RECOMMENDED PRACTICE

DNV-RP-F109

# On-Bottom Stability Design of Submarine Pipelines

OCTOBER 2010

*This document has been amended since the main revision (October 2010), most recently in November 2011.*
*See "Changes" on page 3.*



DET NORSKE VERITAS AS

# FOREWORD

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# CHANGES

## General

As of October 2010 all DNV service documents are primarily published electronically.

In order to ensure a practical transition from the “print” scheme to the “electronic” scheme, all documents having incorporated amendments and corrections more recent than the date of the latest printed issue, have been given the date October 2010.

An overview of DNV service documents, their update status and historical “amendments and corrections” may be found through http://www.dnv.com/resources/rules_standards/.

## Main changes October 2010

Since the previous edition (October 2007), this document has been amended, most recently in April 2009. All changes have been incorporated and a new date (October 2010) has been given as explained under "General".

## Amendments November 2011

The description of Main changes were corrected and the document layout was updated to reflect the current standard.

## ACKNOWLEDGEMENTS

The following companies are gratefully acknowledged for their contributions to this Recommended Practice:

DHI

Hydro

Marintek

PRCI

Statoil

Woodside

DNV is grateful for the valuable cooperation and discussions with the individual personnel representing these companies.

# CONTENTS

# 1. General

## 1.1 Introduction

The present document considers on-bottom stability design for submarine pipelines subjected to wave and current loading. The premises of the document are based on technical development and experience.

The basic principles applied in this document are in agreement with most recognised rules and reflect state-of-the-art industry practice and latest research.

Other data and/or methods than those described herein may be used if a sufficient level of safety can be documented.

## 1.2 Objective

The main objective of this document is to provide rational design criteria and guidance for assessment of pipeline on-bottom stability subjected to wave and current loading.

## 1.3 Relationships to other codes

This document formally supports and complies with the DNV Offshore Standard “Submarine Pipeline Systems”, DNV-OS-F101, 2000 and is considered to be a supplement to relevant National Rules and Regulations.

In case of conflict between requirements of this RP and a referenced DNV Offshore Code, the requirements of the code with the latest revision date shall prevail.

In case of conflict between requirements of this code and a non DNV referenced document, the requirements of this code shall prevail.

**Guidance note:**

Any conflict is intended to be removed in next revision of that document.

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## 1.4 Safety philosophy

The safety philosophy adopted herein complies with section 2 in DNV-OS-F101.

The design of submarine pipelines against excessive displacement due to hydrodynamic loads is ensured by use of a Load and Resistance Factors Design Format (LRFD).

For the absolute stability criterion, the set of safety factors is calibrated to acceptable failure probabilities using reliability-based methods.

For other design criteria, the recommended safety level is based on engineering judgement in order to obtain a safety level equivalent to modern industry practice.

## 1.5 Symbols

### 1.5.1 Latin

$A_p$ Pipe outer area including coating $= \pi \cdot D^2/4$ .
$A_w$ Orbital semi-diameter of water particles $= K \cdot D/2\pi$ .
$b$ Pipe buoyancy per unit length $= \rho_w \cdot g \cdot \pi \cdot D^2/4$ .
$d$ Water depth.
$d_{50}$ Mean grain size.
$D$ Pipe outer diameter including all coating.
$g$ Acceleration of gravity. Should be taken as 9.81m/s$^2$.
$G$ Transfer function.
$G_c$ Soil (clay) strength parameter $= \dfrac{s_u}{D \cdot \gamma_s}$ .
$G_s$ Soil (sand) density parameter $= \dfrac{\gamma_s'}{g \cdot \rho_w}$ .
$F_Y$ Horizontal hydrodynamic (drag and inertia) load.
$F_Z$ Vertical hydrodynamic (lift) load.
$F_R$ Passive soil resistance, Ref. Eq. 3.23.
$F_C$ Vertical contact force between pipe and soil, Ref. Eq. 3.24.
$H_s$ Significant wave height during a sea state.
$H^*$ Maximum wave height during a sea state.

$K_b$ Equivalent sand roughness parameter $= 2.5 \cdot d_{50}$.
$k$ Wave number given by $\frac{\omega^2}{g} = k \cdot \tanh k \cdot d$ .
$k_T$ Ratio between period of single design oscillation and design spectrum $= T^* / T_u$ .
$k_U$ Ratio between oscillatory velocity amplitude of single design oscillation and design spectrum $= U^* / U_s$ .
$k_V$ Ratio between steady velocity component applied with single design oscillation and with design spectrum.
$K$ Significant Keulegan-Carpenter number $= U_s \cdot T_u / D$ .
$K^*$ Keulegan-Carpenter number for single design oscillation $= U^* \cdot T^* / D$ .
$L$ Significant weight parameter $= \frac{w_s}{0.5 \cdot \rho_w \cdot D \cdot U_s^2}$ .
$L^*$ Weight parameter related to single design oscillation $= \frac{w_s}{0.5 \cdot \rho_w \cdot D \cdot (U^* + V^*)^2}$ .
$M$ Steady to oscillatory velocity ratio for design spectrum $V / U_s$ .
$M^*$ Steady to oscillatory velocity ratio for single design oscillation $V^* / U^*$.
$M_n$ Spectral moment of order $n$.
$N$ Spectral acceleration factor $= \frac{U_s}{g \cdot T_u}$ .
$r_{tot}$ Load reduction factor.
$r_{pen}$ Load reduction factor due to penetration.
$r_{tr}$ Load reduction factor due to trench.
$r_{perm}$ Load reduction factor due to a permeable seabed.
$R_D$ Reduction factor due to spectral directionality and spreading.
$s$ Spectral spreading exponent.
$s_g$ Pipe specific density $= (w_s + b) / b$ .
$s_u$ Un-drained clay shear strength.
$s_s$ Relative grain density.
$S_{\eta\eta}$ Wave spectral density
$T_u$ Spectrally derived mean zero up-crossing period $= 2 \cdot \pi \cdot \sqrt{M_0 / M_2}$ .
$T_p$ Peak period for design spectrum.
$T_n$ Reference period $= \sqrt{d / g}$ .
$T^*$ Period associated with single design oscillation.
$U_w$ Wave induced water particle velocity.
$U_s$ Spectrally derived oscillatory velocity (significant amplitude) for design spectrum, perpendicular to pipeline.
$U_{s\theta}$ Spectrally derived oscillatory velocity (significant amplitude) for design spectrum, at an angle $\theta$ to the pipeline.
$U^*$ Oscillatory velocity amplitude for single design oscillation, perpendicular to pipeline.
$V$ Steady current velocity associated with design spectrum, perpendicular to pipeline.
$V^*$ Steady current velocity associated with design oscillation, perpendicular to pipeline.
$w_s$ Pipe submerged weight per unit length.
$y$ Lateral pipe displacement
$Y$ Non-dimensional lateral pipe displacement $= y / D$ .
$z$ Elevation above sea bed.
$z_r$ Reference measurement height over sea bed.
$z_0$ Bottom roughness parameter.
$z_p$ Penetration depth.
$z_t$ Trench depth.

### 1.5.2 Greek

| | |
|---|---|
| $\alpha$ | Generalised Phillips' constant. |
| $\mu$ | Coefficient of friction. |
| $\theta$ | Shields parameter. |
| $\theta_c$ | Angle between current direction and pipe. |
| $\theta_w$ | Angle between wave heading and pipe. |
| $\rho_w$ | Mass density of water, for sea water normally equal to 1 025 kg/m$^3$. |
| $\gamma_{SC}$ | Safety factor. |
| $\gamma_W$ | Safety factor. |
| $\gamma_s$ | Dry unit soil weight. Can be taken as 18 000 N/m$^3$ for clay. |
| $\gamma_s'$ | Submerged unit soil weight. For sand normally in the range 7 000 (very loose) to 13 500 N/m$^3$ (very dense). |
| $\varphi_c$ | Angle of friction, cohesionless soil |
| $\tau$ | Number of oscillations in the design bottom velocity spectrum = $T / T_u$ |
| $\tau_s$ | Shear stress applied from water flow to seabed, Ref. Eq. 4.3. |
| $\omega$ | Wave frequency $= 2\pi / T$ |
| $\omega_p$ | Peak wave frequency $= 2\pi / T_p$ |

# 2. Design

## 2.1 Target failure probability

Excessive lateral displacement due to the action of hydrodynamic loads is considered to be a *serviceability limit state* SLS with the target safety levels given in DNV-OS-F101., Ref. /1/.

