requirements set in /7/, and is taken as a basis for establishing a suitable design basis for mooring systems in moderate water depths.

Drilling platforms generally tend to be dimensioned for consequence class 1 (CC1), while floating production

platforms tend to be dimensioned for consequence class 2 (CC2). In practice there are, however, situations when drilling platforms need to be designed for CC2 safety requirements, e.g. when a drilling platform operates in the proximity of an existing installation.

The amount of experience with floating production systems is much less than for drilling platforms and is not considered sufficient to set a target level for CC2. Instead, general guidance from /12/ is applied. This guidance indicates a factor of 10 between the target failure probabilities of the two consequence classes.

--- End of Guidance Note ---

The recommended design code for plate anchors in clay is described in Chapter 3. The emphasis is on the basis for calculation of the characteristic anchor resistance $R_C(z_i)$, and the minimum penetration depth z_i of the anchor to satisfy the specified reliability levels.

In Chapter 4 the alternative to design a mooring system by direct application of structural reliability analysis is introduced, instead of using the simplified design code in Chapter 3.

Although installation of plate anchors is not a major issue in this RP, installation effects on the penetration depth of the anchor is an important design consideration, especially when anchors are installed in layered soil. Requirements to verification of the final depth of the as-installed plate anchor are presented in Chapter 5.

3. Design code for plate anchors

3.1 Characteristic and design line tension

The characteristic line tension T_C and the design line tension T_d are defined in Section 2.1.

A recipe for calculation of the characteristic tension components is given in /7/.

3.2 Characteristic anchor resistance, R_C

3.2.1 Basic equations

The characteristic resistance $R_C(z_i)$ of a plate anchor at penetration depth z_i in layered clay is given by the following general equation

$$R_C(z_i) = N_c \cdot s_c \cdot \eta \cdot s_{u,mean}(z_i) \cdot A_{plate} \cdot U_{cy} \quad (3.1)$$

$$= R_S(z_i) \cdot U_{cy} = R_{cy}(z_i)$$

in which

$$R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot s_{u,mean}(z_i) \cdot A_{plate} \quad (3.2)$$

is the characteristic static resistance and

$$R_{cy}(z_i) = N_c \cdot s_c \cdot \eta \cdot \tau_{f,cy}(z_i) \cdot A_{plate} \quad (3.3)$$

is the cyclic anchor resistance.

The other parameters in Eq. (3.1) are

$$U_{cy} = \frac{\tau_{f,cy}}{s_u}$$

Cyclic loading factor

$$N_c$$

Bearing capacity factor

$$s_c = 1 + 0.2 \cdot W_F / L_F$$

Shape factor

$$W_F$$

Plate width

$$L_F$$

Plate length

$$\eta$$

Empirical reduction factor

$$s_{u,mean}$$

Mean undrained shear strength

$$A_{plate}$$

Plate area

$$z_i$$

Penetration depth

Characteristic values of the soil properties are defined as the mean values of the respective variables.

In the following, the parameters involved in the calculation of the static anchor resistance, R_S , are described, followed by a description of the recommended procedure for calculation of the cyclic anchor resistance R_{cy} by correcting R_S for the effects of cyclic loading.

3.2.2 Bearing capacity factor N_c

In Eq. (2.3), N_c is a bearing capacity factor, valid for plain strain conditions (strip footing) and isotropic, incompressible material, which is adjusted for the plate geometry through the shape factor s_c , which is a function of the ratio between the equivalent plate width W_F and the equivalent plate length L_F .

In the context of calculation of failure resistance of plate anchors in layered clay, the soil volume involved is related to the smallest dimension of the plate, i.e. the plate width W_F . At failure, the resistance of the plate is assumed to be mobilised within a spherical volume around the plate with a diameter equal $3W_F$.

For anchors penetrated less than the minimum depth $z_{min} = 4.5 \cdot W_F$, which qualifies for assuming "deep penetration" below the seabed, the N_c -value for plain strain conditions will vary with depth between 5.14 and 12.0. An expression for $N_c = (N_c)_{shallow}$ that accounts for the depth effect in the 'shallow' failure zone is given in Eq. (3.4), valid for a strip footing and for $z_i = z_{plate} \leq 4.5 W_F$. This relationship is also shown in Figure 3-1.

$$(N_c)_{shallow} = 5.14 \cdot \left(1 + 0.987 \cdot \arctan\left(\frac{z_i}{W_F}\right) \right) \quad (3.4)$$

which is valid for $N_c \leq 12.0$.

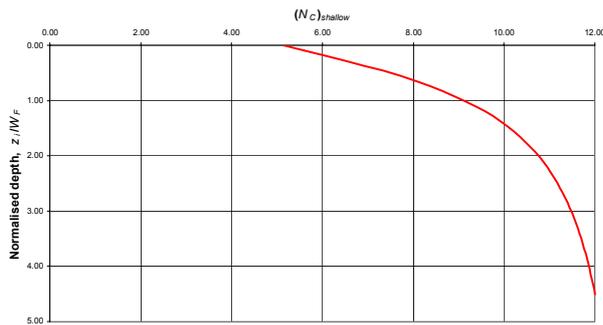


Figure 3-1 Variation of N_c in shallow failure zone.

For anchors penetrating deeper than $4.5W_F$ below the seabed, i.e. $z_{plate} > 4.5W_F$, it is recommended to use $N_c = (N_c)_{deep} = 12$.

3.2.3 Empirical reduction factor, η

The η -factor is an empirical reduction factor, set equal to 0.75 based on well controlled onshore field tests in a close to normally consolidated clay /8/ and /9/. This factor accounts for

- progressive failure (strain-softening) of the clay within the soil volume involved as the plate anchor is loaded to failure, and
- strength anisotropy, to the extent the actual average undrained shear within the soil volume affected by the failure differs from the assumed shear strength

Guidance Note

In a situation when the anchor has reached failure under the actual load conditions, the soil volume affected by this failure will have undergone a progressive type of failure. By progressive failure of the clay in this case is meant the degradation of the undrained shear strength, first in the zones experiencing the highest shear strain levels, then gradually expanding to other parts of the soil until the whole volume of soil involved in the failure is in a state of failure. The strain-softening characteristics of the actual clay can be interpreted from soil strength tests as the ratio between the post-peak strength (at large shear strains) and the peak strength of the clay.

Depending on the characteristics of the clay, the type of strength tests (UU triaxial, CAU triaxial or DSS tests), and the quality of the soil specimen tested, the strain-softening may be more or less pronounced.

It is desirable to continue testing both soil and anchors to improve the basis for assessment of the effect of strain-softening on the undrained shear strength and on the anchor resistance. It is recommended to use the actual laboratory test data to assess whether there is a justification for using a higher or a lower η -value for the actual site. As a guidance, for the test site in /8/ and /9/ the UU triaxial, CAU triaxial and the DSS tests showed an average strain-softening effect corresponding to an η -factor of about 0.80-0.85. However, the reduction in strength had not stopped at the strain reached when the tests were stopped. With this in mind combined with the anisotropy effect mentioned above, a value of 0.75 has been set as a default value in this design code.

Another potential correction factor, not addressed herein, is due to load eccentricity. If the plate anchor is not designed so that the load becomes normal to the plate at failure, there may be a need for a separate correction factor to account for the

load eccentricity. For the anchor tests referred to above there was no need to correct for eccentricity at failure.

--- End of Guidance Note ---

3.2.4 Anchors in layered clay

For a plate anchor in a two-layer clay profile, the most simple expression for the static anchor resistance R_S is

$$R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot (s_{u,0} + k_1 \cdot z_1 + \Delta s_u(z_1) + k_2 \cdot (z_i - z_1)) \cdot A_{plate} \quad (3.5)$$

in which

- $s_{u,0}$ seabed shear strength intercept
- k_1 and k_2 depth shear strength gradient of the respective layers
- $z_i = z_I$ layer boundary with a step change Δs_u

For a normally consolidated clay profile with $\Delta s_u = k_2 = 0$, Eq. (3.5) simplifies further to

$$R_S(z_i) = N_c \cdot s_c \cdot \eta \cdot (s_{u,0} + k_1 \cdot z_i) \cdot A_{plate} \quad (3.6)$$

Eqs. (3.5) and (3.6) assume that the undrained shear strength as given by the soil profile is used directly in the calculation of the static anchor resistance. This is a reasonable assumption for anchors installed in a normally consolidated clay, but not for anchors in layered clay. This is because the average undrained shear strength in the soil volume influencing the anchor resistance is dependent on both the variation of the shear strength across the layer boundary and the size of the anchor. This dependency is particularly evident for shallow penetrations of the anchor into a stiff layer underlying a softer layer. In the expression for the static anchor resistance R_S in Eq. (3.2) the effect of the discontinuity in the undrained shear strength at the transition from the soft to the stiff clay layer is accounted for by using the mean undrained shear strength $s_{u,mean}$.

In calculating $s_{u,mean}$ the plate is assumed to divide the soil volume influencing the anchor resistance in two half-spheres, one located above and the other below the plate, termed Zone I and Zone III, respectively. Each of these zones is assigned an average undrained shear strength termed $s_{u,I}$ and $s_{u,III}$, respectively.

The mean undrained shear strength $s_{u,mean}$ to be used for calculation of the anchor resistance is the average value of $s_{u,I}$ and $s_{u,III}$.

$$s_{u,mean} = (s_{u,I} + s_{u,III}) / 2 \quad (3.7)$$

For calculation of $s_{u,I}$ and $s_{u,III}$, Zone I and Zone III are both divided into six slices, and the undrained shear strength within each slice is multiplied by a weighting factor a_i ($i = 1$ to 6) reflecting the relative contribution of that slice to the average undrained shear strength of the

respective zone. The weighting factors are calculated for a circular cross-section through the centre of the plate at depth $z_i = z_{plate}$.

The reference depth for calculation of $s_{u,mean}$ is the depth of the plate z_{plate} , and Zone I and Zone III extend $1.5W_F$ up and down, respectively, from z_{plate} , see Figure 3-2.

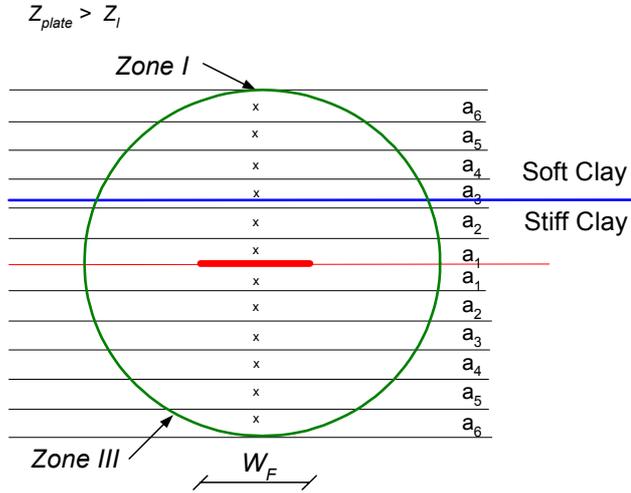


Figure 3-2 Calculation of $s_{u,I}$ and $s_{u,III}$ when $z_{plate} > z_I$.

When calculating $s_{u,mean}$ for plate depths $z_{plate} < z_I$, the two zones are assumed to extend only $0.5W_F$ up and down from the depth of the plate, see Figure 3-3. By reducing the diameter of the sphere representing the boundary for calculating $s_{u,mean}$ for $z_{plate} < z_I$, a better representation of the undrained shear strength near the layer boundary is obtained.

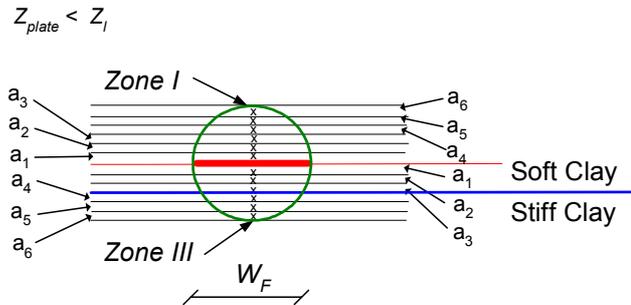


Figure 3-3 Calculation of $s_{u,I}$ and $s_{u,III}$ when $z_{plate} < z_I$.

The expressions for $s_{u,I}$ and $s_{u,III}$ are

$$s_{u,I} = \sum_{i=1}^6 s_u \left(z_{plate} + \frac{W_F}{b} (1 - 2 \cdot i) \right) \cdot a_i \quad (3.8)$$

$$s_{u,III} = \sum_{i=1}^6 s_u \left(z_{plate} - \frac{W_F}{b} (1 - 2 \cdot i) \right) \cdot a_i$$

in which

$$a_1 = 0.216 \quad (\text{for slice closest to the plate})$$

$$a_2 = 0.210$$

$$a_3 = 0.197$$

$$a_4 = 0.175$$

$$a_5 = 0.141$$

$$a_6 = 0.061$$

$$b = 8 \quad (\text{for } z_{plate} > z_I)$$

$$b = 24 \quad (\text{for } z_{plate} < z_I)$$

For anchors in layered clay penetrating deeper than $1.5 \cdot W_F$ into a stiff clay layer underlying a softer layer, $s_{u,mean}$ is equal to the undrained shear strength at the depth of the plate, i.e. $s_{u,mean}(z_i = z_{plate}) = s_u(z_i = z_{plate})$, provided that the undrained shear strength varies linearly with depth in the stiff layer, as shown by Eq. (3.6).

If the penetration into the stiff layer is less than $1.5 \cdot W_F$, the undrained shear strength to be used in the calculation of anchor resistance will be a function of the strength of both the softer and the stiffer layer, as well as of the plate width W_F , which may be expressed through the mean strength $s_{u,mean} = f(z_{plate}, W_F)$, as defined by Eqs. (3.7) and (3.8) above.

For anchors penetrating deeper than $4.5W_F$ below the seabed into a normally consolidated clay, i.e. $z_{plate} > 4.5W_F$, $s_{u,mean}$ is equal to the undrained shear strength at the depth of the plate, i.e. $s_{u,mean}(z_i = z_{plate}) = s_u(z_i = z_{plate})$, provided that the undrained shear strength of the clay varies linearly with depth, as shown by Eq. (3.6).

3.2.5 Cyclic loading factor U_{cy}

3.2.5.1 Basic equations

The cyclic loading factor U_{cy} is defined as

$$U_{cy} = \frac{\tau_{f,cy}}{s_u} \quad (3.9)$$

where $\tau_{f,cy}$ is the cyclic shear strength for a particular, specified storm load amplitude history and s_u is the static undrained shear strength, for the volume of soil influencing the resistance of the anchor.

The cyclic loading factor in purely two-way cyclic loading is denoted U_{cy0} . The equivalent number of cycles to failure N_{eqv} is defined as the number of cycles at a stress amplitude equal to $\tau_{f,cy}$ required to cause failure in cyclic loading.

The loading of the anchor is a one-way cyclic loading, and the cyclic loading factor U_{cy} for this one-way cyclic loading depends on the normalised average shear stress τ_a/s_{uD} , where τ_a denotes the average shear stress and s_{uD} is the static DSS undrained shear strength. By capitalisation on laboratory test results for Drammen clay subject to one-way cyclic loading, the model presented above for the cyclic loading factor for a particular clay in purely two-way cyclic loading can be expanded to one-way cyclic loading of the same clay as presented in the following.