If this displacement leads to significant strains and stresses in the pipe itself, these load effects should be dealt with in accordance with e.g. DNV-OS-F101.

## 2.2 Load combinations

The characteristic load condition shall reflect the most probable extreme response over a specified design time period.

For permanent operational conditions and temporary phases with duration in excess of 12 months, a 100-year return period applies, i.e. the characteristic load condition is the load condition with $10^{-2}$ annual exceedance probability. When detailed information about the joint probability of waves and current is not available, this condition may be approximated by the most severe condition among the following two combinations:

1) The 100-year return condition for waves combined with the 10-year return condition for current.
2) The 10-year return condition for waves combined with the 100-year return condition for current.

For a temporary phase with duration less than 12 months but in excess of three days, a 10-year return period for the actual seasonal environmental condition applies. An approximation to this condition is to use the most severe condition among the following two combinations:

1) The seasonal 10-year return condition for waves combined with the seasonal 1-year return condition for seasonal current.
2) The seasonal 1-year return condition for waves combined with the seasonal 10-year return condition for current.

One must make sure that the season covered by the environmental data is sufficient to cover uncertainties in the beginning and ending of the temporary condition, e.g. delays.

For a temporary phase less than three days an extreme load condition may be specified based on reliable weather forecasts.

**Guidance note:**

The term load condition refers to flow velocity close to the seabed. The highest wave induced water particle velocity does normally not correspond to the highest wave and its associated period, but for a slightly smaller wave with a longer period. This effect is more pronounced in deeper waters.

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## 2.3 Weight calculations

Pipe weight should be based on nominal thicknesses of steel wall and coating layers. If metal loss due to corrosion, erosion and/or wear is significant, the wall thickness shall be reduced to compensate for the expected average weight reduction.

Pipe content can be included with its minimum nominal mass density in the relevant condition.

## 2.4 Resistance calculations

Resistance, both the Coulomb friction part and that from passive resistance should be calculated based on nominal pipe weight.

## 2.5 Design criterion

Away from end constraints, the design criterion for lateral stability may be written on a general form as:

$$\frac{Y(L\ ,K,M,N,\tau,G_s,G_c)}{Y_{allowable}} \leq 1.00$$

where $Y_{allowable}$ is the allowed lateral displacement scaled to the pipe diameter. If other limit states, e.g. maximum bending and fatigue, is not investigated, it is recommended to limit the sum of the lateral displacement in the temporary condition and during operation to 10 pipe diameters. When considering the displacement criterion, one should keep in mind that instability in this sense is an accumulated “damage” that may also get contributions for storms that are less severe than the design storm that is normally analysed.

For larger displacements one should perform a full dynamic analysis with adequate analysis tools, or e.g. data bases established by such analyses. Special considerations with respect to bending and fatigue should be made.

The design curves given in Section 3.5 are based on maximum displacement from several dynamic analyses with varying seed value for the random phase shift and can thus be regarded as upper bound values. I.e. no additional safety factors are required. It should be noted that these analyses are one dimensional, neglecting pipe bending – and axial stiffness, and that close to constraints and/or if very large displacements are allowed, two (or three) dimensional analyses may be required.

# 3. Design Methods

## 3.1 Introduction

The purpose of this section is to provide design methods and acceptance criteria for vertical and lateral stability of pipelines.

A design equation is presented for vertical stability, i.e. sinking, in sea water.

Design in order to ensure vertical stability of pipelines resting on the seabed or buried in soil is presented in general terms.

For lateral on-bottom stability, three design methods are presented in detail:

- dynamic lateral stability analysis
- a generalised lateral stability method based on data base results from dynamic analyses/simulations
- an absolute lateral static stability method.

The dynamic lateral stability analysis gives general requirements to a time domain simulation of pipe response, including hydrodynamic loads from an irregular sea-state and soil resistance forces.

The generalised lateral stability method and the absolute lateral static stability method give detailed specific design results for two approaches to stability design.

The generalised lateral stability method is based on an allowable displacement in a design spectrum of oscillatory wave-induced velocities perpendicular to the pipeline at the pipeline level. The design spectrum is characterised by spectrally derived characteristics $U_s$ (oscillatory velocity), $T_u$ (period) and the associated steady current velocity $V$. As a special case a “virtually stable” case is considered whereby the displacement is limited to about one half pipe diameter and is such that it does not reduce the soil resistance and the displacements do not increase no matter how long the sea-state is applied for.

The absolute lateral static stability method is a “design wave” approach, i.e. it ensures absolute static stability for a single design (extreme) wave-induced oscillation. The design oscillation is characterised by oscillatory velocity amplitude $U^*$ and period $T^*$ and the associated steady component $V^*$. Often $V^* = V$, however some hydrodynamic models account for a local mean velocity $V^*$ within a wave-induced oscillation and this may be different to the overall mean velocity $V$.”

## 3.2 Vertical stability in water

In order to avoid floatation in water, the submerged weight of the pipeline shall meet the following criterion:

$$\gamma_W \cdot \frac{b}{w_s + b} = \frac{\gamma_W}{s_g} \leq 1.00 \qquad \textbf{(3.1)}$$

If a sufficiently low probability of negative buoyancy is not documented, the safety factor $\gamma_W$ = 1.1 can be applied.

## 3.3 Vertical stability on and in soil

Pipes that are intended to be buried should be checked for possible sinking or floatation. Sinking should be considered with maximum content density, e.g. water filled, and floatation should be considered with minimum content density, e.g. air filled.

If the specific weight of the pipe is less than that of the soil (including water contents), no further analysis is required to document the safety against sinking. For lines to be placed on or in soils having low shear strength, a consideration of the soil stress may be necessary. If the soil is, or is likely to be, liquefied, the depth of sinking should be limited to a satisfactory value, by consideration of the depth of liquefaction or the build up of resistance during sinking.

If the specific gravity of the pipe is less than that of the soil, the shear strength of the soil should be documented as being sufficient to prevent floatation. Consequently, in soils which are or may be liquefied, the specific weight of the pipe should not be less than that of the soil if burial is required.

Exposed lines resting directly on the seabed should be checked for possible sinking in the same manner as explained above for buried pipes.

## 3.4 Dynamic lateral stability analysis

### 3.4.1 Introduction

The objective of a dynamic lateral stability analysis is to calculate the lateral displacement of a pipeline subjected to hydrodynamic loads from a given combination of waves and current during a design sea state.

The surface wave spectrum must be transformed to a time series for the wave induced particle velocity at the pipe position at the sea-bed. Normally a constant current velocity is added to the wave induced velocity and the hydrodynamic loads are based on the relative velocity and acceleration between the pipe and the total particle velocity.

The resisting force from the soil consists normally of two parts, a pure friction term and a passive resistance term depending on the pipe's depth of penetration into the soil.

The dynamic simulation should be performed for a complete sea state. If no information is available on the duration of sea states, a sea state of three hours is recommended.

On-bottom stability is a highly non-linear phenomenon with a large degree of stick/slip response. This is particularly important to keep in mind for large values of current to wave ratios and large wave periods, and more so for stiff clay and rock than for soft clay and sand where the build up of penetration and passive resistance is more pronounced.

Storm build up may be modelled by applying a linear ramp function on wave induced particle velocity and acceleration so that the load increases from zero to full load during approximately the first 20 per cent of the analysis. This will subject the pipe to moderate waves with small displacement that leads to increased penetration and increased passive resistance.

Very small time increments may be required to accurately capture the highly non-linear stick – slip behaviour of a stability problem.

The application of different phase shift between the harmonic wave components give rise to different time series realisations with varying maximum wave height and sequence of waves that both are important factors for the calculated maximum displacement. Hence, at least seven analyses with randomly, or onerously, chosen seeds to the random number generator should be performed. When the standard deviation in the resulting displacement has stabilised, the mean value plus one standard deviation should be used as design value.

The pipe may be modelled by finite beam elements extending over a part of, or the whole pipeline length. In this case, end conditions may be accounted for.

If end effects are negligible, e.g. for the intermediate part of a long pipeline, the pipe can be modelled by a mass point.

A compressive axial force, due to internal pressure and/or increased temperature will tend to increase the lateral displacement and should be accounted for.

A very heavy pipe will resist the hydrodynamic loads from the largest wave in the design sea state and the criterion for achieving this absolute stability requirement is given in Section 3.6.

A slightly lighter pipe will experience some displacement during the largest waves but instead of being moved a large distance, it will be rugged into a depression, passive soil resistance will be built up and the displacement will not increase with time. This displacement will typically be less than half the pipe diameter, and the corresponding weight parameter is here denoted $L_{stable}$.

An even lighter pipe will regularly be moved out of its depression and can assume that the displacement is proportional with time, i.e. number of waves in the design condition.

Figure 3-1 shows typical results from dynamic on-bottom stability analyses. This figure illustrates well how the response varies with the seed used for the random number generator to the phase shift. E.g. for $L$ = 4, the displacement ranges from 3 to 8 pipe diameters.

The weight corresponding to the point "PROPORTIONAL" in Figure 3.1 is the value above which the displacement becomes proportional to time.

The weight corresponding to the point "STABLE" is the value below which the displacement becomes independent of time.

The weights corresponding to the points "Disp+/-" on individual response curves (i.e. for given seed) are values for which the displacement is the same, within ± 0.01m, for 500 waves as for 1 000 waves (500 followed by the same 500 once more).

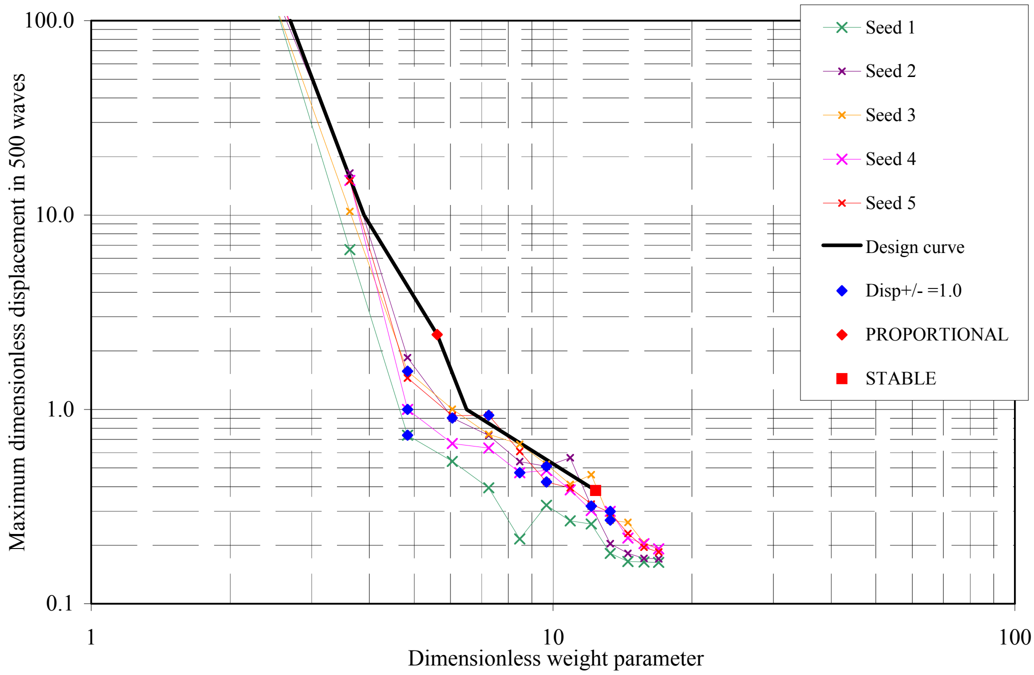


**Figure 3-1**
**Displacement versus weight – results from dynamic analyses**

On-bottom stability may follow one of three distinct approaches:

1) Ensuring absolute stability, Ref. Section 3.6: This approach is based on force equilibrium ensuring that the hydrodynamic loads are less than the soil resistance under a design extreme oscillatory cycle in the sea state considered for design.
2) Ensuring no break-out, Ref. Section 3.5. This approach allows some small displacements under the largest waves in a sea state. However, maximum displacement is small, less than about one half diameter which ensures that the pipe does not move out of its cavity, i.e. the pipe is virtually stable. This approach may take advantage of the build-up of passive resistance during the small displacements that the pipe will experience. There will be no accumulated displacement and maximum displacement can be considered to be independent of time.
3) Allowing accumulated displacement, Ref. Section 3.5. In this approach one specifies a certain, larger, allowable displacement during the sea state considered in design. The pipe will then several times during the sea state break out of its cavity and the calculated displacement should be assumed to be proportional with time, i.e. number of waves in the sea state considered. One should also in this context note that the displacement is an accumulated damage and that a sea state less severe than the one considered in design may also move the pipe, i.e. add to the damage.

### 3.4.2 Current Conditions

The steady current flow at the pipe level may have components from:

— tidal current,
— wind induced current,
— storm surge induced current and
— density driven currents.

The current velocity may be reduced to take account of the effect of the bottom boundary layer and directionality:

$$V(z) = V(z_r) \cdot \frac{\ln(z + z_0) - \ln z_0}{\ln(z_r + z_0) - \ln z_0} \cdot \sin\theta_c \qquad \textbf{(3.2)}$$

**Table 3-1 Seabed roughness**

| *Seabed* | *Grain size $d_{50}$ [mm]* | *Roughness $z_0$ [m]* |
|---|---|---|
| Silt and clay | 0.0625 | $\approx 5 \cdot 10^{-6}$ |
| Fine sand | 0.25 | $\approx 1 \cdot 10^{-5}$ |
| Medium sand | 0.5 | $\approx 4 \cdot 10^{-5}$ |
| Coarse sand | 1.0 | $\approx 1 \cdot 10^{-4}$ |
| Gravel | 4.0 | $\approx 3 \cdot 10^{-4}$ |
| Pebble | 25 | $\approx 2 \cdot 10^{-3}$ |
| Cobble | 125 | $\approx 1 \cdot 10^{-2}$ |
| Boulder | 500 | $\approx 4 \cdot 10^{-2}$ |

For a clayey seabed the seabed roughness parameter of silt should be used.

The mean perpendicular current velocity over a pipe diameter applies:

$$V_c = V_c(z_r) \cdot \left( \frac{\left(1 + \frac{z_0}{D}\right) \cdot \ln\left(\frac{D}{z_0} + 1\right) - 1}{\ln\left(\frac{z_r}{z_0} + 1\right)} \right) \cdot \sin\theta_c \qquad \textbf{(3.3)}$$

Where the directionality of the current velocity is accounted for through $\theta_c$ that is the angle between current velocity and the pipeline axis. If information on directionality is unavailable, the current should be assumed to act perpendicular to the pipeline.