The Drammen clay data base is maybe the only cyclic test data set that contains results which can be used to assess the effect of the average shear stress level τ_a on the cyclic loading factor U_{cy} . The following expression for U_{cy} has been developed for the Drammen clay data base.

$$U_{cy} = U_{cy, Drammen} \cdot \left(\frac{U_{cy0}}{U_{cy0, Drammen}} \cdot \left(1 - \frac{\tau_a}{s_{u,D}} \right) + \frac{\tau_a}{s_{u,D}} \right) \quad (3.10)$$

in which

$$U_{cy, Drammen} = a_0 + a_1 \cdot \left(\frac{\tau_a}{s_{u,D}} \right) + a_2 \cdot \left(\frac{\tau_a}{s_{u,D}} \right)^2 + a_3 \cdot \left(\frac{\tau_a}{s_{u,D}} \right)^3 \quad (3.11)$$

and

$$U_{cy0, Drammen} = a_0 \quad (3.12)$$

The coefficients a_0 , a_1 , a_2 , and a_3 depend on the equivalent number of cycles N_{eqv} , and are determined from laboratory tests on Drammen clay subjected to one-way cyclic loading. The following expressions for the four coefficients are found to honour the test data, see Figure 3-4.

$$\begin{aligned} a_0 &= -0.1401 \cdot \ln(N_{eqv}) + 1.2415 \\ a_1 &= 0.0995 \cdot \ln(N_{eqv}) + 1.0588 \\ a_2 &= -0.5795 \cdot \ln(N_{eqv}) + 0.3426 \\ a_3 &= 0.6170 \cdot \ln(N_{eqv}) - 1.6048 \end{aligned} \quad (3.13)$$

Guidance Note

As a minimum, a series of two-way, DSS type, cyclic loading tests and compatible static tests, should be conducted to establish the necessary site-specific data base for use together with the Drammen clay test data base as proposed herein.

The use of the Drammen clay data in lieu of site-specific data for one-way cyclic loading is considered to be adequate.

This is so because for $\tau_a/s_{u,D}$ near its lower limit of zero, the use of the Drammen clay data has the character of a pure normalisation, whereas for $\tau_a/s_{u,D}$ near its upper limit of 1.0, the dependency on N_{eqv} vanishes as U_{cy} approaches 1.0.

The equivalent number of cycles N_{eqv} is the number of cycles of the extreme load amplitude, expressed by $\tau_{f,cy}$, in a cyclic loading history that would have the same effect as the application of the complete load history to the clay.

If the cyclic loading factor U_{cy} is set equal to 1.0 it will normally be on the safe side.

--- End of Guidance Note ---

3.2.5.2 Correction for OCR

For calculation of the OCR of the underlying stiff clay, given the variation of $s_{u,D}$ both in the soft and the stiff clay layer, the following formula may be used

$$OCR^m = \left(\frac{(s_{u,D} / \sigma'_{v,0})_{OCR}}{(s_{u,D} / \sigma'_{v,0})_{OCR=1}} \right) \quad (3.14)$$

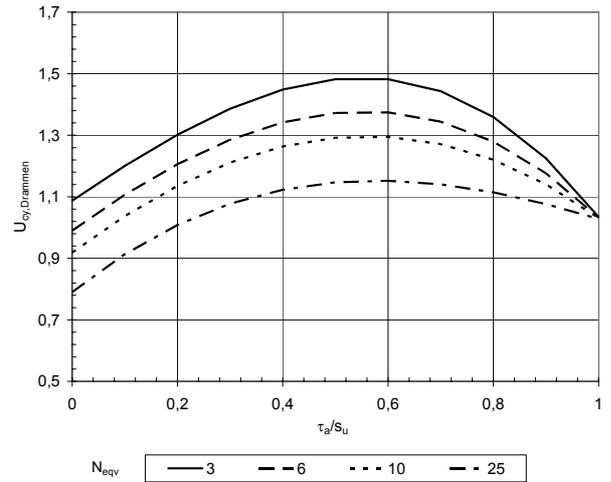


Figure 3-4 Model for U_{cy} vs. τ_a/s_u and N_{eqv} for Drammen clay.

which also can be written

$$OCR = \left(\frac{(s_{u,D})_{OCR}}{(s_{u,D})_{OCR=1}} \right)^{1/m} \quad (3.15)$$

Knowing the undrained shear strength of the normally consolidated clay $s_{u,D} = (s_{u,D})_{OCR=1}$, one may thus calculate the resulting OCR for an overconsolidated layer with $s_{u,D} = (s_{u,D})_{OCR}$. The exponent m will be a function of the clay characteristics, but for guidance a value of 0.82 has been found appropriate both for the Drammen clay and for the Marlin clay.

An approximate expression has been derived for an adjustment factor K_{OCR} for correction of U_{cy} from Eq. (3.11) for the effect of OCR, which is based on the cyclic test data in the Drammen clay data base. The derived expression for K_{OCR} is

$$K_{OCR} = 1 - B \cdot \frac{2}{\pi} \cdot \arctan[A \cdot (OCR - 1)] \quad (3.16)$$

in which

$$A = 0.359 \cdot \exp\left(-0.543 \cdot \frac{\tau_a}{s_{u,D}}\right)$$

$$B = 0.429 \cdot \ln\left[1 + 1.496 \cdot \left(1 - \frac{\tau_a}{s_{u,D}}\right)\right]$$

$$OCR = \left[\frac{(s_{u,D})_{OCR}}{(s_{u,D})_{OCR=1}} \right]^{1/m}$$

The cyclic loading factor U_{cy} with correction for OCR can then be calculated from

$$(U_{cy})_{OCR} = K_{OCR} \cdot (U_{cy})_{OCR=1} \quad (3.17)$$

In Figure 3-5, the adjustment factor K_{OCR} is shown as a function of OCR and $\tau_a/s_{u,D}$.

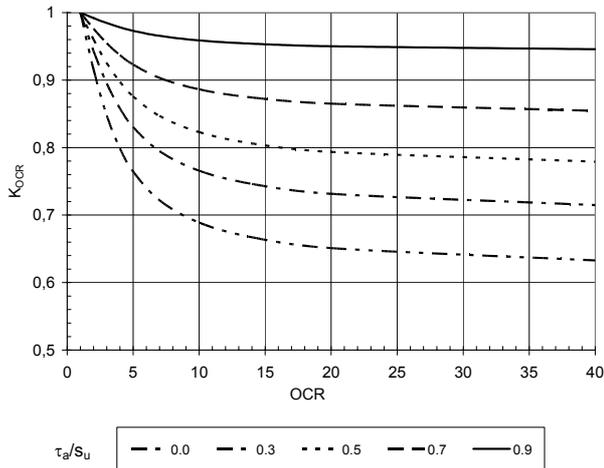


Figure 3-5 Adjustment factor K_{OCR} vs. OCR

3.2.5.3 Recommended procedure for calculation of U_{cy}

The recommended procedure for calculation of the characteristic value of U_{cy} is outlined in the following.

As for the other resistance properties, a value based on mean properties is to be used. The value depends on the value of the normalised mean shear stress, $\tau_a/s_{u,D}$. The value of $\tau_a/s_{u,D}$ can be calculated according to

$$\frac{\tau_a}{s_{u,D}} = \frac{T_{d,mean}}{T_d} \cdot U_{cy} \quad (3.18)$$

which is based on conditions prevailing at the failure situation. It appears that $\tau_a/s_{u,D}$ is dependent on U_{cy} . This implies that an initial guess of U_{cy} must be made. The sought-after value of U_{cy} is determined from the relationship

$$U_{cy} = K_{OCR} \cdot U_{cy,Drammen} \cdot \left[\left(\frac{U_{cy0}}{U_{cy,Drammen}} \right) \cdot \left(1 - \frac{\tau_a}{s_{u,D}} \right) + \frac{\tau_a}{s_{u,D}} \right] \quad (3.19)$$

in which U_{cy0} and $U_{cy0,Drammen}$ are the mean values of U_{cy} in purely two-way cyclic loading for the clay in question and for Drammen clay, respectively.

$U_{cy,Drammen}$ is a function of $\tau_a/s_{u,D}$ and can be represented by Eq. (3.11). K_{OCR} is also a function of $\tau_a/s_{u,D}$ and can be represented by Eq. (3.16). Note that Eq. (3.11) accounts for the equivalent number of cycles N_{eqv} , see details in Eq. (3.13), whereas the expression for K_{OCR} in Eq. (3.16) accounts for the overconsolidation ratio OCR .

Characteristic values are to be used for both N_{eqv} and OCR . The characteristic value of N_{eqv} is to be taken as the mean value of N_{eqv} . Similarly, the characteristic value of OCR is to be taken as the mean value OCR .

The characteristic value of the cyclic loading factor U_{cy} is obtained through the following iterative procedure:

- 1) Assume initial value of U_{cy}
- 2) Insert this value into Eq. (3.18) and get $\tau_a/s_{u,D}$
- 3) Calculate $U_{cy,Drammen}$ according to Eq. (3.11) for $\tau_a/s_{u,D}$ from Step 2
- 4) Calculate U_{cy} according to Eq. (3.19) for $U_{cy,Drammen}$ from Step 3 and $\tau_a/s_{u,D}$ from Eq. (3.18) with K_{OCR} from Eq. (3.16), $U_{cy0,Drammen}$ from Eq. (3.12), and U_{cy0} assigned the appropriate value for the clay in question.
- 5) Update U_{cy} in Step 1 and repeat Steps 2-4 until convergence.

3.3 Design anchor resistance, R_d

Adopting the formulation in Eq. (3.1) the design anchor resistance becomes

$$R_d(z_i) = \frac{R_C(z_i)}{\gamma_m} = \frac{R_S(z_i) \cdot U_{cy}}{\gamma_m} = \frac{R_{cy}(z_i)}{\gamma_m} \quad (3.20)$$

The design criterion to satisfy is

$$R_d(z_i) - T_d \geq 0 \quad (3.21)$$

where T_d is given by Eq. (2.2).

This leads to the following expression for the design anchor resistance

$$R_d(z_i) = \frac{R_C(z_i)}{\gamma_m} = N_c \cdot s_c \cdot \eta \cdot s_{u,mean}(z_i) \cdot A_{plate} \cdot U_{cy} \cdot \left(\frac{1}{\gamma_m} \right) \quad (3.22)$$

The characteristic anchor resistance $R_C(z_i)$ is then equal to the cyclic anchor resistance $R_{cy}(z_i)$, which in turn implies that the characteristic undrained shear strength is set equal to the cyclic shear strength $s_u(z_i) = \tau_{f,cy}(z_i)$.

$s_{u,mean}$ is the mean undrained shear strength, see description in Section 3.2.4, and is used as the characteristic undrained shear strength over the soil volume influenced by the anchor, and it incorporates also the influence of anchor size on the characteristic undrained shear strength. This formulation of the characteristic undrained shear strength shall be used when the anchor is installed into layered clay.

Inserting $R_d(z_i)$ from Eq. (3.20), $R_{cy}(z_i)$ from Eq. (3.3), and $R_S(z_i)$ from Eq. (3.5) into Eq. (3.21) gives the following inequality to satisfy for a two-layer clay profile, without accounting for the effect of plate width on the undrained shear strength.

$$N_c \cdot s_c \cdot \eta \cdot (s_{u,0} + k_1 \cdot z_i + \Delta s_u(z_i) + k_2 \cdot (z_i - z_1)) \cdot A_{plate} \cdot U_{cy} \cdot \left(\frac{1}{\gamma_m}\right) - T_d \geq 0 \quad (3.23)$$

In order to account for the influence of the plate width in this context, the mean undrained shear strength, $s_{u,mean}(z_i)$, must be used, see Eq. (3.25) below.

Resolving Eq. (3.23) and expressing the design requirement in terms of a minimum penetration depth by setting $z_i = z_{plate}$ give the following simple design requirement for a 1-layer profile (with $\Delta s_u = 0$)

$$z_{plate} \geq \frac{1}{k_1} \cdot \left[\frac{T_d \cdot \gamma_m}{N_c \cdot s_c \cdot \eta \cdot A_{plate} \cdot U_{cy}} - s_{u,0} \right] \quad (3.24)$$

for a plate installed to a depth $z_i \geq z_{min}$ into a clay profile with a seabed shear strength intercept $s_{u,0}$ and a shear strength gradient k_1 .

Eq. (3.24) will be sufficient for calculation of the anchor resistance in a normally consolidated clay, since accounting for the plate width in the calculation of the undrained shear strength, or not, will not change the result of the calculation in this case.

For a plate penetrating into the second stiffer layer of a 2-layer profile with the layer boundary at depth $z = z_1$, the anchor must penetrate beyond the minimum penetration depth $z_i = z_1 + z_{min}$ to avoid that the lower shear strength of the upper softer layer affects the anchor resistance. If the penetration into the stiff layer is less than z_{min} the undrained shear strength to be used in the calculation of anchor resistance will be the mean strength $s_{u,mean} = f(z_i, W_F)$, i.e. it will be a function of both the penetration depth z_i and the plate width W_F .

By using $s_{u,mean} = f(z_i, W_F)$ in lieu of the actual s_u at depth $z_i = z_{plate}$ the effect of the variation of s_u near the layer boundary has been taken into account.

Substituting the expression for s_u in Eq. (3.23) with $s_{u,mean}$ leads to the following limit state function for the plate anchor:

$$N_c \cdot s_c \cdot \eta \cdot s_{u,mean}(z_i) \cdot A_{plate} \cdot U_{cy} \cdot \left(\frac{1}{\gamma_m}\right) - T_d \geq 0 \quad (3.25)$$

in which T_d is taken from Eq. (2.2), and the partial safety factors γ_{mean} , γ_{dyn} , and γ_m are given in Table 3-1.

Two consequence classes are defined, see Section 2.3, which apply to both the ULS and the ALS:

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

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

The target annual probability level is set to 10^{-4} for Consequence Class 1 and 10^{-5} for Consequence Class 2, see also Chapter 4.

Table 3-1 Partial safety factor for line tension and anchor resistance

Limit State:	ULS		ALS	
	1	2	1	2
Consequence Class:				
Partial safety factor				
γ_{mean}	1.10	1.40	1.00	1.00
γ_{dyn}	1.50	2.10	1.10	1.25
γ_m	1.40	1.40	1.00	1.30

Guidance Note

As defined in Section 2.2, the ALS is intended to ensure that the mooring system has adequate resistance to withstand the failure of one mooring line. If the extreme line tension causes the anchor to move beyond its failure displacement, then the anchor has failed. However, in many cases the anchor will still have a significant residual resistance after failure that makes an anchor failure less dramatic than a line failure.

An important exception to this scenario is when an anchor is installed to a shallow depth into a stiff layer underlying a softer layer, see Section 3.2.4, in which case an anchor failure may be comparable with a line failure in terms of its consequences. Subsequent failure of any anchor is conservatively assumed to imply mooring system failure in the ALS. Thus, the ALS is formulated to avoid anchor failure, similarly to the ULS.

--- End of Guidance Note ---

4. Reliability analysis

4.2 General

A mooring system may be designed by direct application of structural reliability analysis, as an alternative to the simplified design calculation presented in Chapter 3.

Such an analysis should be at least as refined as the reliability analysis used to calibrate the present design code, see /3/ and /6/, and must be checked against the results of the calibration, for at least one relevant test case.

4.3 Target annual probabilities

The probability levels given in Table 4-1 have been applied in the calibration, and should also be applicable in a comparable reliability analysis.

Table 4-1 Probability levels

Limit state	Consequence class	Target annual probability of failure
ULS	1	10^{-4}
	2	10^{-5}
ALS	1	10^{-4}
	2	10^{-5}

The target level chosen for mooring lines in CC1 is based on experience with DNV's rules for design of mooring lines for mobile offshore drilling units over two decades. These requirements have been found to correspond to a probability of failure of 10^{-4} .

It is appropriate to apply the same target reliability to plate anchors in CC1, because plate anchor failure is comparable to line failure and has similar consequences.

5. Installation effects on penetration depth

5.1 General

The necessary anchor penetration depth to meet the prescribed reliability level is calculated according to the design code, see Chapter 3. The calculated depth z_{calc} is the depth at which the anchor is assumed to fail under the influence of the applied design line tension history, and is identical to z_{plate} .

Since this maximum anchor resistance cannot be mobilised without anchor displacement, it may be necessary to consider how much of the penetration depth is lost due to mobilisation of the anchor resistance, i.e. the anchor failure displacement, Δz_f . The most significant loss in penetration depth occurs, however, during anchor keying and rotation Δz_k .

The target penetration depth z_t will be the calculated depth z_{calc} plus the keying distance Δz_k and the anchor failure displacement Δz_f . The keying distance and anchor failure displacement are discussed, and guidance for their assessment is given, in Appendix I.

5.2 Anchor keying and rotation

5.2.1 Push-in type plate anchors

Plate anchors should be subjected to adequate keying loads to ensure that sufficient anchor fluke rotation will take place so that the anchor will become the reliable mooring system component as designed to be. The keying load required and the amount of estimated fluke rotation should be based on reliable geotechnical analysis and verified by prototype or scale model testing using adequate instrumentation. The keying analysis used to establish the keying load should also include analysis of the anchor's rotation when subjected to the ULS and ALS loads. The use of a keying flap will often reduce the loss of penetration depth due to keying and rotation.

Examples of push-in type plate anchors are the SEPLA, the PADER, the PPA, and the BLADE, which are briefly described in Appendix B.

5.2.2 Drag-in plate anchors

Drag-in plate anchors are installed by dragging the anchor through the soil, using e.g. the bollard pull from an anchor handling vessel (AHV), which gives a penetration vs. drag curve with the anchor plate approaching a horizontal position. An important difference between a drag-in plate anchor and a push-in type plate anchor is thus that the fluke of a drag-in plate anchor is close to horizontal rather than vertical immediately after installation.

Depending on the installation direction of the anchor, with respect to the centre of the mooring pattern, the keying distance will vary, see more about this in Appendix I.

Examples of drag-in type plate anchors are the STEVMANTA and the DENLA, which are described in Appendix B.

5.3 Verification of as-installed penetration depth

Methods for verification of the as-installed depth of the plate anchor after installation and keying should be developed, tested and verified with the involvement of anchor manufacturers, installation contractors and oil companies.

Reliable methods to measure the inclination of the plate relative to the vertical or horizontal plane should be developed and used as part of the instrumentation to verify that the plate anchor has been installed to the target penetration depth and that the subsequent keying is accomplished as planned and assumed. The results from these installation measurements will be used for comparison with the installation acceptance criteria, particularly the keying load and the post-keying penetration depth of the anchor. The plate inclination measurements will be an aid in connection with the final assessment of the anchor installation.

The calculated penetration depth z_{calc} comes out as a result of the design calculation, e.g. equal to z_{plate} in Eq. (3.24) for a 1-layer normally consolidated clay. However, this depth does not account for the potential loss in penetration depth due to anchor keying, Δz_k , and anchor failure displacement, Δz_f , which is discussed in more detail in Appendix I.

6. Requirements to soil investigation

The geotechnical design and analysis of plate anchors for permanent and temporary mooring systems should be based on reliable information concerning the soil properties at the location.

Reliable information concerning the soil properties should be based on a site specific soil investigation and the interpretation of the soil properties by a recognised geotechnical expert. The design soil parameters in the different soil strata should be determined from a programme that tests the soil in as nearly an undisturbed

state as feasible. Because the quality of soil samples can be expected to decrease with increasing water depth, it is recommended to include in-situ testing techniques for deepwater sites. In addition, soil samples will be required to characterise the soil types and provide other basic engineering property data. It is also recommended that a high-quality, high-resolution geophysical sub-bottom survey, combined with a geological interpretation of that survey, be combined with the geotechnical data in order to assess restraints imposed on the design by geological features and to aid in the interpretation of the geotechnical data obtained during the site investigation.

The planning and execution of soil investigations for design of 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 /10/, which makes extensive references to international standards. Examples of integrated geoscience studies for permanent mooring systems are given in /11/ and /12/.

For design of 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 plate anchors in clay are the intact undrained shear strength s_u , the remoulded undrained shear strength $s_{u,r}$, the clay sensitivity S_r , and the cyclic shear strength $\tau_{f,cy}$ for each layer of significance.

As a minimum, the soil investigation should provide the basis for specification of a representative soil profile and the undrained shear strengths, s_u and $s_{u,r}$, respectively, for each significant soil layer between the seabed and the estimated anchor penetration depth z_b , plus an adequate margin. The number of soil borings/in situ tests required to map the soil conditions within the mooring area should be decided from case to case.

Plate anchors are primarily of interest for soft normally to slightly overconsolidated clay, although thin sand layers may be penetrated both with drag-in plate anchors and push-in type plate anchors.

The ultimate penetration depth z_{ult} 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 ultimate penetration depth in terms of fluke widths W_F . In very soft clay z_{ult} may be up to 12-15 W_F , decreasing with increasing strength of the clay.

In the case of push-in type plate anchors, the limiting penetration depth is governed by the height of the suction follower, as for the SEPLA anchor, or by the capacity of the crane, as for the PADER anchor.

The necessary depth of a soil investigation is a project specific decision, but the depth should be well beyond the maximum expected penetration of the anchor. The upper few metres of the soil profile are of interest particular for assessment of the initial anchor penetration resistance and, for drag-in plate anchors, the choice of fluke angle. This part of the soil profile will also have an influence on the penetration (cutting) resistance against the embedded part of the anchor line and the resulting inverse catenary, which is important for all types of plate anchor.

General requirements to the soil investigation for plate anchor foundations, in addition to the recommendations in /10/, are provided in Appendix J.

7. References

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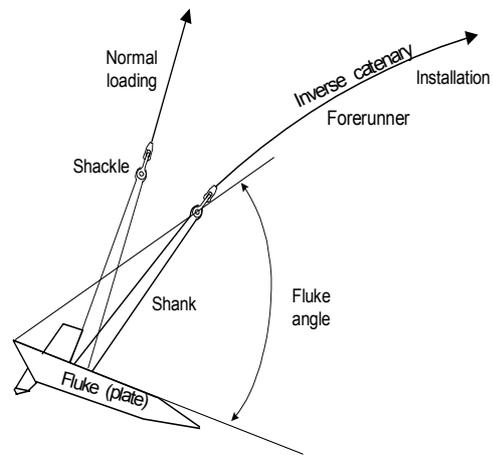
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Appendix A Characteristics and behaviour of drag-in plate anchors

A1 General

Drag-in plate anchors have been developed for use in combination with taut mooring systems (TMS) and they can resist both the vertical and the horizontal loads transferred to the anchors in such a system. This anchor is installed as a conventional fluke anchor, see /4/, and when the target installation load T_i has been reached it is triggered to create normal loading against the fluke (plate). In this normal loading mode the anchor acts as an embedded plate with a high pullout resistance. Examples of drag-in plate anchors currently in use are the DENLA from Bruce Anchor and STEVMANTA from Vryhof Ankers, see brief description of these anchors in Appendix B.



A2 Drag-in plate anchor components

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

- the shank
- the fluke
- the shackle
- the forerunner

Guidance Note

The *fluke angle* is the angle arbitrarily defined by the fluke plane and a line passing through the rear of the fluke and the anchor shackle. It is important to have a clear definition (although arbitrary) of how the fluke angle is being measured. Normally the fluke angle is fixed within the range 30° to 50° , the lower angle used for sand and hard/stiff clay, the higher for soft normally consolidated clays. Intermediate angles may be more appropriate for certain soil conditions (layered soils, e.g. stiff clay above softer clay). The advantage of using the larger

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

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. The *forerunner* is the line segment attached to the anchor shackle, which will embed together with the anchor during installation. The anchor penetration path and the ultimate depth/resistance of the anchor are significantly affected by the type (wire or chain) and size of the forerunner, see Figure A-2.

The *inverse catenary* of the anchor line is the curvature of the embedded part of the anchor line, see Figure A-2.

--- End of Guidance Note ---

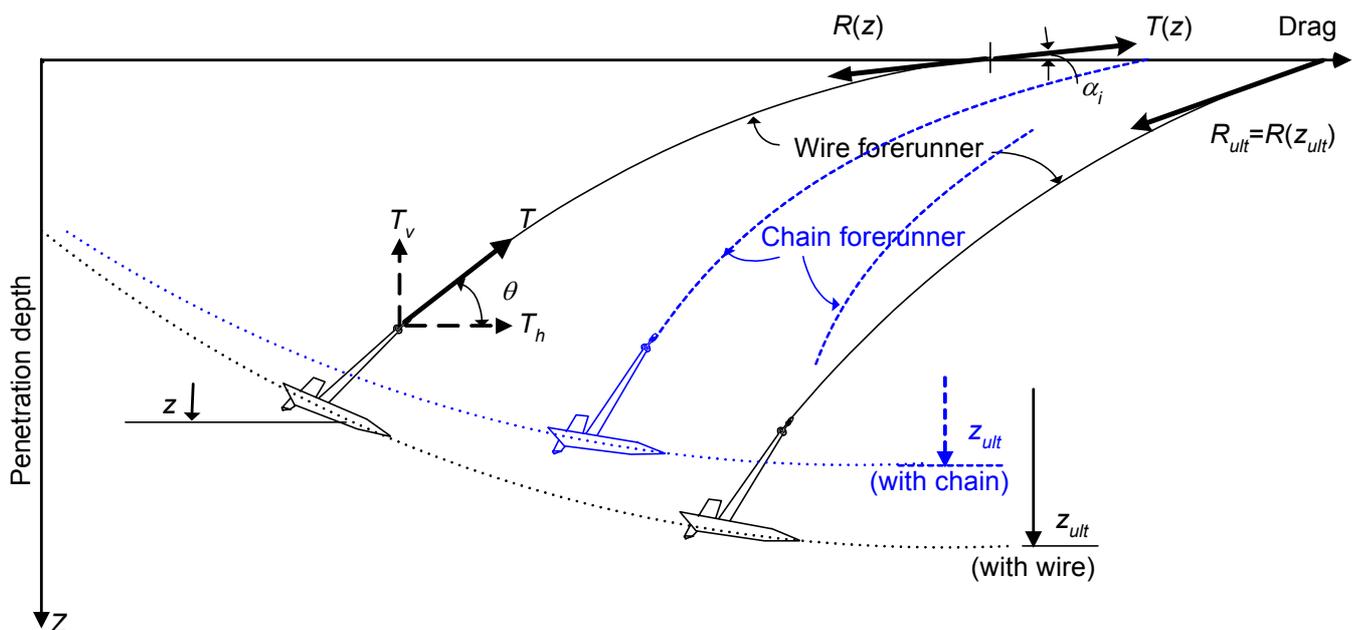


Figure A-2 Installation behaviour of drag-in plate anchors and definition of R_{ult}

A3 Penetration phase

The pullout resistance of a drag-in plate anchor depends on the ability of the anchor to penetrate and to reach the target installation tension (T_i).

The penetration path for a drag-in plate anchor and the ultimate penetration depth z_{ult} depends on

- 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 (chain or wire and nominal diameter), and
- the installation uplift angle α_i at the seabed level.

The predicted ultimate penetration z_{ult} of the anchor is crucial for sizing the anchor, given T_i and the shear strength profile.

Guidance Note

In clay without significant layering, a fluke anchor normally penetrates along a path, where the ratio between incremental penetration and drag decreases with depth, see Figure A-2 with the DENLA anchor as an example. At the ultimate penetration depth z_{ult} the anchor is not penetrating any further. The anchor is "dragging" with a near horizontal fluke, and the tension in the line is constant. At the ultimate penetration depth the anchor has reached its ultimate penetration resistance R_{ult} . It is important not to overestimate z_{ult} . In the worst case the target installation tension T_i 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 assume that the anchor is installed to a depth z_i , which is significantly less than z_{ult} .

It has been demonstrated in /1/ that a non-zero installation uplift angle α_i , see Figure A-2, can be acceptable under certain conditions as discussed in more detail in Appendix C, see also /3/. Drag-in plate anchors in deep water will normally be installed under an uplift angle, in which case there will be no line on the seabed. A high uplift angle reduces the contribution from the anchor line to the anchor resistance $R(z)$, which leads to an increased penetration of the anchor for a given installation tension T_{min} . If the uplift angle becomes excessive during installation the ultimate penetration depth may, however, be reduced, see Appendix C, Section C3 regarding short scope installation in deep water.

--- End of Guidance Note ---

Installation of drag-in plate anchors in layered soil requires special caution, see more about this in Appendix D.

For a successful anchor installation it is important to work out a detailed plan and dimension the spread so that the necessary installation tension can be reached ensuring that the reliability of the anchor points after installation satisfy the design assumptions. Guidance for planning of the installation is given in Appendix E.

Guidance Note

The cutting resistance of a chain forerunner will be greater than the resistance of a steel wire, with the result that a chain forerunner will have a steeper curvature (inverse catenary) at the anchor shackle than a wire forerunner, i.e. the angle θ at the shackle is larger, see Figure A-2. This increases the upward vertical component T_v of the line tension T at the shackle

leading to less penetration with a chain forerunner than with a wire forerunner, and less resistance for a given drag distance. Increasing the diameter of a steel wire forerunner will also reduce the anchor penetration for the same reason.

--- End of Guidance Note ---

Consolidation effects on the pullout resistance should normally be disregarded, but it should be noted that stoppage during installation of a drag-in plate anchor will increase the restart installation resistance after the stoppage due to the consolidation (setup) effect.

Possible long term consolidation effects on the pullout resistance may be evaluated on a case-by-case basis, see Appendix F.

The basis for calculation of the effects of consolidation, cyclic loading and uplift at the seabed are discussed in Appendix F and Appendix G, respectively. Although creep is not foreseen to become a problem with drag-in plate anchors designed to satisfy the ULS and ALS conditions and the design procedure recommended herein, this is a design issue that needs to be addressed. Tentative guidance for addressing this issue is given in Appendix G.

An analytical tool for prediction of the penetration trajectory versus installation tension should be calibrated against well controlled and instrumented anchor test data. An example of such an analytical tool is the DIGIN programme, see Appendix H, and /5/, developed by DNV in the course of the JIP on fluke anchors and drag-in plate anchors, see /1/ and /2/.

Appendix B Examples of plate anchors

B1 The STEVMANTA anchor

The STEVMANTA anchor from Vryhof Ankers has no rigid shank, but a system of wires connected to a fixed plate angle adjuster. In the Vryhof Anchor Manual /B-1/ two installation methods are described in detail, the one-line installation method and the two-line installation method.

For illustration of the principles of the STEVMANTA anchor, the double-line installation method as illustrated in Figure B-1 will be used, see also /B-2/. The installation line is attached to the front-shackle of the angle adjuster, whereas the actual mooring line is attached to the back-shackle.

When pulling in the installation line, the fluke angle adjusts to the angle set for deep penetration in the actual soil conditions. This angle is determined by the length of the wires attached to the front of the fluke relative to the length of the wires attached to the rear of the fluke, which is controlled by the angle adjuster.

When the target installation tension has been reached, and held constant for a period of 15 to 30 minutes, the installation line is buoyed off and the mooring line is tensioned. This rotates the angle adjuster and creates the normal loading mode. The STEVMANTA is typically installed in the direction towards the mooring centre, which means that the anchor plate will need only a small rotation to reach the normal loading mode. If the anchor is installed in the direction away from the mooring centre, or along the periphery of the anchor pattern, more rotation is required to reach a normal loading mode.

Guidance for assessment of the necessary line tension (keying load) to rotate the plate in the clay is given in Appendix I.