The reference current should be measured at a depth where the mean current vary only slightly in the horizontal direction. On a relatively flat seabed, this reference height should be larger than 1m depending on the seabed roughness.

The non-linear interaction between wave and current flow may be accounted for by modification of the steady current velocity profile.

### 3.4.3 Short term wave conditions

The wave induced oscillatory flow condition at the pipe level may be calculated using numerical or analytical wave theories. The wave theory shall be capable of describing the conditions at the pipe location, including effects due to shallow water, if applicable.

The short-term, stationary, irregular sea states may be described by a wave spectrum $S_{hh}(\omega)$ i.e. the power spectral density function of the sea surface elevation. Wave spectra may be given in table form, as measured spectra, or in an analytical form.

For the JONSWAP spectrum, which is often appropriate, the spectral density function reads:

$$S_{\eta\eta}(\omega) = \alpha \cdot g^2 \cdot \omega^{-5} \cdot \exp\left( -\frac{5}{4}\left(\frac{\omega}{\omega_p}\right)^{-4} \right) \cdot \gamma^{\exp\left(-0.5\left(\frac{\omega-\omega_p}{\sigma \cdot \omega_p}\right)^2\right)} \qquad \textbf{(3.4)}$$

The Generalised Phillips' constant is given by:

$$\alpha = \frac{5}{16} \cdot \frac{H_s^2 \cdot \omega_p^4}{g^2} \cdot (1 - 0.287 \cdot \ln\gamma) \qquad \textbf{(3.5)}$$

The spectral width parameter is given by:

$$\sigma = \begin{cases} 0.07 & \text{if } \omega \le \omega_p \\ 0.09 & \text{else} \end{cases} \qquad \textbf{(3.6)}$$

In lieu of other information, the peak-enhancement factor may be taken as:

$$\gamma = \begin{cases} 5.0 & \varphi \leq 3.6 \\ \exp(5.75 - 1.15\varphi) & 3.6 < \varphi < 5.0; \\ 1.0 & \varphi \geq 5.0 \end{cases} \quad \varphi = \frac{T_p}{\sqrt{H_s}} \qquad \mathbf{(3.7)}$$

The Pierson-Moskowitz spectrum appears for $\gamma$= 1.0.

The JONSWAP spectrum describes wind sea conditions that are reasonable for the most severe sea states. However, moderate and low sea states, not dominated by limited fetch, are often composed of both wind-sea and swell. A two peak (bi-modal) spectrum should be considered to account for swell if considered important. See e.g. Ref. /3/.

The wave induced velocity spectrum at the sea bed $S_{UU}(\omega)$ may be obtained through a spectral transformation of the waves at sea level using a first order wave theory:

$$S_{UU}(\omega) = G^2(\omega) \cdot S_{\eta\eta}(\omega) \qquad \mathbf{(3.8)}$$

The transfer function $G$ transforms sea surface elevation to wave induced flow velocities at sea bed and is given by:

$$G(\omega) = \frac{\omega}{\sinh(k \cdot d)} \qquad \mathbf{(3.9)}$$

where $d$ is the water depth and $k$ is the wave number established by iteration from the transcendental equation:

$$\frac{\omega^2}{g} = k \cdot \tanh(k \cdot d) \qquad \mathbf{(3.10)}$$

The spectral moments of order $n$ is defined as:

$$M_n = \int_0^\infty \omega^n \cdot S_{UU}(\omega) \mathrm{d}\omega \qquad \mathbf{(3.11)}$$

Significant flow velocity amplitude at pipe level is:

$$U_s = 2\sqrt{M_0} \qquad \mathbf{(3.12)}$$

It is not recommended to consider any boundary layer effect on the wave induced velocity.

Mean zero up-crossing period of oscillating flow at pipe level is:

$$T_u = 2\pi \sqrt{\frac{M_0}{M_2}} \qquad \mathbf{(3.13)}$$

Assuming linear wave theory, $U_s$ may be taken from Figure 3-2 and $T_u$ from Figure 3-3 in which:

$$T_n = \sqrt{\frac{d}{g}} \qquad \mathbf{(3.14)}$$

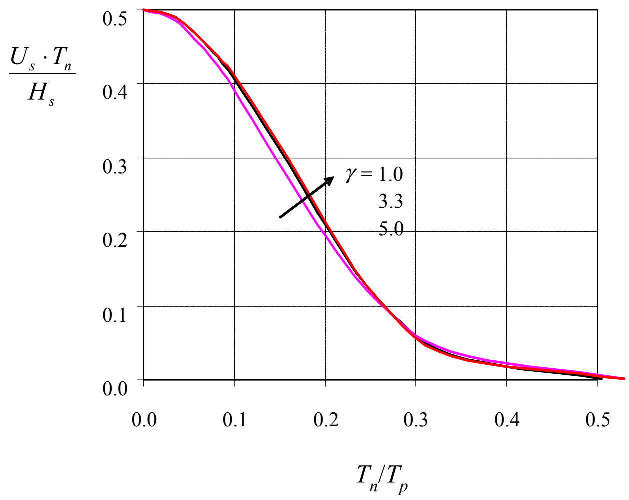


**Figure 3-2**
**Significant flow velocity amplitude $U_s$ at sea bed level**

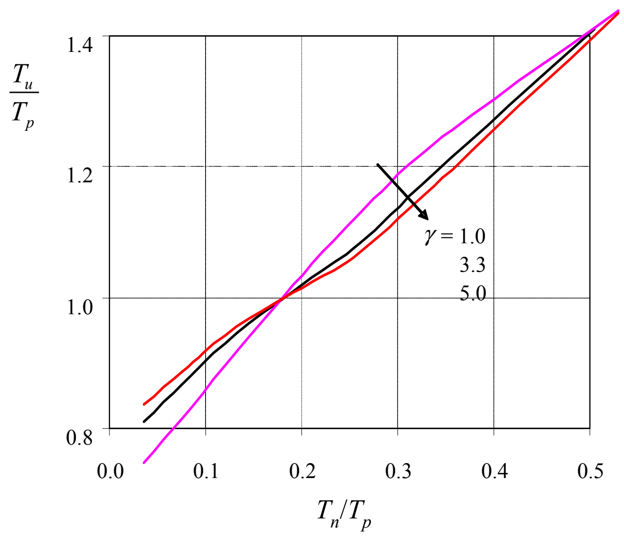


**Figure 3-3**
**Mean zero up-crossing period of oscillating flow $T_u$ at sea bed level**

The ratio between the design single oscillation velocity amplitude and the design spectral velocity amplitude for $\tau$ oscillations is:

$$k_U = \frac{U^*}{U_s} = \frac{1}{2} \cdot \left( \sqrt{2 \cdot \ln \tau} + \frac{0.5772}{\sqrt{2 \cdot \ln \tau}} \right) \quad \textbf{(3.15)}$$

The ratio between design single oscillation velocity period and the average zero up-crossing period (both at seabed level) is site specific. In absence of other data, this can be taken as:

$$k_T = \frac{T^*}{T_u} = \begin{cases} k_t - 5 \cdot (k_t - 1) \cdot T_n / T_u & \text{for} \quad T_n / T_u \leq 0.2 \\ 1 & \text{for} \quad T_n / T_u > 0.2 \end{cases} \quad \textbf{(3.16)}$$

$$k_t = \begin{cases} 1.25 & \text{for} \quad \gamma = 1.0 \\ 1.21 & \text{for} \quad \gamma = 3.3 \\ 1.17 & \text{for} \quad \gamma = 5.0 \end{cases}$$

See Ref. /3/ regarding the applicability of linear wave theory.