When the anchor is installed by the double-line installation method, the anchor is retrieved by again pulling in the installation line as shown in Figure B-1.

B2 The DENLA anchor

The DENLA anchor from Bruce Anchor Ltd. has rigid shank, which is much smaller relative to the size and area of the fluke than one normally see on conventional fluke anchors. The principles for installing the DENLA drag-in plate anchor and preparing it for hook-up to the floater are illustrated in Figure B-2, see also Figure A-1 in Appendix A and /B-3/.

The anchor handling vessel (AHV), while steaming away from the centre of the mooring pattern, installs the DENLA to the target installation tension T_i . Then the AHV steams back, towards the centre of the mooring pattern, over the buried DENLA, applying a vertical/backward pull on the shank.

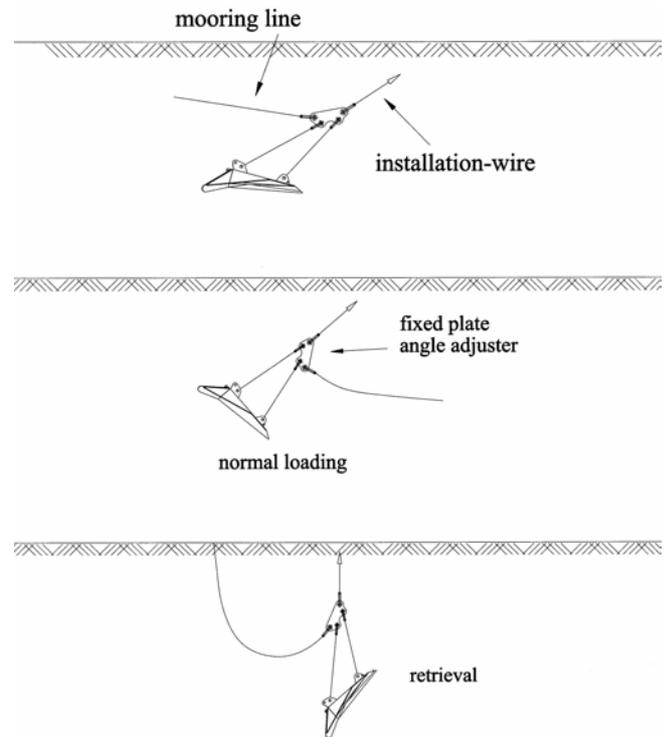


Figure B-1 Installation, normal loading and retrieval of the STEVMANTA anchor.

The shear pin controlling the installation fluke angle breaks at a certain predefined (triggering) load and the shank rotates and locks in a position, which creates normal loading on the fluke (plate). The AHV then continues steaming towards the centre and rotates the anchor into a position which gives approximately normal loading for the specified mooring line angle Φ with the seabed. To retrieve the anchor, the anchor is pulled in the installation direction, which rotates the shank until it closes and locks at a small fluke angle before the anchor is retrieved.

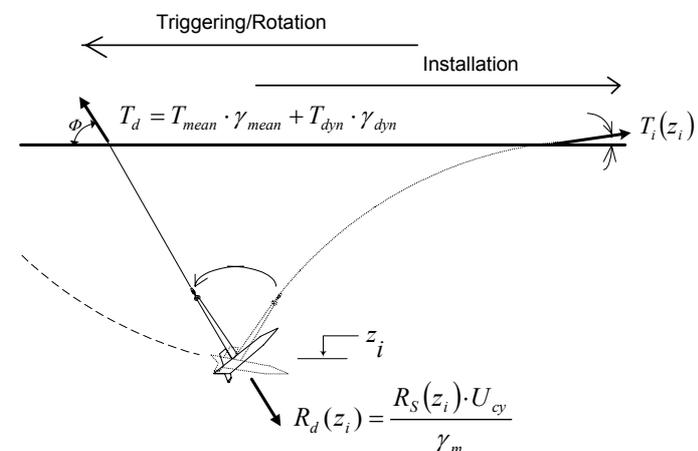


Figure B-2 Principles of the DENLA anchor.

B3 The SEPLA anchor

The SEPLA from Technip Offshore Moorings Inc., which is described in /B-4/, consists of a rectangular anchor plate with a full-length keying flap running along its top edge, see illustration in Figure B-3. The keying flap is mounted with an offset hinge such that soil pressure along its top edge will force the flap to rotate with respect to the plate, effectively quadrupling the vertical end bearing area of the SEPLA and preventing it from moving back up its installation track when tensioned. The mooring line is attached to the anchor plate by means of twin plate steel shanks. Typically, for MODU applications, the anchor plate is solid plate steel 2.5m to 3.0m in width and from 6m to 7.3m in length. For permanent installations, the plate will typically be a double-skin or hollow construction, 4.5m x 10m in size.

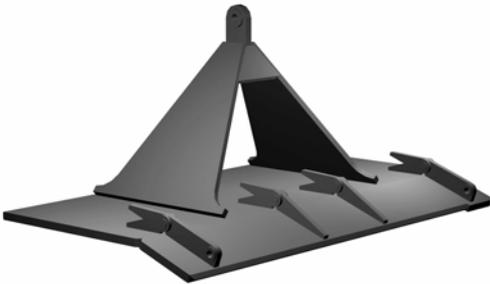


Figure B-3 Illustration of the SEPLA anchor.

The SEPLA uses a suction anchor (renamed to “follower”) for embedment to the target penetration depth. For installation, the SEPLA is mounted in slots at the bottom of the follower and retained by the mooring line and recovery bridle.

The suction follower, with the SEPLA slotted into its base, is lowered to the seabed, allowed to self-penetrate and then suction embedded in a manner similar to a suction anchor. Once the SEPLA has reached its target penetration depth, the mooring line and retrieval bridle that hold the SEPLA secure in the bottom of the follower are released by the installation ROV. The pump flow direction is then reversed and water is pumped back into the follower into the follower; the follower moves upward, leaving the SEPLA in place.

The SEPLA will be in a vertical orientation at this time. For a recoverable application, a small submersible buoy will support the recovery bridle. At this time, the mooring line is tensioned by the installation vessel in the direction the SEPLA is to be loaded. This “keying” load will:

- pull the initially vertical mooring line through the soil so that it forms the classic inverse catenary shape from the seabed to the anchor shackle,

- start rotation of the SEPLA anchor plate towards an orientation perpendicular to the direction of the mooring line at the anchor end, and
- set the keying flap to prevent further loss of penetration depth, beyond that necessary to set the keying flap.

At this time the SEPLA is ready to develop its full resistance.

B4 The PADER anchor

The PADER plate anchor from Subsea7 is penetrated vertically into the seabed. The driving force is gravity but the resistance can be reduced by circulating water through jetting nozzles at the tip of the plate. The system allows for a controlled rotation of the plate just as it reaches its final position.

The plate anchor consists of a thick steel plate in a standard material. Around the outer edge, rectangular hollow sections have been welded to the plate. These are part of the manifold arrangement to feed the nozzles located at the nose of the plate.

The plate is equipped with four padeyes, see illustration in Figure B-4. The bridle to the two bottom padeyes are welded plate elements and include a dome at the back of the plate to be able to distribute the applied force from the padeye to the plate. They are fed through the plate and welded to the plate around the circumference of the dome and the forward side at the padeye. A triangular stiff lifting bridle is connected to the two bottom padeyes. Two wire segments connects to the upper two padeyes. In this way the bridle is folded up against the main plate during installation and minimises the penetration resistance. The two upper padeyes are conical holes through the plate body.

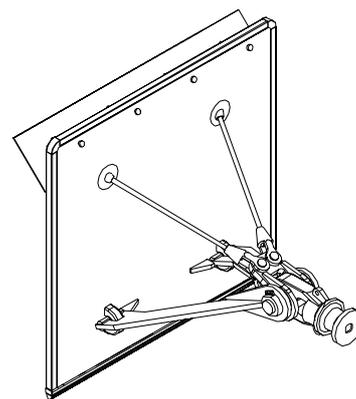


Figure B-4 Illustration of the PADER anchor.

The plate tip is equipped with a manifold and several nozzles where fluid can be circulated from the surface through the drill string to cut loose the soil in front of the plate and reduce the side friction of the plate as it

penetrates the soil. The geometry of the plate tip and the position of the nozzles control the penetration direction of the plate.

The plate has been equipped with a flap on the back of the plate. During penetration the flap will be locked to the plate with the plate connector. Once the connector is released from the plate the flap will spring out. During final rotation of the plate the flap will hinder the plate of moving upwards as a result of the tensioning of the mooring line.

B5 The PPA Anchor System

The PPA (Position Penetrated Anchor) anchor system from Viking Marine Mooring A/S consists of an installation tool and a plate anchor, and is designed for both taut and catenary mooring system. The main function of the Operation & Control System is to operate, control, monitor and record the installation of plate anchors. Figure B-5 shows an illustration of the setup for installation of the plate anchor. The installation tool is lowered to the seabed by means of an A-frame or a crane on the installation vessel, and the reaction force required for embedment of the anchor is provided by four suction anchors, as shown. The anchor is then pushed down hydraulically to the target penetration depth. According to the manufacturer's description, plates varying from 4m² to 60m² in size can be installed.

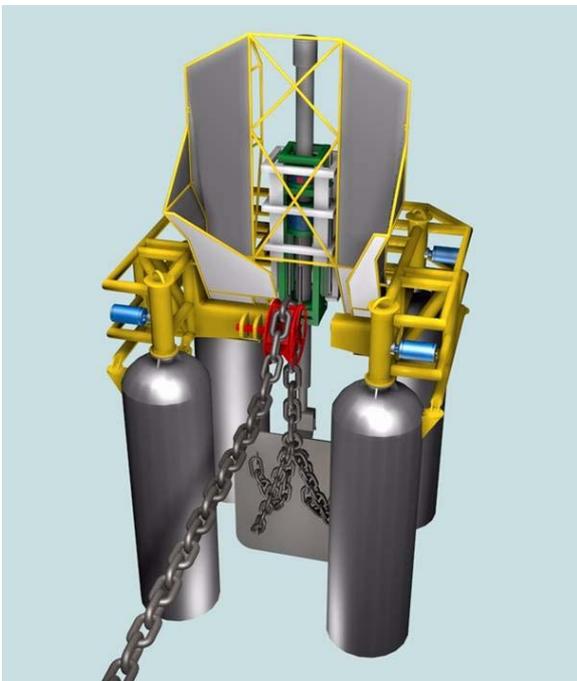


Figure B-5 Installation setup for the PPA anchor.

B6 The BLADE anchor

The BLADE anchor from ABB Anchor Contracting is most beneficial for use as part of a catenary mooring system, where the anchor is subjected to mainly horizontal loads.

The main components of the Blade Anchor are:

- Vertical plate of low penetration resistance, 'blade', stiffened by vertical ribs
- A series of flaps horizontally hinged together and to the vertical plate

The mooring line is connected to the outer flap; thus the mooring load is transferred into the soil partly by the flaps and partly by the vertical plate, see illustration in Figure B-6.

In order to utilize fully the capacity of the anchor, the flaps are designed so that only horizontal loads are exerted onto the vertical plate. Increasing the flap area results in smaller vertical components acting onto the vertical plate and larger capacity of the anchor.



Figure B-6 Illustration of the BLADE anchor.

B7 References

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Appendix C Installation uplift angle of drag-in plate anchors

C1 General

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

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

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

At some point, the length of the seabed part becomes zero and a further increase in the line tension or decrease in distance will result in a situation where the anchor line intersects the seabed under an uplift angle (α), see Figure A-2.

It has been a recognised understanding for some time that both fluke anchors and drag-in plate anchors can be installed with an uplift angle, although only a small number of tests have been performed with a non-zero uplift angle. There is a potential for significant cost savings if a safe installation uplift angle can be documented and agreed upon. In the following, guidelines are given for achieving this.

C2 Assessment of a safe uplift angle

Non-zero uplift angles during installation typically occur when anchors are installed using a short scope of line, either by bollard pull (and blocked line) or by winch pull (from a stationary vessel).

An anchor should under no circumstances be set with an anchor line giving an initial non-zero uplift angle from start of the installation. This would reduce the possibility for the anchor to enter the soil. As a minimum, the embedment of the fluke should be 3 fluke widths (W_F) before uplift is applied. This will also limit the possible maximum uplift angle for all practical means considering the path reaching an ultimate depth. An uplift angle exceeding 10° should not be expected during installation of a drag-in plate anchor according to this procedure, even if the anchor approaches its ultimate depth, z_{ult} .

The penetration path is only slightly affected by the uplift angles following upon the adoption of the installation procedure described above. If the anchor were 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 is negligible.

The anchor line may have either a wire or a chain forerunner, and the effect of using one type of line or the other affects the behaviour of the anchor. An anchor penetrated with a wire will reach a larger ultimate depth than an anchor with a chain, since the soil cutting resistance is less for a wire than for a chain, see sketch in Figure A-2. 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.

The penetration path becomes shallower the higher the uplift angle at the seabed is. The maximum possible uplift angle (α_{max}) is the angle, which makes the anchor drag at a constant depth, and gradually pulls the anchor out of the soil for higher angles. Tentatively, a safe α -angle may be set to 50% of α_{max} , although limited to $\alpha = 10^\circ$. The effect on the installation anchor resistance R_i of increasing the uplift angle from 0° to $\theta/2$ may be assumed to vary linearly according to the following simple expression

$$R_{L,\alpha} = R_{L,\alpha=0} (1 - \alpha / \alpha_{max}) \quad (C-1)$$

(valid for $\alpha < \alpha_{max}/2$ and $\alpha < 10^\circ$)

where R_L is the contribution to the installation anchor resistance R_i from the embedded part of the anchor line.

C3 Short scope and high uplift angle

The relationship between the installation uplift angle and the maximum achievable anchor resistance may be a factor to consider when installing drag-in plate anchors in deep water. The loss in anchor resistance by using a short scope, and thus accepting a reduced penetration depth, can be compensated for by an increase in anchor size. Site-specific prototype anchor tests may be used to optimise the installation procedure, and the anchor may be designed accounting for the short scope installation procedure using the design code described herein.

Appendix D Drag-in plate anchors in layered clay

D1 General

Guidance for assessment of the penetration ability of drag-in plate anchors in layered clay is given in the following. Layering is understood herein as a soil layer sequence comprising a soft layer overlain and/or underlain by a relatively stiffer clay (or sand) layer.

Drag-in plate anchors are particularly suitable for soft normally consolidated clays and their general behaviour is addressed in Appendix A, where Figure A-2 relates to anchors in clay without significant layering.

Experience has shown, however, that drag-in plate anchors 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 width W_F of the actual anchor. Penetration through the upper stiff layer may sometimes require a smaller fluke angle than desirable for penetration through the underlying soft layer. This may be resolved by designing a shear pin, which fails for a shackle load that is sufficient to penetrate the anchor through the surface layer. When this shear pin breaks the fluke angle opens up to an angle, which is found to be suitable for deep embedment into the underlying soft clay layer.

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 tension T_i may not be reached, visualised by continuous drag at constant tension. Designing the anchor for less than ultimate penetration as discussed in Appendix A 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. To consider 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 the required 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 anchor and to avoid unexpected behaviour of the anchor during the installation phase, see further about requirements to soil investigation in Chapter 6 and Appendix J.

Appendix E Installation and testing of drag-in plate anchors

E1 General

Both the offshore and the onshore testing of drag-in plate anchors have focussed a great deal on the performance ratio P_r . The DENLA and STEVMANTA anchors tested under controlled onshore conditions, see /8/ /9/, have given performance ratios in the range $P_r = 1.5 - 2.5$, which agree fairly well with the experience from offshore tests. Higher values have been published and an explanation to this discrepancy is offered below. It is desirable to continue testing of these anchors and other plate anchors, since the database is rather thin, although many tests are of a high quality.