### 3.4.4 Wave directionality and spreading

The effect of main wave directionality and wave spreading is introduced in the form of a reduction factor on the significant flow velocity, i.e. projection onto the velocity normal to the pipe and effect of wave spreading.

$$U_w = R_D \cdot U_{w\theta}$$

The reduction factor is given by

$$R_D = \sqrt{\int_{-\pi/2}^{\pi/2} D_w(\theta)\mathrm{d}\theta}$$

where the wave energy spreading directional function is given by a frequency independent cosine power function:

$$D_w = \begin{cases} \dfrac{1}{\sqrt{\pi}} \cdot \dfrac{\Gamma(1+s/2)}{\Gamma(0.5+s/2)} \cdot \cos^s\theta \cdot \sin^2(\theta_w - \theta) & |\theta| < \dfrac{\pi}{2} \\ 0 & \text{else} \end{cases}$$

The angle $\theta_w$ is the angle between wave heading and pipe. $\Gamma$ is the gamma function and $s$ is a site specific spreading parameter.

Normally $s$ is taken between 2 and 8. If no information is available, the most conservative value in the range 2 to 8 shall be selected. A value in the range 6 to 8 may generally be used in the North Sea.

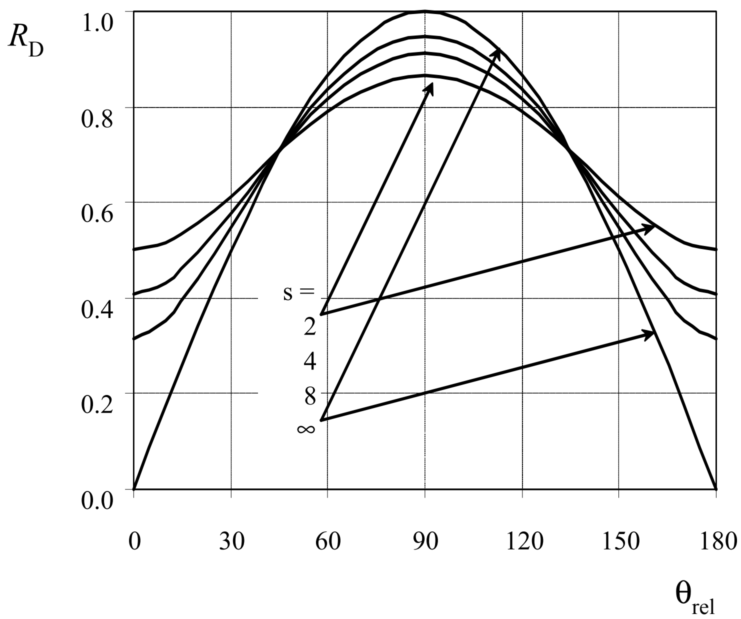


**Figure 3-4**
**Reduction factor due to wave spreading and directionality**

An alternative approach is presented in /16/.

### 3.4.5 Hydrodynamic loads

Experiments have shown that the standard Morison type of force calculations based on ambient flow velocity and with time invariant coefficients have proven inadequate for calculating lateral displacement of pipes due to hydrodynamic loads. This will, for pipes that do experience some displacement, lead to an overestimation of total displacement. The main reasons for this are:

— Even under a regular sinusoidal wave without current, the hydrodynamic forces do not show a regular form as predicted by the Morison type of equations, but a form that is evidently a superposition of harmonic

functions with different frequency and phase.
- Especially the lift force is depending on flow history effects, i.e. the properties of the previous half period wave which is due to the fact that waves produces a wake that is swept back and forth over the pipeline affecting both the magnitude of the forces and their relative phase.
- For the most common design condition where a current velocity is superimposed on an irregular wave velocity, the Morison type of equation yields poor load prediction, especially for the lift force in half periods when the two velocity components oppose each other.
- In addition comes the fact that the force coefficients are highly dependent on the current to wave ratio and the Keulegan-Carpenter number that, in the case of irregular wave, will vary for the individual irregular waves.

Several force models have been developed in order to account for the effects listed above, e.g. Refs. /6/, /7/, /8/ and /9/.

**Load reduction due to pipe soil interaction**

The hydrodynamic loads may be reduced due:

- a permeable seabed $r_{\mathrm{perm,i}}$,
- pipe penetrating the seabed $r_{\mathrm{pen,i}}$ and/or
- trenching $r_{\mathrm{trench,i}}$.

Total load reduction is then:

$$r_{tot,i} = r_{perm,i} \cdot r_{pen,i} \cdot r_{tr,i} \qquad \textbf{(3.17)}$$

The subscript "$i$" takes the value $y$ for the horizontal load and $z$ for the vertical load.

**Load reduction due to permeable seabed**

A permeable seabed will allow flow in the seabed underneath the pipe and thus reduce the vertical load. If the vertical hydrodynamic load used in an analysis are based on load coefficients derived from the assumption of a non-permeable seabed, the following load reduction applies:

$$r_{perm,z} = 0.7 \qquad \textbf{(3.18)}$$

**Load reduction due to penetration**

Load reduction factors due to penetration are in the horizontal and vertical directions, respectively, Ref. /14/:

$$r_{pen,y} = 1.0 - 1.4 \cdot \frac{z_p}{D} \quad \text{however} \quad \geq 0.3 \qquad \textbf{(3.19)}$$

$$r_{pen,z} = 1.0 - 1.3 \cdot \left( \frac{z_p}{D} - 0.1 \right) \quad \text{however} \quad \geq 0.0 \qquad \textbf{(3.20)}$$

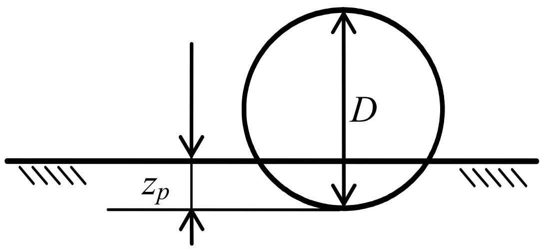


**Figure 3-5**
**Definition of penetration**

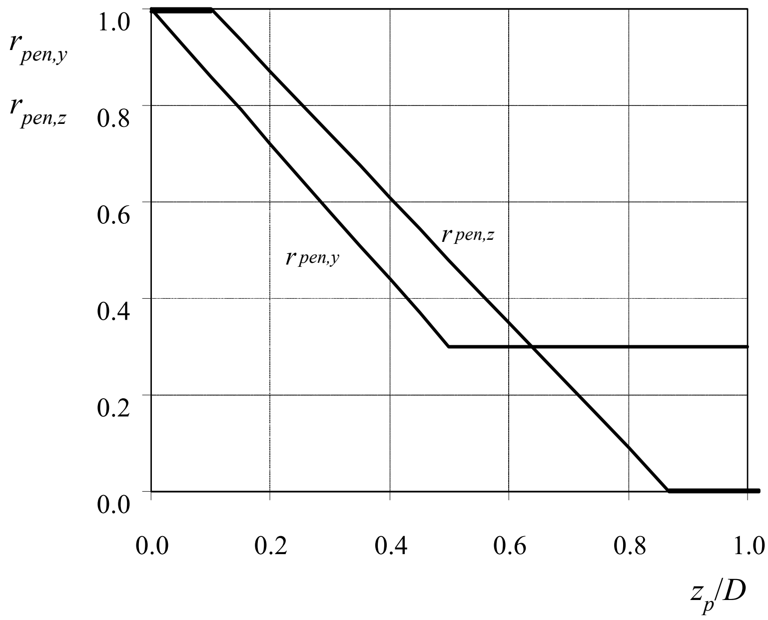


**Figure 3-6**
**Peak load reduction due to penetration**

## Load reduction due to trenching

Load reduction factors due to trenching are in the horizontal and vertical directions, respectively, Ref. /14/:

$$r_{tr,y} = 1.0 - 0.18 \cdot (\theta - 5)^{0.25} \cdot \left(\frac{z_t}{D}\right)^{0.42} \quad , \quad 5 \le \theta \le 45 \qquad \textbf{(3.21)}$$

$$r_{tr,z} = 1.0 - 0.14 \cdot (\theta - 5)^{0.43} \cdot \left(\frac{z_t}{D}\right)^{0.46} \quad , \quad 5 \le \theta \le 45 \qquad \textbf{(3.22)}$$

The trench depth is to be taken relative to the seabed level at a width not greater than $3 \cdot D$ away from the pipe.

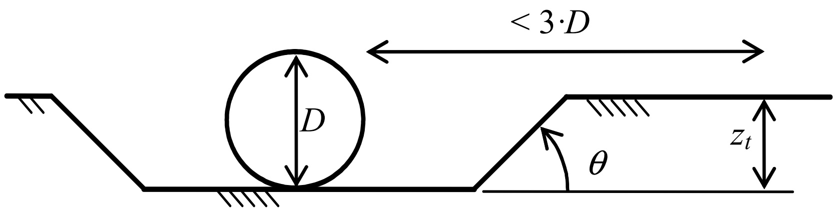


**Figure 3-7**
**Definition of trench parameters**

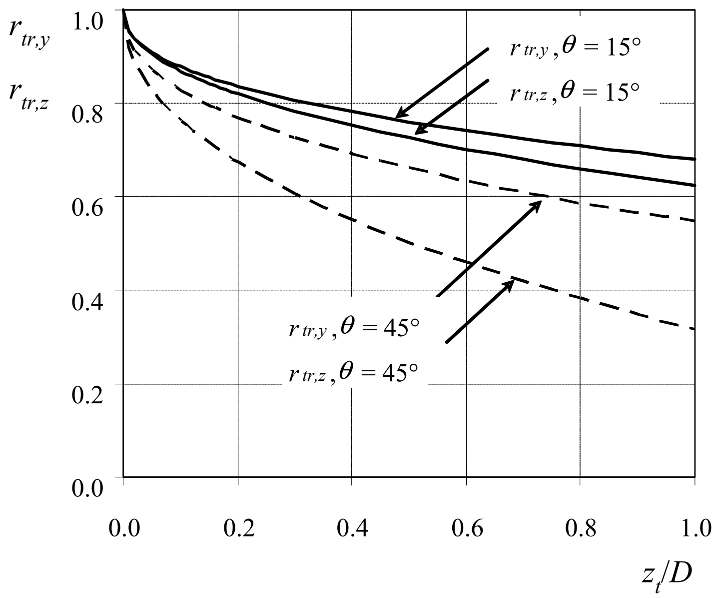


**Figure 3-8**
**Peak load reduction due to trenching**

### 3.4.6 Soil Resistance

Soil resistance consists in general of two parts: a pure Coulomb friction part; and, a passive resistance $F_R$ due to the build up of soil penetration as the pipe moves laterally:

Sand is here defined as a soil that is permeable and with negligible cohesive effects. The most important parameters for describing pipe sand interaction on sand are the coefficient friction and submerged sand weight.

Special considerations should be made if the sand contains a high fraction of calcium carbonate, Ref. Appendix B.

Clay is here defined as a soil that is not permeable and with significant cohesive effects.

Rock is here defined as crushed rocks with a 50 per cent diameter fractile larger than 50 mm.

The coefficient of friction $\mu$ can normally, for a concrete coated pipe, be taken as 0.6 on sand, 0.2 on clay and 0.6 on rock.

A model for passive resistance on sand and clay is described below whereas this effect should be neglected on rock.

A typical model for passive soil resistance consists of four distinct regions:

1) An elastic region where the lateral displacement is less than typically 2% of the pipe diameter.
2) A region where significant displacement may be experienced, up to half the pipe diameter for sand and clay soils in which the pipe soil interaction causes an increase in the penetration and thus in the passive soil resistance.
3) After break-out where the resistance and penetration decrease.
4) When the displacement exceeds typically one pipe diameter, the passive resistance and penetration may be assumed constant.

Reference is made to Refs. /2/, /3/, /4/ and /5/ for further details and other models.

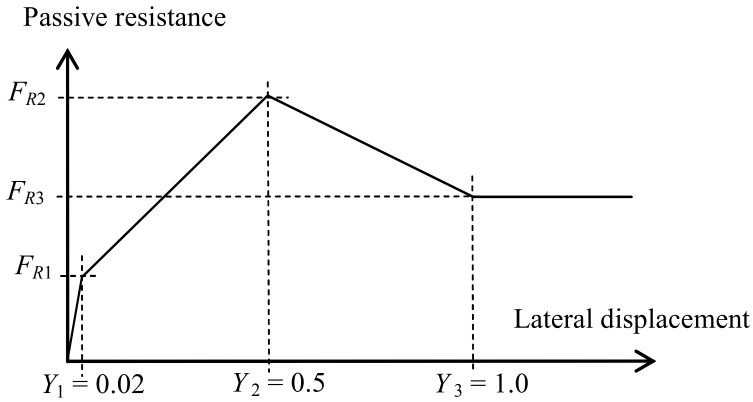


**Figure 3-9**
**Passive resistance**

In the elastic region, $Y \leq Y_1$, the stiffness $k$ can be taken as 50-100 N/m for sand and 20-40 N/m for clay. The stiffness increases with sand density and clay shear strength. No work is done and penetration is constant and equal to the initial penetration.

In the region $Y_1 < Y \leq Y_2$, the pipe soil interaction creates work which again increases the penetration and thus the passive resistance.

Note that the value of the break-out resistance $F_{R2}$ cannot be computed a priori as it is dependent on the accumulated pipe displacement in the region between $Y_1$ and $Y_2$.

If the displacement exceeds $Y_2$, the pipe is assumed to break out. The accumulated work is set to zero and no work is done in this region. Penetration is reduced linearly from the break out value at $Y_2$ to half this value at $Y = Y_3$ and passive resistance is reduced accordingly.

For a displacement larger than $Y_3$, penetration and passive resistance can be assumed to be constant. No work is done.

On displacement reversal in the elastic regime, the force displacement follows linear elastic line at constant penetration.

On displacement reversal outside the elastic regime, from the point $Y$, $F_R$ the force displacement can be calculated according to a curve corresponding to that the origin has shifted a distance $Y - F_R/k$ and the initial penetration is set equal to the current penetration.

Passive resistance $F_R$ on sand can be taken as

$$\frac{F_R}{F_C} = \begin{cases} \left(5.0 \cdot \kappa_s - 0.15 \cdot \kappa_s^2\right) \cdot \left(\frac{z_p}{D}\right)^{1.25} & \text{if } \kappa_s \leq 26.7 \\ \kappa_s \cdot \left(\frac{z_p}{D}\right)^{1.25} & \text{if } \kappa_s > 26.7 \end{cases} \quad \textbf{(3.23)}$$

$$\kappa_s = \frac{\gamma_s' \cdot D^2}{w_s - F_Z} = \frac{\gamma_s' \cdot D^2}{F_C} \quad , \quad F_C = w_s - F_Z \quad \textbf{(3.24)}$$

Passive resistance on clay can be taken as

$$\frac{F_R}{F_C} = \frac{4.1 \cdot \kappa_c}{G_C^{0.39}} \cdot \left(\frac{z_p}{D}\right)^{1.31} \quad \textbf{(3.25)}$$

$$G_C = \frac{s_u}{D \cdot \gamma_s} \quad \text{and} \quad \kappa_c = \frac{s_u \cdot D}{w_s - F_Z} = \frac{s_u \cdot D}{F_C} \quad \textbf{(3.26)}$$

It must be documented that the soil parameters used in the calculation of passive resistance are valid within the actual pipe penetration.