Guidance Note

From an installation point of view a drag-in plate anchor can be compared with a fluke anchor, and it may be tempting to use results from drag-in plate anchor tests to develop charts similar to those found in /E-1/ for fluke anchors. In fact, in the Anchor Manual 2000 from Vryhof Anchors, see /E-2/, a design chart for the STEVMANTA anchor is included. It should be noted, however, that the performance ratio assumed in this chart is $P_r = 3.0$, which is unconservative and not recommended for use until well documented test data can support such high values.

It may well be that one under certain offshore testing conditions can obtain a higher performance ratio than 2.5, but an explanation to this may be found in the offshore testing procedures. The final anchor installation tension T_{min} is normally maintained over a specified period of time, e.g. 15 to 30 minutes, which takes out part (but not all) of the loading rate effect included in the anchor installation resistance. The actual loading rate effect remaining in the line tension during this holding period is weather dependent and increases with the motions of the installation vessel. Also the pullout test is affected by the sea state prevailing during the time of testing. If the weather gets rougher the motions of the installation vessel increases and the tension variation during the pullout test may become quite significant. The tension required for assessment of the performance ratio is the mean value measured during the installation and the pullout test, both values affected by uncertainty, which increases with the sea state during testing.

With the performance ratio P_r being defined as the ratio between the installation pullout resistance $R_{p,i}$ and the target installation resistance T_i it is therefore quite possible to explain the large scatter in P_r from offshore tests.

In order to improve the basis for interpretation of anchor test results it is recommended to take records of the weather conditions during the period of anchor testing and to measure the pullout rate, see further guidance in Section E3.2 and Appendix G.

However, use of design charts as presented in /E-2/ as a basis for final design of drag-in plate anchors is not recommended due to the inherent uncertainties in such charts with reference to the above discussion.

As a general reference to installation procedures normally adopted for both fluke anchors and drag-in plate anchors, as well as descriptions of associated installation equipment, the Anchor Manual from Vryhof Anchors /E-2/ is recommended as a useful supplement to this RP.

All reasonable efforts should therefore be made to ensure that the measurements are reliable and include the most crucial test data for maximum usefulness of the results and improvement of the database. This should be fully appreciated when installing both test anchors and prototype anchors. Tentative

guidelines for monitoring of drag-in plate anchor tests and commercial anchor installations are given in Section E3.

--- End of Guidance Note ---

The performance ratio is useful as a means to assess the necessary size (bollard pull) of the installation vessel(s), but should not be used as a basis for design of drag-in plate anchors.

The most important design parameter is the penetration depth of the anchor, which needs to be verified and compared with the target installation depth after anchor installation, see Chapter 5.

E2 Contributions to the anchor resistance

E2.1 General

In the following, the basic contributions to the resistance of drag-in plate anchors are presented and explained. The parameters involved will be described as they relate to the anchor installation, anchor testing, and anchor performance ratio. For anchor design, reference is made to Chapter 3.

E2.2 Anchor installation

The anchor should be installed by continuous pulling until the minimum installation tension T_{min} has been reached. Stoppage of the anchor installation at a smaller line tension should be avoided, since reconsolidation of the remoulded clay around the anchor during the stoppage period may increase the penetration resistance such that the necessary pulling force to restart the anchor exceeds the capacity of the available installation equipment. The penetration depth of the anchor will then be limited to the depth reached with the installation tension applied before the stoppage. Since the design anchor resistance, see Eq. (3.22), is directly related to the undrained shear strength at the installation depth, the reduced penetration depth will also reduce the reliability of that particular anchor point, such that the specified reliability level cannot be achieved. Measures should be taken to avoid this situation in the planning and execution of the anchor installation.

Figure E-1a) illustrates an anchor installation for the case without uplift ($\alpha_i = 0$, $L_{s,i} > 0$). The installation anchor resistance $R_i(z_i)$ is equal to the target installation line tension $T_i(z_i)$, both referring to the dip-down point and anchor penetration depth z_i . The minimum installation tension T_{min} , which shall be measured and documented as discussed in Section E2.4, has to exceed $T_i(z_i)$ by the seabed friction developed over the length $L_{s,i}$ between the dip-down point and the touch-down point. The anchor is assumed to penetrate to a depth z_i if T_i is mobilised in the dip-down point and held for a specified period of time t_{hold} , i.e.

$$T_i(z_i) = T_{min} - \mu \cdot W_l' \cdot L_{s,i} \quad (\text{E-1})$$

where

- μ = coefficient of seabed friction (also cohesive in nature)
- W_l' = submerged weight of anchor line per unit length

Figure E-1b) illustrates a situation where the anchor line intersects the seabed in the dip-down point under an uplift angle ($\alpha_i > 0$, $L_{s,i} = 0$) during the final stage of anchor installation, leading to $R_i(z_i) = T_i(z_i) = T_{min}$.

The installation anchor resistance $R_i(z_i)$ is thus dependent on the correct assessment of length $L_{s,i}$ and the resulting seabed friction.

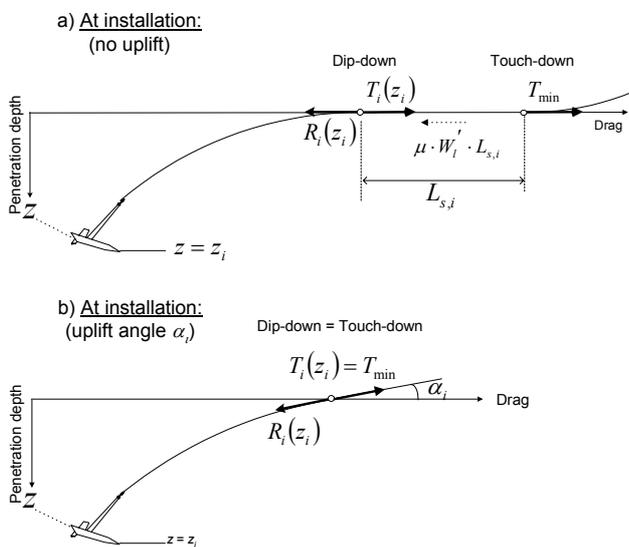


Figure E-1 Installation with and without uplift.

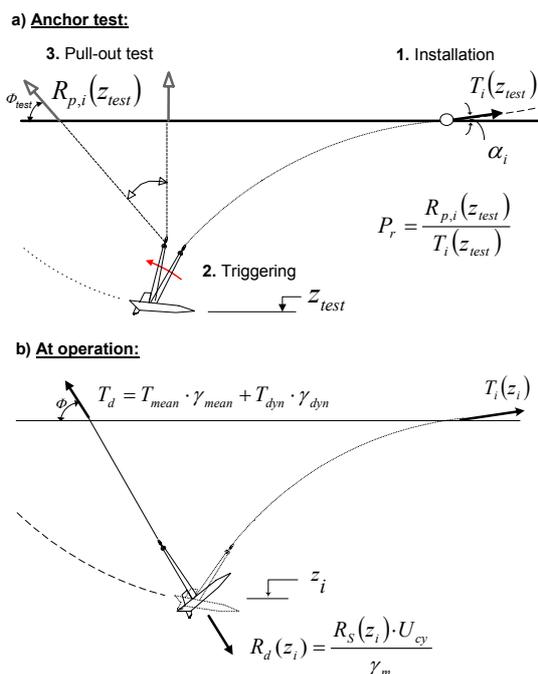


Figure E-2 Anchor test and design.

E2.3 Anchor performance ratio P_r

The assessment of the target installation tension T_i is a crucial design issue, which is directly related to the anchor performance ratio P_r , which is illustrated in Figure E-2a). This ratio can be predicted using recognised geotechnical principles combined with empirical correction factors derived from offshore and onshore field anchor tests, see /8/.

The sketch in Figure E-2a) shows a typical anchor test situation, where the anchor, in this case a Denla anchor, is pulled in to a penetration depth z_{test} applying a line tension $T_i(z_{test})$ and an uplift angle α_i . After triggering and rotation of the anchor, actually reducing the penetration depth somewhat although disregarded in this discussion for clarity, a pullout test is performed, which gives the installation pullout resistance $R_{p,i}(z_{test})$. The performance ratio P_r is then defined as

$$P_r = \frac{R_{p,i}(z_{test})}{T_i(z_{test})} \quad (E-2)$$

Guidance Note

Note in Figure E-2a) that an offshore pullout test may be performed by applying either a vertical or an inclined pulling force in the anchor line. An offshore test is normally carried out with inclined loading in order to reduce the cyclic fluctuations in the line tension due to vessel motions, whereas vertical loading can be used in an onshore test. Anchor installation and installation measurements are discussed in more detail below.

It should be appreciated that both the installation resistance R_i and the installation pullout resistance $R_{p,i}$ when measured in an offshore drag-in plate anchor test, are weather (loading rate) dependent, which in the end will affect the back-calculated performance ratio P_r for the actual test. Loading rate effects on the anchor performance are further below and in Appendix G, see also /8/.

Assume that results as given by Eq. (E-2) are available from an instrumented offshore test, that the undrained shear strength increases linearly with depth, and that the performance ratio P_r derived for depth z_{test} is representative also for depth z_i , then the installation pullout resistance $R_{p,i}$ at depth z_i may be estimated as follows

$$R_{p,i}(z_i) = R_{p,i}(z_{test}) \cdot (z_i / z_{test}) \quad (E-3)$$

The installation pullout resistance measured in an offshore test includes a loading rate effect, which is expressed by a loading rate factor U_r , see Appendix G. By dividing the measured $R_{p,i}(z_{test})$ by this loading rate factor one would obtain the static pullout resistance $R_S(z_{test})$ at the test depth z_{test} .

$$R_S(z_i) = \psi \cdot R_{p,i}(z_{test}) \cdot (z_i / z_{test}) \quad (E-4)$$

where

- ψ = inverse loading rate factor ($=1/U_r$), which accounts for the strain rate effect on s_u , when comparing an offshore pullout test with a static pullout test, see discussion in Appendix G (typically $\psi = 0.8$ for soft clay)

The cyclic loading effect on the static undrained shear strength of soft to lightly overconsolidated clay, and consequently on the static pullout resistance of an anchor in that clay, may be expressed by a cyclic loading factor U_{cy} . This gives the following expression for prediction of the target installation

tension $T_i(z_i)$, which is relying on the anchor performance ratio P_r obtained from site specific anchor tests

$$T_i(z_i) = \psi \cdot U_{cy} \cdot R_{p,i}(z_{test}) \cdot (z_i / z_{test}) / P_r \quad (\text{E-5})$$

where

U_{cy} = cyclic loading factor, see details in Section 3.2.5.3 and Appendix G

On the basis of a careful interpretation of available site specific and/or other relevant anchor tests the performance ratio P_r for design of drag-in plate anchors at the actual site may be assessed. Given the characteristic line tensions for the ULS and ALS conditions, the target installation line tension $T_i(z_i)$ and the minimum installation line tension T_{min} can then be assessed, which can be used for assessment of the necessary bollard pull of the AHV.

E2.4 Minimum installation tension

In the lack of direct measurements of the anchor installation depth, a minimum installation tension T_{min} may be prescribed, based on a conservatively assessed performance ratio P_r . As pointed out above, reaching the minimum installation tension does not necessarily mean that the design requirements have been fulfilled, and the reliability of the performance ratio decreases the more complex the soil profile is, see Appendix D.

Anchor installation should follow procedures, which have been presented and agreed to by all parties well ahead of the installation. T_{min} should be held for a specified holding period, which period may be soil dependent, in clay typically 15 to 30 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 and testing log should document the events and the measurements taken from start to end of the installation.

The possible extra resistance due to seabed friction during anchor installation should be accounted for according to Eq. (E-1), see also Guidance Note in Appendix H.

E3 Installation measurements

E3.1 General

When planning the installation of prototype or test anchors the most essential boundary conditions for the installation must be taken into consideration. Well ahead of the installation, such background information should be compiled and documented.

If practical (e.g. if ROV assistance is available during anchor installation) it is recommended to check the position and orientation of the anchor, as well as the alignment, straightness and length on the seabed of the as laid anchor line, before start of tensioning.

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

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

- line tension
- anchor drag
- plate penetration depth after triggering/rotation

The line tension should be measured as a function of time from start to end of the installation

If manual measurements are taken intermittently, see checklist below, such events should be time-stamped in relation to the line tension log.

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

After installation the triggering of the anchor should be controlled by means as provided for each anchor type. As a final preparation before the anchor is ready for hook-up to the floater the anchor is to be rotated into its normal loading mode. The necessary line tension to achieve this should be assessed based on previous experience with the actual anchor, see also guidance in Appendix I.

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

E3.2 Checklist

1) Before the installation.

- a) Assessment of the most likely soil stratigraphy at the anchor location and the soil strength of significant layers (from soil investigation report), see Chapter 6, Appendix I and Appendix J for guidance.
- b) Specification of the anchor and the installation line configuration.
- c) Specification of the fluke angle(s) to be used, and how this angle is defined, see Appendix A for guidance.
- d) Estimate of friction resistance at the stern roller.
- e) Equipment and procedures for anchor installation, e.g. type and tensioning system of the vessel, method of laying and tensioning of the anchors, availability of ROV, etc.
- f) Type of measurements to be undertaken, and procedures to be applied, from check list below.

2) During the installation.

- a) Line tension (at deck level)¹
- b) Drag (method of measurement, reference point)
- c) Penetration depth (after triggering/ rotation of the anchor)
- d) Vessel speed versus time

¹ The installation tension should be measured as accurately as possible, e.g. by means of the TENTUNE method /E-3/.

- e) Final uplift angle (calculated)

3) Final installation measurements

- a) Maintaining T_{min} (during specified holding period, $t_{hold} = 15$ to 30 minutes)
- b) Measure tension vs. time during holding period (mean tension $\geq T_{min}$)
- c) Drag (corresponding to final penetration depth)

4) Triggering/rotation of anchor

- a) In a one-line installation scenario verify that the anchor has triggered through measurements (and observation) of line tension variation, e.g. a sudden drop in line tension followed by a rapid increase, when the shear pin breaks.
- b) In a two-line installation scenario the triggering is accomplished by changing from the installation line to the mooring line.
- c) After triggering, verify that the anchor has rotated into its normal loading position and estimate approximate line angle θ with the horizontal (see Figure A-2) by measuring and documenting a line tension at deck level at least equal to T_{min} .
- d) Penetration depth of anchor shackle and plate (for final keying load, see Section I3)

5) Offshore pullout test of anchor

- a) Line tension (at deck level) ¹
- b) Penetration depth (at start and end of test)
- c) Line angle with the horizontal during test (method of measurement)
- d) Pullout speed (e.g. vessel or winch speed and line angle at stern roller versus time)
- e) Event log versus time (stoppages should be avoided)

The database for drag-in plate anchors, loaded to their ultimate resistance R_{ult} and ultimate penetration depth z_{ult} , is still very limited, and the test anchors are rather small to match the available pull force. The largest anchors tested in connection with offshore projects have therefore not reached z_{ult} , but for the future it would be fruitful for the industry if the most significant parameters (line tension, drag, final penetration depth, pullout line tension and pullout rate) are recorded during all installations, at least in a few locations out of many.

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

E4 Anchor installation vessels

The bollard pull of the most powerful new generation anchor handling vessels is in the range 3 to 3.5 MN. Depending on the required minimum installation tension T_{min} , one or two AHV's may be required. If feasible, and practically possible, the anchor tensioning can also be done from a special tensioning vessel/barge or from the floater itself. If two opposite anchors are tensioned simultaneously, line tensions up to 5 to 6 MN or even 10 MN can be reached.