Total penetration can be taken as the sum of initial penetration and penetration due to pipe movement:

$$z_p = z_{pi} + z_{pm} \qquad \textbf{(3.27)}$$

Initial penetration on sand can be taken as:

$$\frac{z_{pi}}{D} = 0.037 \cdot \kappa_s^{-0.67} \qquad \textbf{(3.28)}$$

Initial penetration on clay can be taken as:

$$\frac{z_{pi}}{D} = 0.0071 \cdot \left(\frac{G_c^{0.3}}{\kappa_c}\right)^{3.2} + 0.062 \cdot \left(\frac{G_c^{0.3}}{\kappa_c}\right)^{0.7} \qquad \textbf{(3.29)}$$

It must be documented that the soil parameters used in the calculation of passive resistance are valid within the actual pipe penetration.

Total penetration can be taken as the sum of

— initial penetration due to self weight
— piping
— penetration due dynamics during laying and
— penetration due to pipe movement under the action of waves and current.

The phenomenon of general seabed stability and soil liquefaction are briefly described in Chapter 4.

Maximum pipe weight (e.g. water filled during the system pressure test) and zero lift force can be assumed in the calculation of $\kappa_s$ and $\kappa_c$ for initial penetration on sand and clay, respectively.

For a pipe lying in a trench, the resistance from the trench wall may be accounted for through an equivalent penetration equal to:

$$\frac{z_{pt}}{D} = \frac{1}{2} \cdot \tan\theta \quad \text{however} \quad \leq \frac{z_t}{2} \qquad \textbf{(3.30)}$$

which corresponds to a situation where the pipe has moved one half diameter into trench wall with a trench angle equal to $\theta$, but limited to half the depth of the trench depth $z_t$.

**Piping**

Piping is here defined as a phenomenon where a layer of sand is moved from its position immediately under the pipeline due to the hydrodynamic pressure difference on each side of the pipeline. This will lower the pipe and the increased resistance may be accounted for in stability design.

The pressure difference $p_1 - p_2$ is here taken conservatively as the horizontal load acting on the pipeline divided by the exposed area, whereas the resisting pressure is depending on the sand weight, its friction angle and penetrations depth, Ref. /13/. Piping will occur if the driving pressure gradient is greater than the resisting, i.e.:

$$\frac{F_Y}{\left(D - z_{pi} - z_{pp}\right)} \geq \gamma_s \cdot \frac{\alpha \cdot D \cdot \tan\varphi_s}{\cos\alpha + \sin\alpha \cdot \tan\varphi_s}$$

$$\cos\alpha = 1 - \frac{2 \cdot z_p}{D} \quad , \quad \frac{z_p}{D} \leq 0.20 \qquad \textbf{(3.31)}$$

The friction angle $\varphi_s$ may vary from 30° for very loose sand up to 43° for very dense, cemented sand.

In order to ensure with a sufficient probability that piping occurs prior to the design sea state, the horizontal load used in Eq. (3.31) shall correspond to a return period no more than one tenth of the design sea state.

It is required to document that the sand properties used in Eq. (3.31) are actually valid down to the calculated penetration due to piping. E.g. a clay layer will prevent piping.

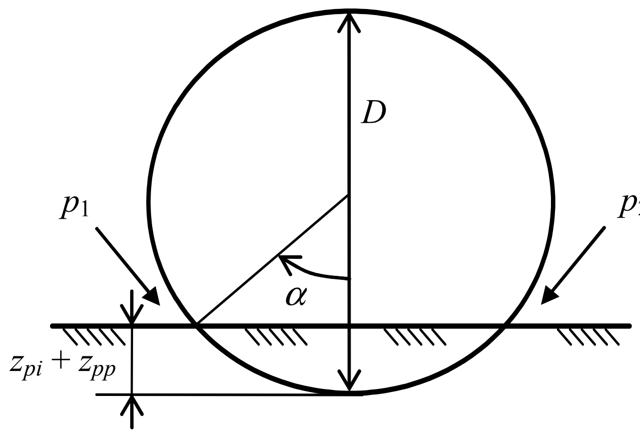


**Figure 3-10**
**Piping parameters**

A pipeline will experience several less sever sea states prior to its design condition that may not make the pipe break out of its penetration, but may rock the pipe back and forth hence increasing the penetration and passive resistance. This additional penetration can be accounted for as initial penetration when the design sea state is analysed. To quantify this penetration one may analyse the pipe for an environment with a return period not larger than one tenth of the design conditions.

## 3.5 Generalized lateral stability method

### 3.5.1 Introduction

It may be shown, see e.g. Ref. /12/, that the dimensionless lateral pipe displacement $Y$ is to a large extend governed by a set of non-dimensional parameters:

$$Y = f(L, K, M, N, \tau, G_s, G_c) \tag{3.32}$$

The non-dimensional parameters are all defined in Section 1.

Since there are a relatively limited number of significant input parameters, the on-bottom stability problem is well suited for establishing databases in which the displacement is given for its set of input parameters.

One can take advantage of a large reduction in weight requirement by allowing some displacement which would be limited to a value that most pipelines can experience without problems with e.g. large strains.

The dynamic analyses that the given weights are based on a flat seabed neglecting effect from axial forces due to e.g. elevated temperature, pressure and restraints at pipe ends.

### 3.5.2 Design curves

This section provides design curves for on-bottom stability design with an allowed lateral displacement in the range from less than half a pipe diameter, i.e. for a virtually stable pipe, up to a significant displacement of 10 diameters during the given sea state. The weight required for obtaining a virtually stable pipe is here denoted $L_{stable}$ whereas the weight required for obtaining a 10 pipe diameter displacement is denoted $L_{10}$. The curves are obtained from a large number of one dimensional dynamic analyses, i.e. on flat seabed and neglecting bending and axial deformation of the pipe.

One should note that all cases with high values of $N$, $K$ and $M$ do not necessarily represent realistic physical conditions. The given values are not valid for extreme cases requiring a pipe specific weight $s_g$ larger than 3. Neither should this method be used for $s_g < 1.05$. The specific weight of a pipe is given by:

$$s_g = 1 + \frac{2}{\pi} \cdot N \cdot K \cdot L \tag{3.33}$$

At deep waters, $K$ may be very small whereas the presence of current gives a large value of $M$. In such cases it is recommended to require absolute stability according to Section 3.6.

$L_{stable}$ is independent of sea state duration whereas $L_{10}$ is valid for 1 000 waves and can be assumed to be proportional to the number of waves $\tau$ in the sea state. If $L < L_{stable}$, then displacement should conservatively be regarded as varying linearly with number of waves in the sea state:

$$Y_\tau = 0.5 + (10 - 0.5) \cdot \frac{\tau}{1000} = 0.5 + 0.0095 \cdot \tau \qquad \textbf{(3.34)}$$

E.g. a three hour sea state with $T_u > 10.8$ s will expose the pipe to less than 1 000 waves, and the expected displacement can be scaled down accordingly.

Linear interpolation can be performed in $M$ and $K$.

Required weight for an intermediate displacement criterion can be calculated according to the following formula:

$$\log L_Y = \log L_{stable} + \frac{\log(L_{stable5} / L_{10})}{\log(0.5/(0.01 \cdot \tau))} \cdot \log(Y/0.5) \qquad \textbf{(3.35)}$$

This design approach is applicable to $N \leq 0.024$ for clay and $N \leq 0.048$ for sand.

Interpolation can be performed in $G_c$ for clay assuming $L$ to be proportional with $\sqrt{G_c}$. (The effect of varying soil density for pipes on sand has been neglected.) Note that the curves are valid for $G_c \leq 2.78$ only. For higher values of $G_c$ it is recommended to require absolute stability.