The chosen scenario for anchor installation shall ensure that the specified minimum installation tension T_{min} can be reached. The bollard pull, winch capacity and break test load (BTL) of the installation wire on the AHV will have to be assessed on this basis. If T_{min} cannot be reached due to pulling limitations set by the AHV, the design anchor resistance R_d according to Eq. (3.22), and thus the intended reliability level of the anchors, will not be achieved.

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

E5 References

- /E-1/ API Recommended Practice 2SK (1996), *Recommended Practice for Design and Analysis of Stationkeeping Systems for Floating Structures*, 2nd Edition, effective from March 1997.
- /E-2/ Vryhof (1999), *Anchor Manual 2000*, Vryhof Ankers. Krimpen a/d Yssel, The Netherlands.
- /E-3/ Handal, E. and Veland, N. (1998), *Determination of tension in anchor lines*, 7th European Conference on Non-Destructive Testing, Copenhagen, 26-29 May, 1998.

Appendix F Consolidation effect

F1 General

The zone of clay being remoulded during penetration of a drag-in plate anchor will reconsolidate after completion of the anchor installation. The consolidation of this volume of clay will have little or no effect on the normal loading resistance of the anchor, since this involves the resistance of a volume of clay that was not remoulded during the penetration phase. The effect of this consolidation should, however, be considered if the anchor penetration is delayed and the anchor has to be restarted again. The increase in penetration resistance due to such temporary stoppage during installation depends on the anchor characteristics, the soil characteristics, the duration of the stoppage, etc. as discussed in the following.

After completion of the anchor penetration phase the anchor is triggered and prepared for normal loading and hook-up to the floater. This will normally involve a variable degree of rotation of the anchor plate, leading to more significant soil disturbance than during penetration of the anchor. This rotation may therefore affect (reduce) the anchor resistance that can be achieved in a pullout test performed shortly after this rotation took place. In this design procedure the possible increase in the pullout resistance due to consolidation of this partly remoulded clay has been disregarded.

The justification for this is that the mobilisation of the anchor resistance requires a certain amount of anchor displacement, which is associated with a progressive type of failure of the clay. This progressive failure implies that the intact undrained shear strength of the clay cannot be counted on, since a large volume of the soil in the failure zone is already beyond the peak resistance and is partly remoulded when the maximum anchor resistance is recorded.

As mentioned in Appendix I, the amount of rotation required for a drag-in plate anchor to reach the normal loading position may vary depending on the installation scenario, which consequently also affects the amount of remoulding caused by the anchor rotation.

For push-in type plate anchors, the remoulding due to anchor keying and rotation will be equal to, or exceed, that for drag-in plate anchors, since a push-in plate is normally in a vertical position immediately after installation.

F2 Assessment of the effect of consolidation

During penetration of a plate anchor, the sliding resistance will be governed by the remoulded shear strength, $s_{w,r}$, 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 $s_{u,r}$

$$S_t = s_u / s_{u,r} \quad (\text{F-1})$$

The minimum α -value is tentatively set equal to the inverse of the sensitivity, i.e.

$$\alpha_{min} = 1 / S_t \quad (\text{F-2})$$

After an anchor has been installed to a certain installation tension (drag-in type) or depth (push-in type), the remoulded soil will gradually reconsolidate and regain its intact shear strength. As a result, the resistance against further penetration will increase. This effect is in the literature referred to as soaking, set-up or consolidation.

The effect of soil consolidation is that the installation anchor resistance R_i will increase as a function of the time elapsed since installation t_{cons} to a maximum value, which depends on the soil sensitivity S_t . For a particular anchor and depth of penetration this increase may be described through the consolidation factor U_{cons} , which is a function of the soil consolidation characteristics, as well as the geometry, depth and orientation of the anchor.

$$U_{cons} = f(t_{cons}, S_t, \text{etc.}) \quad (\text{F-3})$$

From a geotechnical point of view there should be no major difference between plate anchors and e.g. piles or the skirts of a gravity base structure, when the effects of installation and subsequent reconsolidation on the clay undrained shear strength are considered. The consolidated resistance R_{cons} is the installation resistance with superimposed consolidation effect as shown in Eq. (F-4).

$$R_{cons} = R_i \cdot U_{cons} = R_i + \Delta R_{cons} \quad (\text{F-4})$$

The degree of consolidation that can be applied to the frictional part of the resistance can be assessed by looking at the drainage characteristics in a zone adjacent to the anchor, which is influenced (remoulded) due to the anchor penetration. The width of this zone depends on the anchor geometry and the actual soil characteristics. Guidance for modelling and calculation of the consolidation effect can be obtained using the experience from e.g. tests on piles.

In practice, the anchor installation should be planned such that stoppage before the minimum installation tension T_{min} has been reached is avoided.

Appendix G Cyclic loading effects

G1 General

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 /G-1/. This provides a basis for understanding also how cyclic loading may affect the resistance of drag-in plate anchors.

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 0.5 to 2 hours in a static consolidated undrained triaxial test, and this higher loading (or strain) 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 most direct, and preferred, approach to account for both the loading rate effect and the cyclic degradation effect is to determine the cyclic shear strength $\tau_{f,cy}$ of the clay, following the strain accumulation procedure described in /G-1/.

The strain accumulation method utilises so-called strain-contour diagrammes 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 taut (or catenary) mooring system the loads transmitted to the anchors through the mooring lines will always be in tension, i.e. no shear stress reversal in the soil. The resulting one-way cyclic loading has a less degrading effect on the shear strength than two-way cyclic loading (with shear 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 and low-frequency load cycles, particularly the amplitude of cyclic loads relative to the mean line tension T_{mean} , including the line pretension T_{pre} .

If cyclic soil data, applicable for the actual site, are available, the cyclic shear strength $\tau_{f,cy}$ may be determined according to the procedure outlined in /G-1/. The cyclic shear strength $\tau_{f,cy}$, as defined in /G-1/, 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 defined in Eq. (G-1) below.

$$U_{cy} = \tau_{f,cy}/s_u = f [t_{su}/t_{cy}, \text{ soil data, load history, etc}] \quad (\text{G-1})$$

where

$\tau_{f,cy}$	=	cyclic shear strength with time to failure
t_{cy}	=	(1/4)·(load period)
s_u	=	static undrained shear strength with time to failure $t = t_{su}$
t_{su}		set equal to 1 hour

G2 Loading rate effects

The loading rate plays a role both during installation and pullout of a drag-in plate anchor, which has been demonstrated in the field tests described in /8/. Tests on push-in plate anchors /9/ at the same test site confirm this. Similar results have been reported based on testing of piles.

Important work on the effect of loading rate on axial pile capacity has been published by Bea and Audibert /G-2/, followed by Kraft et al /G-3/, and later by Briaud and Garland /G-4/. The following relationship is suggested in /G-4/ for description of the effect of the loading rate, v , on pile capacity, Q

$$(Q_1/Q_2) = (v_1/v_2)^n \quad (\text{G-2})$$

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

A loading rate factor U_r may be introduced, which expresses the loading rate effect on the resistance of a plate anchor, i.e.

$$U_r = (v_1/v_2)^n \quad (\text{G-3})$$

One practical problem with Eq. (G-3) is to determine representative values for the loading rates v_1 and v_2 at extreme line tension and at end of installation, respectively. Another problem is to assess the value of exponent n in Eq. (G-3).

The effect of loading rate on the installation anchor resistance R_i may be significantly reduced by holding the minimum installation tension T_{min} over a specified period of time t_{hold} while taking measurements as recommended in Appendix E.

The effect of loading rate on the pullout resistance $R_{p,i}$ measured in offshore or onshore anchor tests may vary from one test to another. This has the consequence that the measured pullout resistance of the test anchor includes a certain loading rate effect. Based on the results from the instrumented drag-in plate anchor tests at Onsøy in Norway, as reported in /8/, an approach has been proposed for quantification of the loading rate effect and establishment of a quasi-static pullout resistance R_S . This approach is explained in the following.

G3 Loading rate factor versus strain rate

It has been possible to establish a relationship between results from the anchor tests in clay /8/ and extensive laboratory tests on Drammen clay /G-1/ and Troll clay /G-5/. The following approach was used:

- 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 to the reference (laboratory) strain rate v_{ref} ($=3$ %/hour):
 - failure strain = 3 %, time to failure $t_{su} = 1$ hour
- 3) Relate the peak cyclic shear stress $\tau_{r,cy}$ in the first cycle ($N_{eqv} = 1$) to the actual strain (loading) rate v ($=14,400$ %/hour):
 - 1-way cyclic loading tests, load period $t=10$ sec., i.e.
 - time to failure $= (1/4)$ cycle period = 2.5 sec.
 - axial (permanent) failure strain = 10%
- 4) Determine the loading rate factor U_r as a function of the strain v based on results from steps 2) and 3).
- 5) Apply this experience from triaxial testing to the recorded relationships between anchor pullout resistance and strain rate in the Onsøy tests /8/.

Following this approach it was possible to express the loading rate factor U_r as a function of the strain rate v as shown in Figure G-1.

Since the triaxial test data suggest that the static undrained shear strength varies exponentially with the strain (or loading) rate, it is convenient to relate the loading rate factor U_r to the strain rate as shown in Eq. (G-4).

$$U_r = (v / v_{ref})^n \quad (\text{G-4})$$

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

The exponent derived from the triaxial tests on Drammen clay and Troll clay was 0.040 and 0.041, respectively, when the reference strain rate was set to $v_{ref} = 3$ %/hour. Combining the results from the anchor tests at Onsøy with the criterion that also this line must intersect the static resistance line at a strain rate of 3 %/hour, an exponent $n = 0.054$ is found for the anchor tests, assuming the failure strain to be 5 %.

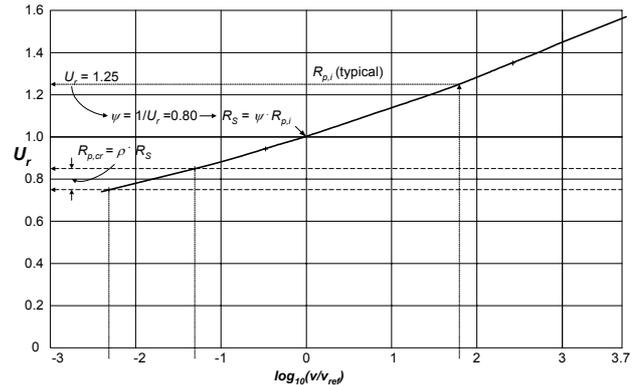


Figure G-1 Loading rate factor versus strain rate v , with $v_{ref} = 3$ %/hour (tentative).

Tentatively it is suggested that a typical offshore pullout test corresponds to a strain rate of about 180 %/hour, which gives $U_r = 1.25$ according to Eq. (G-4), i.e. a 25 % increase in anchor resistance above the static pullout resistance R_S at $v_{ref} = 3$ %/hour.

This means that the measured values of $R_{p,i}$ need to be multiplied by a factor $\Psi = 1/U_r = (1/1.25) = 0.80$ to get the static pullout resistance R_S . Until more test data become available $\Psi = 0.80$ is recommended as a default value, see also Appendix E about guidance for measurements during installation and testing of drag-in plate anchors.

Assuming further that the lines can be extrapolated backwards towards strain rates less than 3 %/hour, a threshold strain rate associated with sustained loading may be established, which would give only negligible anchor creep over the actual operational period. Anchor creep is discussed in Section G4 following.

G4 Assessment of creep pullout resistance

Anchors for deepwater mooring in taut mooring system will be subjected to significant mean line tensions T_{mean} 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 embedded to some 20 to 30 m depth below seabed, creep should not represent a serious threat to the reliability of the mooring system, if the anchors are designed to satisfy the ULS and ALS requirements according to the design code outlined in Chapter 2.

The creep pullout anchor resistance $R_{p,cr}$ is defined such that anchor creep is avoided during the design life of the floater, if the design mean line tension T_{d-mean} , does not exceed $R_{p,cr}$, i.e.

$$T_{d-mean} \leq R_{p,cr} \quad (\text{G-5})$$

With reference to Figure G-1 $R_{p,cr}$ is related to the static anchor pullout resistance R_S through the creep factor ρ

$$R_{p,cr} = \rho \cdot R_S \quad (\text{G-6})$$

The value of the creep factor ρ should reflect the design life of the floater on the actual location, the intensity and duration of the various sea states, the type of clay and its characteristic properties. Tentatively, the creep factor ρ is expected to lie in the range 0.75 to 0.85, which according to Figure G-1 would correspond to a maximum strain rate ν of about 0.015 to 0.15 %/hour.

The loading rate factor U_r may be presented as a function of time to failure T_f as indicated in Figure G-2. The basis for this figure is taken from a comprehensive paper by Berre and Bjerrum /G-7/, where the experience from tests on Drammen clay was presented.

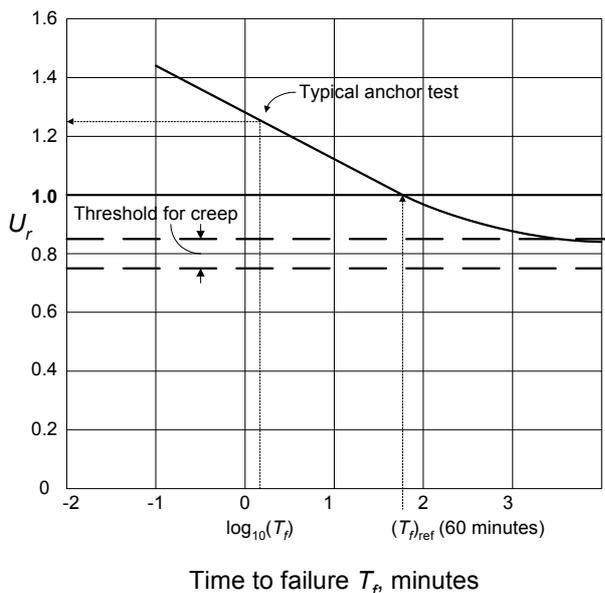


Figure G-2 Loading rate factor U_r versus time to failure T_f , with $(T_f)_{ref} = 60$ minutes.

The curve in Figure G-2 has been adjusted slightly to fit with a time to failure $T_f = 60$ minutes instead of 140 minutes as used in /G-7/. Using the times to failure in the anchor tests and the loading rate factor derived in Figure G-1, a straight-line slope representing the anchor tests has been plotted in Figure G-2. The curved shape for $T_f > 60$ minutes is roughly taken from /G-7/.

G5 Assessment of the cyclic loading effect

The expression for the cyclic resistance R_{cy} of a plate anchor is given in Eq. (3.3).

If no relevant cyclic soil data exist for the site, and experience from better documented sites with similar soil conditions cannot be drawn upon, one must be conservative in the assessment of $\tau_{f,cy}$.

Guidance for assessment of both the loading rate factor U_r and the cyclic loading factor U_{cy} can be found in the published information about cyclic behaviour of clay, e.g. tests on Drammen clay in /G-4/, on Troll clay in /G-5/ and on Marlin clay in /G-6/. It is noted based on the test results presented for the Marlin clay that carbonate content may significantly affect the cyclic response of clay. Caution is therefore warranted in the use of experience from tests on non-carbonate clay, if the actual clay contains more than 10 % carbonate.

Guidance Note

Basis for an approximate assessment of the effect of cyclic loading is provided in the following.

Loading rate factor U_r

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

$$U_r = \tau_{f,cy}/s_u \quad (\text{for } N_{eqv} = 1).$$

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

Figure G-3a) shows the loading rate factor U_r as a function of the average shear stress level τ_a/s_{uD} during the test. It is worth noting that the loading rate effect is most pronounced for average shear stress ratios τ_a/s_{uD} in the range 0.5 to 0.7, and that for higher ratios the effect reduces rapidly (probably a loading rate effect), particularly for the carbonate type Marlin clay (Unit IIb), which has a carbonate content of 15 - 20 % according to /G-6/.

Based on the mooring analysis it will be possible to define the mean, low-frequency and wave-frequency components of the characteristic line tension, such that a basis is obtained for assessment of a likely range for the parameter τ_a/s_{uD} , see 3.2.5.3 for detailed recommendations. Typically the line tension in a taut mooring system generates average shear stress ratios τ_a/s_{uD} in the range 0.5 to 0.8. For this range $U_r = 1.4 - 1.75$ for four of the examples shown in Figure G-3a), but may be as low as 1.2 (or lower) as indicated by the curve for the Marlin carbonate clay.

Cyclic loading factor U_{cy}

Following the strain accumulation procedure as described in detail in /G-4/, and briefly summarised in this appendix, the cyclic test data may be used for prediction of the cyclic loading factor U_{cy} , see 3.2.5.3 for details.

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

Experience has shown that the N_{eqv} may vary significantly from case to case, but the most dramatic effect on the cyclic undrained shear strength $\tau_{f,cy}$ is normally found in the range $N_{eqv} = 1 - 10$. Typically, N_{eqv} lies in the range 10 - 30, so the curves for $N_{eqv} = 10$ in Figure G-3b) and c) may be unconservative in many cases. Therefore, the curves shown in Figure G-3 should only be used as illustrations.

It is recommended to acquire site specific cyclic test data if the effects of cyclic loading are to be accounted for in the design. If such data are not available it is recommended to make conservative assumptions about the cyclic loading effect. By conservative is meant that the strength and plasticity properties of the clay should be evaluated and compared with the database, that the stress history of the soil profile is

assessed, that possible carbonate content is accounted for, etc. When looking at range of U_r and U_{cy} reported for the different clays in Figure G-3 it is evident that experience from testing of one clay will not necessarily be representative of the behaviour of another clay in another geological environment. Unless a site specific cyclic testing programme has been designed and executed, the empirical data like those shown in the figure and elsewhere in the literature should therefore be used with caution. The procedure for calculation of U_{cy} described in 3.2.5 is recommended for use in practice.

As a further background for the results shown in Figure G-3 Table G-1 gives some characteristics of the tested clay.

Other effects

The cyclic laboratory tests behind Figure G-1 were carried out on normally consolidated clay ($OCR = 1-1.5$), but the effect of OCR on the cyclic behaviour for so-called one-way cyclic

loading (no shear stress reversal), which is a relevant assumption when mooring line tension is considered, is moderate. Typically U_r and U_{cy} will be reduced by up to 5 % when OCR increases from 1 to 4, by up to 15 % when OCR increase from 1 to 7 and by 20 % when OCR increases from 1 to 10, see more about effects of OCR in Section 3.2.5.2. The cyclic response will also be affected by the frequency of loading, e.g. low-frequency versus wave-frequency tension components. The low-frequency component has typically a period, which is about 10 times longer than the wave-frequency component represented in the test results plotted in Figure G-1. Recognising the effect of loading rate an increase in the load rise time t_{cy} from 2.5 seconds to 25 seconds, i.e. one log-cycle change, will give a reduction in the net cyclic loading effect by about 10 %, e.g. a reduction from $U_{cy} = 1.3$ to $U_{cy} = 1.27$.

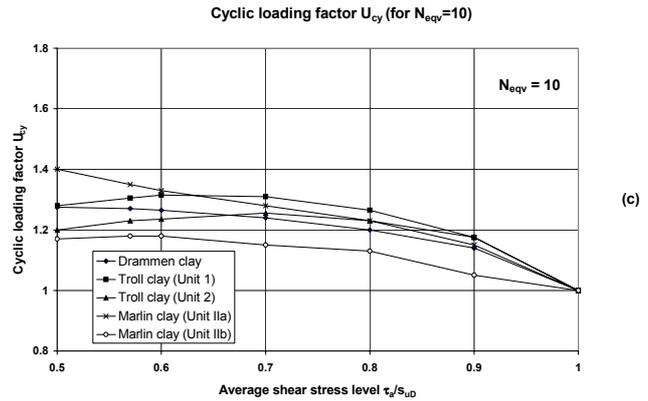
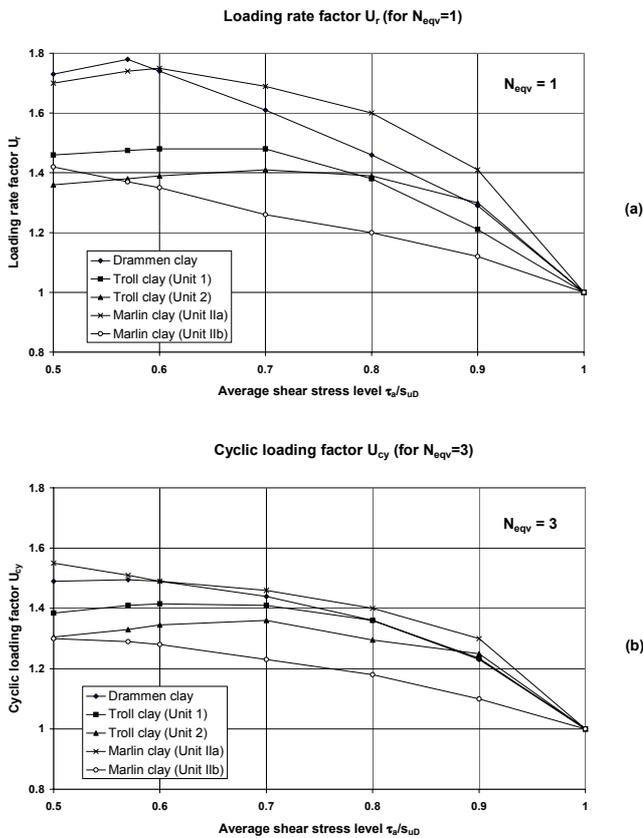


Figure G-3. Example of cyclic direct simple shear test data (from /G-4/, /G-5/ and /G-6/).

Table G-1 Characteristics of tested clay (ref. Figure G-3).

Parameter	Drammen	Troll (Unit 1)	Troll (Unit 2)	Marlin (Unit IIa)	Marlin (Unit IIb)
$s_{u,D}$ [kPa]	8.6	≈20	≈90	≈10	≈30
$z_{lab.test}$ [m]	≈ 50	10 - 20	20 - 60	5 - 15	15 - 35
OCR [-]	1	1.45	1.45	1	1
w [%]	52	47-70	18-26	60-90	40-65
PI [%]	27	37	20	35-60	30-42

Note: Carbonate content in the Marlin clay (Unit IIa) in the range 8-10 %, in Unit IIb in the range 15-20 %.

--- End of Guidance Note ---

+

G6 Anchor pullout resistance v. soil strength

The described approach for assessment of a creep pullout resistance $R_{p,cr}$ may be followed up by using the strain accumulation method to determine the cyclic resistance R_{cy} . To do this, direct simple shear (DSS) cyclic test data are used, but the anchor pullout resistance, with reference to Figure G-3, replaces the undrained shear strength, i.e.

$$\begin{aligned} s_{u,D} &= R_S = \psi \cdot R_{p,i} \\ \tau_{f,cy} &= R_C = U_{cy} \cdot R_S \\ \tau_{cr} &= R_{p,cr} = \rho \cdot R_S \end{aligned}$$

The factors ψ 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} in Eq. (3.9) is equal to the ratio between the cyclic strength $\tau_{f,cy}$ and static undrained shear strength s_u of clay. By writing R_S instead of s_u and R_C instead of $\tau_{f,cy}$ in Eq. (3.9) we may see the relationship between the anchor resistance and the soil strength, see also Figure G-4

Currently $\psi=0.80$ is suggested as a default value for determination of R_S from the installation pullout resistance $R_{p,i}$ measured under offshore testing conditions, see above.

The design mean line tension T_{d-mean} should be less than $R_{p,cr}$ in order to have control of the creep.

G7 References

- /G-1/ Andersen, K. H. and Lauritzen, R. (1988), *Bearing capacity for foundations with cyclic loads*, ASCE Journal of Geotechnical Engineering, Vol. 114, No. 5, May, 1988, pp. 540-555.
- /G-2/ Bea, R.G. and Audibert, J.M.E. (1979), *Performance of dynamically loaded pile foundations*, Proceedings from BOSS'79, Paper No. 68, pp. 728-745. London.
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- /G-4/ Briaud, J-L and Garland, E. (1983), *Loading rate method for pile response in clay*, American Society of Civil Engineers, Vol. 111, No. 3, March 1985, pp. 319-335.
- /G-5/ By, T. and Skomedal, E. (1992), *Soil parameters for foundation design, Troll platform*, Behaviour of Offshore Structures BOSS'92, pp. 909-920.
- /G-6/ Jeanjean, P, Andersen K.H. and Kalsnes B. (1998), *Soil parameters for design of suction caissons for Gulf of Mexico deepwater clays*, Offshore Technology Conference, Paper OTC 8830, pp. 505-519. Houston.
- /G-7/ Berre, T and Bjerrum, L. (1973), *Shear strength of normally consolidated clays*, Proc. 8th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1.1, pp. 75-80. Moscow.

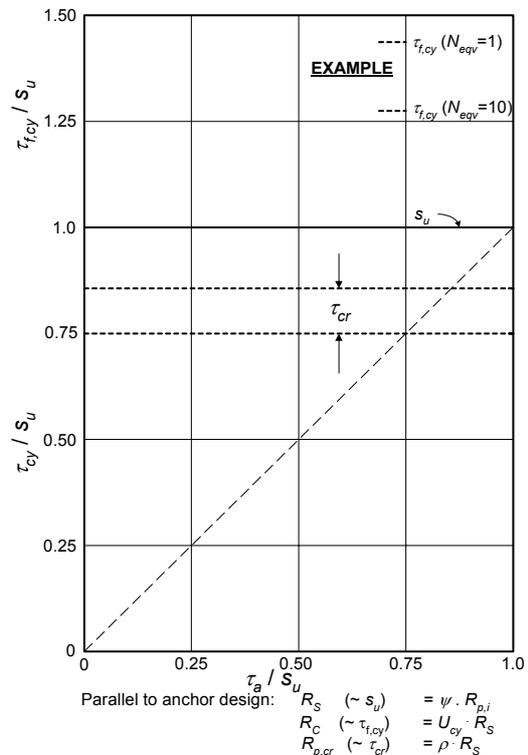


Figure G-4 Anchor pullout resistance v. soil strength.

Appendix H The DIGIN programme

H1 General

An analytical tool for design of drag-in plate anchors should be able to calculate anchor line catenary in soil as well as the force and moment equilibrium of the drag-in plate anchor itself, both in the penetration and in the normal loading mode. Further, the analytical tool should be capable to predict the effect of consolidation on the penetration resistance should there be a stoppage during installation of the anchor and it has to be restarted.

The following section describes in brief the principles for such an analytical tool, called DIGIN, developed by DNV /H-1/. It should be mentioned that the computer programme DIGIN was first developed for design of fluke anchors, and since the penetration of a drag-in plate anchor follows the same principles as the penetration of fluke anchors, the DIGIN programme is equally applicable to drag-in plate anchors. It has been calibrated against full-scale field tests, and the guidelines provided for modelling the anchor, anchor line and soil is based on the experience from the back-fitting analysis of good quality field tests.

The DIGIN programme can be used for prediction of the pullout resistance of drag-in plate anchors, but it is more practical to do this part of the analysis in a spread-sheet programme, following the recommendations in Chapter 3.

H2 Anchor line seabed friction

The resistance due to seabed friction during anchor installation is given by the second term in Eq. (E-1) in Appendix E. The coefficient of seabed friction μ to be used in the prediction of the seabed friction is different for wire and chain.

Guidance Note

Based on the back-fitting analysis of data from measurements on chain segments reported in /H-2/ and estimated values for wire, the following coefficients of seabed friction are recommended for clay¹⁾:

Table H-1 Coefficient of seabed friction			
Wire	Lower bound	Default value	Upper bound
μ	0.1	0.2	0.3
Chain	Lower bound	Default value	Upper bound
μ	0.6	0.7	0.8

The unit friction f along the embedded part of the anchor line as required for calculation of anchor line contribution to the anchor resistance R_i is given by Eq. (H-4).

--- End of Guidance Note ---

H3 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 /H-3/. The normal stress q and the unit soil friction f , which act on an anchor line element in the soil are shown schematically in Figure H-1.

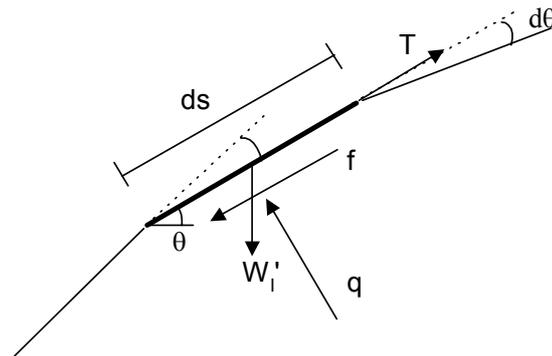


Figure H-1. Soil stresses at an anchor line segment in the soil.

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

$$\frac{dT}{ds} = -f \cdot AS - W_i' \cdot \sin(\theta) \quad (\text{H-1})$$

where

- T = anchor line tension
- θ = orientation of anchor line element ($\theta = 0$ for a horizontal element)
- AS = effective surface of anchor line per unit length of line
- ds = element length

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

$$\frac{d\theta}{ds} = \frac{q \cdot AB - W_i' \cdot \cos(\theta)}{T} \quad (\text{H-2})$$

where

- q = normal stress
- AB = effective bearing area of anchor line per unit length of line

Guidance Note

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

Type of forerunner	AS	AB
Chain	11.3·d	2.5·d
Wire or rope	π·d	d

where

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

-- End of Guidance Note --

The normal stress q on the anchor line is calculated from the following equation:

$$q = N_c \cdot s_u \quad (\text{H-3})$$

where

N_c = bearing capacity factor

s_u = undrained shear strength (direct simple shear strength $s_{u,D}$ is recommended)

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

Guidance Note

Based on the back-fitting analyses reported in /H-2/ and /H-4/ the following bearing capacity factors are recommended for the embedded part of the anchor line in clay¹⁾:

Wire / Chain	Lower bound	Default value	Upper bound
N_c	9	11.5	14

¹⁾ See Table H-2 for values of the effective bearing area AB , which is a pre-requisite for use of the bearing capacity factors given here.

-- End of Guidance Note --

The unit friction f along the anchor line can be calculated from the following formula:

$$f = \alpha_{soil} \cdot s_u \quad (\text{H-4})$$

where

α_{soil} = adhesion factor for anchor line

Guidance Note

Based on the back-fitting analysis of data from measurements on chain segments reported in /H-2/, and estimated values for wire, the following coefficients of seabed friction are recommended for the embedded part of the anchor line clay¹⁾:

Wire	Lower bound	Default value	Upper bound
α_{soil}	0.2	0.3	0.4
Chain	Lower bound	Default value	Upper bound
α_{soil}	0.4	0.5	0.6

¹⁾ See Table H-2 for values of the effective surface area AS , which is a pre-requisite for use of the adhesion factor given here.

--- End of Guidance Note ---

H4 Equilibrium equations 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 mode leading to further anchor penetration in a direction close to the fluke penetration direction, and a second mode leading to reduced or no further penetration. In principle, the soil resistance contributions are the same for the two failure modes, but in the first failure mode the soil resistance normal to the fluke may not take on the ultimate value. Using the symbols shown in Figure H-2 the necessary equilibrium equations are defined and explained in the following.

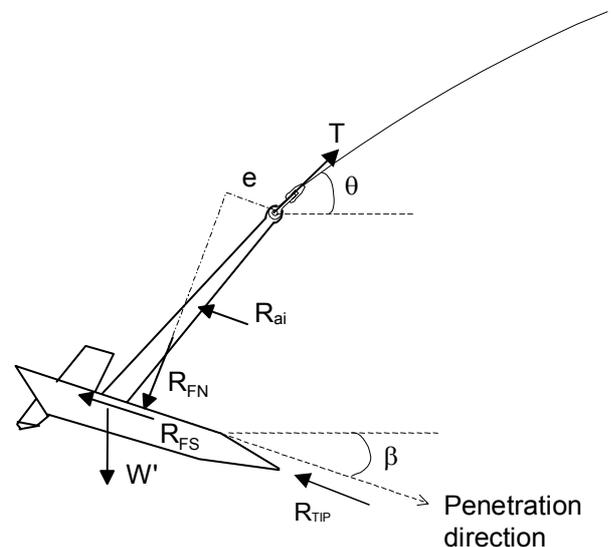


Figure H-2. Principal soil reaction forces on a drag-in plate anchor (penetration direction coincides with fluke penetration direction).

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

Horizontal equilibrium:

$$T \cdot \cos(\theta) = \sum_{i=1}^N R_{ai} \cdot \cos(\beta) + R_{FS} \cdot \cos(\beta) + \quad (\text{H-5})$$

$$R_{TIP} \cdot \cos(\beta) + R_{FN} \cdot \sin(\beta)$$

Vertical equilibrium

$$T \cdot \sin(\theta) = R_{FN} \cdot \cos(\beta) + W'$$

$$\left(\sum_{i=1}^N R_{ai} \cdot \sin(\beta) + R_{FS} \cdot \sin(\beta) + R_{TIP} \cdot \sin(\beta) \right) \quad (\text{H-6})$$

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_{TIP} = tip resistance at the fluke

R_{ai} = soil resistance at the remaining components of the anchor (separated through anchor geometry)

W' = submerged anchor weight

β = penetration direction of fluke

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

$$q = N_c \cdot s_u \quad (\text{H-7})$$

where

N_c = bearing capacity factor

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

$$f = \alpha \cdot s_u \quad (\text{H-8})$$

where

α = adhesion factor for anchor

s_u = undrained shear strength (the direct simple shear strength $s_{u,D}$ is recommended)

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

Guidance Note

Based on the back-fitting analysis performed in the JIP on deepwater anchors /H-2/ the following tentatively values are recommended for the resistance towards the various anchor members in clay:

Table H-5 Bearing and adhesion factors for anchor

Bearing capacity factor N_c ¹⁾			Adhesion factor α	
R_{FN}	R_{ai}	R_{TIP}	R_{TIP}	R_{FS}
12.0 ²⁾	12.0	12.0	$1 / S_t$	$1 / S_t$

¹⁾ Effect of shape, orientation and embedment of the various resistance members on the anchor should be included as relevant.

²⁾ Actual degree of mobilisation of this value as required to satisfy moment equilibrium.

--- End of Guidance Note ---

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

$$\sum_{i=1}^N Rm_{ai} + Rm_{FS} + Rm_{TIP} - (Wm + R_{FN} \cdot e) = 0 \quad (\text{H-9})$$

where

Rm_{FS} = moment contribution from soil sliding resistance at the fluke

Rm_{TIP} = moment contribution from tip resistance at the fluke

Wm = moment contribution from anchor weight

R_{FN} = soil normal resistance at the fluke

e = lever arm between shackle and the line of action of the normal resistance at the fluke

Rm_{ai} = moment contribution from soil resistance at the remaining components of the anchor (separated through anchor geometry).

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

H5 References

- /H-1/ Eklund T and Strøm, P.J. (1998), *DIGIN Users's Manual ver. 5.3*, DNV Report No. 96-3637, Rev. 03, dated 20 April 1998. Høvik
- /H-2/ Eklund T and Strøm, P.J. (1998), *Back-fitting Analysis of Fluke Anchor Tests in Clay*, DNV Report No. 96-3385, Rev. 03, dated 16 September 1997. Høvik
- /H-3/ Vivitrat, V., Valent, P.J., and Ponteiro, A.A (1982), *The Influence of Chain Friction on Anchor Pile Behaviour*, Offshore Technology Conference, Paper OTC 4178. Houston.

/H-4/ Strøm, P.J. and Dahlberg, R. (1998), *Back-fitting Analysis of Drag-in Plate Anchors in Clay*, DNV Report No. 98-3585, Rev. 01, dated 13 January 1999. Høvik

Appendix I Prediction of target penetration depth

I1 General

The target penetration depth, z_t , is the depth that a plate anchor needs to be installed to in order to end up at the calculated, and necessary, penetration depth, z_{calc} . The difference in depth between z_t and z_{calc} is caused by the loss in penetration depth due to keying and rotation, Δz_k , and the failure displacement of the plate, Δz_f , i.e.

$$z_t = z_{calc} + \Delta z_k + \Delta z_f \quad (I-1)$$

for The assessment of the potential loss in penetration depth is an important issue, which needs to be addressed already at the design stage. For all types of plate anchors there is an ultimate penetration depth determined by factors that may be a combination of the type/size of anchor, soil conditions at the site, capacity of the installation spread or structural strength of installation equipment.

The SEPLA, see Section B3, is basically designed for a maximum penetration depth set by the height of the suction follower. For other push-in type plate anchors, like the PADER, see Section B4, the penetration depth, and penetration resistance that can be overcome, may be a function of the installation vessel crane capacity.

The ultimate penetration depth of a drag-in plate anchors is a function mainly of the soil conditions, anchor size and capacity of the installation spread. Of particular significance is the restriction imposed by the maximum installation tension T_i at the dip-down point, either set by the available bollard pull of the installation vessel or by special tensioning devices for cross-tensioning of two anchors, see Section E4.

The loss in penetration depth may be somewhat less for a drag-in plate anchor than for a push-in type, since the plate of a drag-in plate anchor is more horizontal than vertical after completion of the drag-in phase, which means that the necessary rotation to reach a normal loading position is less than for a push-in plate anchor.

On the basis of the above discussion it can be concluded that it is important for the designer to check that the selected type and size of anchor can be installed to the calculated depth plus the predicted loss in penetration depth due to anchor keying/rotation and failure displacement to ensure that the anchor at its final, as-installed depth can fulfil its function as a component in the actual mooring system.

The designer of a plate anchor of any type must understand and account for the behaviour of the actual anchor when assessing the potential loss of penetration depth due to keying and rotation.

Common to all types of plate anchors is the need for displacement to mobilise the resistance. Based on experience, see /8/, the failure displacement, Δz_f , will be significant enough to account for, especially when plate anchors are installed in a layered clay close to a layer boundary. One should also be aware of the loss of anchor resistance as soon as the peak resistance has been exceeded, which is further discussed in Section I5. The failure displacement is normally of significance for the anchor resistance only when plate anchors are installed in layered clay, and even then, if the anchors are properly designed the displacement of the plate due to the applied loads should be much less than the failure displacement.

However, when setting the target penetration depth for plate anchors to be installed to depths less than 1.5 plate widths W_F into a stiff clay underlying a soft clay, a potential overloading of the plate during a storm may in some cases lead to a dramatic reduction in the anchor resistance, which must be avoided.

In normally consolidated clay, the failure displacement has a negligible effect on the anchor resistance.

Finally, the necessary keying load, T_k , must be assessed and agreed upon among the parties involved. This load should be large enough to ensure that the plate behaves as assumed during the life time of the installation. By this is meant that the orientation of the anchor after completed installation and keying should be such that normal loading can be ensured when loads higher than the keying load occur during the operational phase. This is normally achieved through the installation procedures adopted for drag-in plate anchors, but must be paid extra attention when push-in type plate anchors are used.

Tentative guidelines for assessment of the keying distance Δz_k , the keying load T_k and the failure displacement of the plate, Δz_f are given in the following.

I2 Keying distance

Tentatively, the keying distance may be set equal to

$$\Delta z_k = (0.60 \pm 0.20) \cdot W_F \quad (I-2)$$

for push-in type plate anchors equipped with a keying flap. Without a keying flap the keying distance may, in some types of soil, double. Tests have shown that in order to reduce the keying distance, the keying load should not be applied earlier than one hour after completion of the penetration phase. The keying distance is to be added to the predicted anchor penetration depth and will account for the loss in penetration depth due to anchor keying.

Since the plate of a drag-in plate anchor is not vertical after installation, but more horizontal, the loss in penetration depth will be less than given by Eq. (I-2), or even negligible if installed in the direction towards the mooring centre.

The DENLA anchor, see Section B2, installed in its traditional mode, away from the mooring centre, and then triggered and rotated by travelling back towards the mooring centre, will experience somewhat more loss of penetration depth than when installed towards the mooring centre. It is noted that the procedures currently used do no longer require the DENLA anchor to be installed away from the mooring centre, which has been usual so far. If installed in the direction towards the mooring centre, the loss in penetration depth when tensioning the triggered anchor will be small, or negligible. In case of anchor rolling, retrieval of the anchor may impose lateral loading on the anchor shank, which load case may be governing for the structural design of the shank.

STEVMANA anchor, see Section B1, is normally installed in the direction towards the mooring centre, and the final rotation, when the anchor has stopped penetrating after triggering, will often be quite small, leading to negligible or no loss of penetration depth. However, STEVMANTA may also be installed and loaded in different directions, provided that it is structurally competent to take such inclined loads. If installed in the direction away from the mooring centre the STEVMANTA anchor will experience somewhat more loss of penetration depth than when installed towards the mooring centre.

I3 Keying load

Current test data do not give sufficient basis for prescribing the necessary minimum keying load, T_k , to ensure a satisfactory behaviour of the anchor during operation. For a push-in anchor with a keying flap, T_k will probably be less than the peak anchor resistance as recorded in the field tests described in /8/ and /9/. In these tests the pullout resistance included a loading rate effect of about 25% ($U_r = 1.25$) on top of the static anchor resistance R_S .

Based on these tests the necessary keying load, referred to the dip-down point at the seabed, may tentatively be assumed to vary within the range

$$T_k = (0.7 \pm 0.1) \cdot R_S \cdot U_r \quad \text{with } U_r = 1.25 \quad \text{(I-3)}$$

In order to take out part of the loading rate effect, and to obtain further rotation of the plate anchor under constant load, the final keying load should be maintained for a period of 15-20 minutes.

It should be borne in mind that the range of the keying load in Eq. (I-3) is for an effective direction of the load relative to the lever arm, which maximises the moment applied. If the same load is applied in a less effective direction, the lever arm becomes less, which results in a smaller moment for the same load. In order to get the same moment in this case, the load must therefore be increased proportionally.

Drag-in plate anchors installed in the direction towards the mooring centre are already after installation close to the normal loading position, so that application of the keying load may not always have a significant effect on the anchor orientation. This is also the reason for the expected small or negligible loss of penetration depth in this case, see Section I2.

For drag-in plate anchors installed away from the mooring centre, the required keying load will be comparable with that for push-in type plate anchors expressed by Eq. (I-3).

I4 Failure displacement

Tentatively, it is recommended to assume that the failure displacement amounts to 30% of the plate width, i.e.

$$\Delta z_f = (0.3 \pm 0.1) \cdot W_F \quad \text{(I-4)}$$

which should be accounted for in the specification of the target penetration depth of the anchor.

I5 Post-failure resistance

Based on available test data the reduction in the resistance of a plate anchor in a normally consolidated clay when loaded past the failure resistance, may be in the range 20-30% at a post-failure displacement of one anchor plate width W_F . More dramatic reduction in the anchor resistance will occur when the anchor is installed to a limited depth into a stiff clay layer overlain by a softer clay, and there is a significant jump in shear strength at the layer boundary. The actual post-failure reduction in resistance will also depend on the strain softening characteristics of the clay involved.

It should be borne in mind that the objective with the anchor design is to prevent this situation from happening, but if it happens, the anchor has per definition failed.

Appendix J General requirements to soil investigation

J1 Geophysical surveys

The depth of sub-bottom profiling should correspond to the depth of rock or the expected depth of anchor penetration, plus at least the anchor fluke width. The seismic profiles should be tied in to geotechnical borings within the mooring area, which will improve the basis for interpretation of the results from the geophysical survey. Emphasis should be on very high resolution tools for surveying the top 50 metres of sediments.

J2 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 Chapter 6.

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

The challenge to secure soil samples of sufficient quality to determine realistic strength parameters increases with the water depth, and the 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.

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 will 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.

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

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

The T-bar is a tool, which is under development for determination of the in situ undrained shear strength of clay. The T-bar measurements are essentially independent of the water depth at the test site. However, this method is still quite new and is not yet fully qualified for determination of the in situ undrained shear strength in commercial projects, nor by the Classification Societies or the National Authorities.

The soil borings and the in situ tests (e.g. piezocone penetrometer tests, PCPT) should be taken at the anchor locations to a depth exceeding the maximum design penetration of the anchors. Typically, the depth of the borings or PCPTs should exceed the anchor's design penetration by at least three anchor fluke width, accounting for the soil volume involved in the development of the reverse end bearing resistance of a plate.

The number of borings or PCPTs that should be considered depends on the soil variability across the mooring pattern, which may be established by means of sub-bottom profiling. Typically, one boring or PCPT should be taken at each anchor location in cases with lateral variation in soil properties, or at least at two locations over the anchor pattern provided that the sub-bottom profiling shows little variation in soil properties across the pattern. The soil investigation should consider that during the detailed platform and mooring design process, the anchor distances and mooring leg headings may change due to changes in field layout, platform properties and mooring leg properties.

The soil investigation and soil properties interpretation should ideally provide the following information needed for the reliable design of plate anchors for permanent mooring systems, as applicable for the type of anchor, size of anchor and type of anchor loading:

- Definition of soil characteristics, such as general soil description, layering, etc;
- Upper and lower bound undrained anisotropic shear strength properties;
- Unconsolidated, undrained (UU) triaxial shear strength vs depth
- Submerged unit weight;
- Soil stress history and over-consolidation ratio (OCR);
- Soil sensitivity;
- Cyclic shear strength under combined average and cyclic loads for triaxial and direct simple shear (DSS) stress paths;
- Creep data to define loss of strength under sustained load (in cases where large sustained loads, e.g. loop currents, are important). As for above, cyclic stresses should be superimposed on the sustained stresses if relevant for the actual load conditions;

For plate anchor design, most weight should be given to the undrained shear strength derived from DSS and UU triaxial tests. These types of test are considered to give a reasonably representative picture of the intact undrained shear strength of the clay. Clay sensitivity (S_r) is also a significant soil parameter in the plate 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.

With reference to the empirical reduction factor η described in Section 3.2.3, the undrained shear strength tests should also be used to determine the residual undrained shear strength, which indicates the potential loss in strength relative to the peak strength due to strain-softening.

One possibility to acquire soil information, in addition to the data obtained from a standard soil investigation as addressed above, may be to perform a few instrumented anchor tests at the actual location. The objective with these tests would be to obtain data on anchor penetrability, depth-drag relationships, etc as relevant for the design of drag-in plate anchors at the actual location. Regarding considerations related to planning and performance of anchor tests reference is made to Appendix E.

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

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

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

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

If site specific soil data are not provided for assessment of the cyclic loading effect, a conservative assessment of this effect is warranted. For guidance in the planning of a cyclic test programme, and in the assessment of the effects of cyclic loading, see Guidance Note in Appendix G. The examples of DSS cyclic tests shown in Figure G-3 may serve as an indication of how the soil conditions may affect the DSS cyclic shear strength $\tau_{f,cy}$. It is important not to overestimate the effect of cyclic loading in the absence of site specific test data.