Minimum pipe weight required to obtain a virtually stable pipe can found from the following design points independent of $N$:

**Table 3-2 Minimum weight, $L_{stable}/(2 + M)^2$, for pipe on sand, $K \geq 10$**

| *M* \ *K* | *10* | *15* | *20* | *30* | *40* | *≥ 60* |
|---|---|---|---|---|---|---|
| ≤ 0.2 | 1.50 | 1.42 | 1.35 | 1.25 | 1.22 | 1.22 |
| 0.4 | 1.82 | 1.70 | 1.61 | 1.53 | 1.50 | 1.50 |
| 0.5 | 2.19 | 1.97 | 1.83 | 1.69 | 1.61 | 1.61 |
| 0.6 | 2.65 | 2.35 | 2.18 | 1.99 | 1.85 | 1.72 |
| 0.8 | 3.05 | 2.55 | 2.32 | 2.13 | 2.01 | 1.90 |
| 1.0 | 3.05 | 2.55 | 2.40 | 2.20 | 2.06 | 1.95 |
| 1.5 | 2.65 | 2.45 | 2.36 | 2.24 | 2.11 | 2.09 |
| 2.0 | 2.50 | 2.40 | 2.35 | 2.27 | 2.22 | 2.19 |
| 4.0 | 2.45 | 2.40 | 2.39 | 2.37 | 2.37 | 2.37 |
| ≥ 10 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 |

For $K \leq 5$, the required weight is more dependant on $N$ and minimum pipe weight required to obtain a virtually stable pipe can found from the following design points:

**Table 3-3 Minimum weight, $L_{stable}/(2 + M)^2$, for pipe on sand, $K \leq 5$**

| *M* \ *N* | *0.003* | *0.006* | *0.012* | *0.024* | *0.048* |
|---|---|---|---|---|---|
| ≤ 0.2 | 1.55 | 1.45 | 1.34 | 1.24 | 1.13 |
| 0.4 | 2.00 | 1.65 | 1.34 | 1.24 | 1.13 |
| 0.5 | 3.30 | 2.60 | 1.91 | 1.24 | 1.13 |
| 0.6 | 3.75 | 3.07 | 2.38 | 1.70 | 1.13 |
| 0.8 | 4.00 | 3.45 | 2.90 | 2.36 | 1.81 |
| 1.0 | 3.90 | 3.50 | 3.10 | 2.71 | 2.31 |
| 1.5 | 3.25 | 3.13 | 3.00 | 2.88 | 2.75 |
| 2.0 | 2.75 | 2.75 | 2.75 | 2.75 | 2.75 |
| 4.0 | 2.60 | 2.60 | 2.60 | 2.60 | 2.60 |
| ≥ 10 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 |

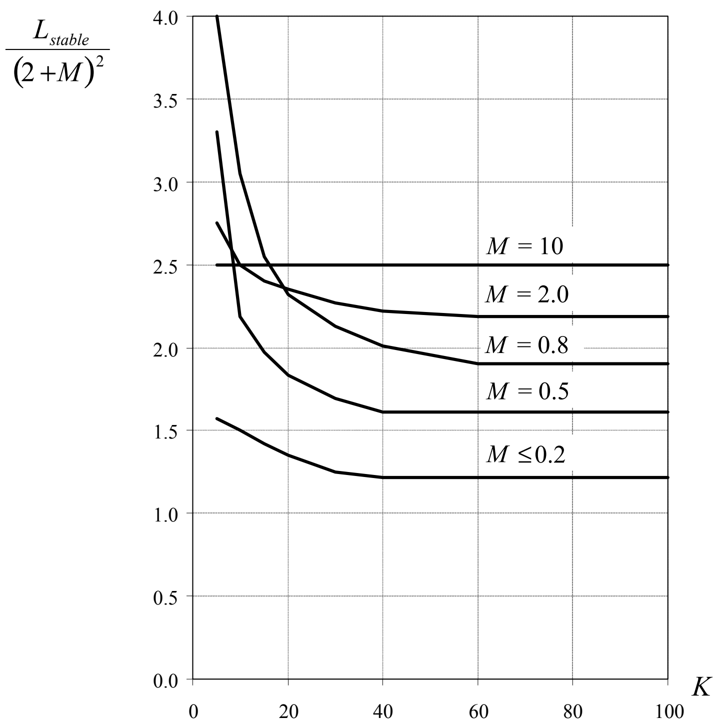


**Figure 3-11**
**Minimum weight, $L_{stable}/(2+M)^2$, for pipe on sand**

Minimum pipe weight required to limit the lateral displacement to 10 pipe diameters for pipes on sand can found from the following design points:

**Table 3-4 Minimum weight, $L_{10}/(2+M)^2$, for pipe on sand**

| *K* / *M* | *= 5* | *10* | *15* | *20* | *30* | *40* | *60* | *≥ 100* |
|---|---|---|---|---|---|---|---|---|
| ≤ 0.2 | 0.20 | 0.41 | 0.61 | 0.81 | 0.69 | 0.69 | 0.69 | 0.69 |
| 0.4 | 0.31 | 0.62 | 0.93 | 0.81 | 0.75 | 0.72 | 0.70 | 0.70 |
| 0.5 | 0.34 | 0.69 | 1.03 | 0.93 | 0.83 | 0.78 | 0.75 | 1.00 |
| 0.6 | 0.79 | 1.20 | 1.13 | 1.10 | 1.07 | 1.05 | 1.03 | 1.02 |
| 0.8 | 0.85 | 1.40 | 1.37 | 1.35 | 1.33 | 1.33 | 1.32 | 1.31 |
| 1.0 | 1.60 | 1.50 | 1.47 | 1.45 | 1.43 | 1.43 | 1.42 | 1.41 |
| 1.5 | 1.80 | 1.70 | 1.67 | 1.65 | 1.63 | 1.63 | 1.62 | 1.61 |
| 2.0 | 1.90 | 1.80 | 1.77 | 1.75 | 1.73 | 1.73 | 1.72 | 1.71 |
| 4.0 | 2.10 | 2.00 | 1.97 | 1.95 | 1.93 | 1.93 | 1.92 | 1.91 |
| ≥ 10 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 | 2.50 |

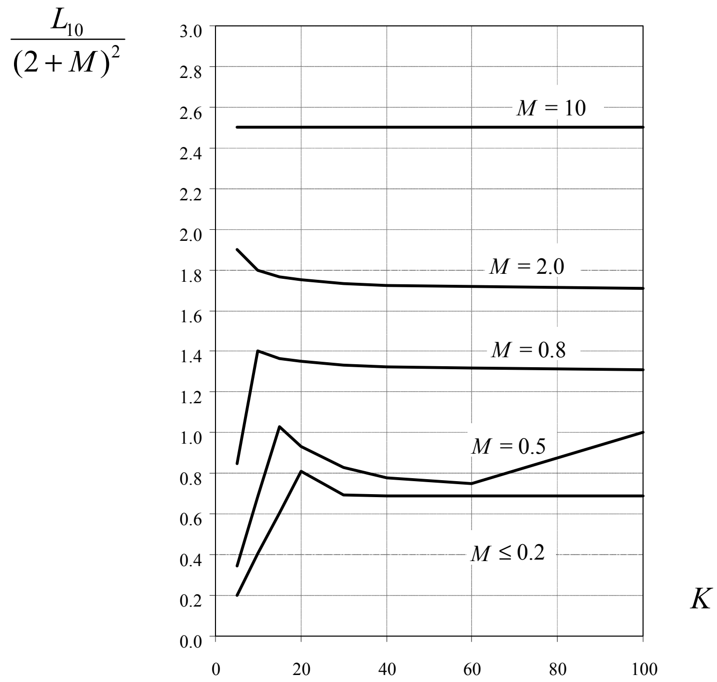


**Figure 3-12**
**Minimum weight, $L_{10}/(2+M)^2$, for pipe on sand**

Minimum pipe weight required to limit maximum relative displacement $Y$ to less than 0.5 on a clayey seabed can be calculated by the following formula:

$$L_{stable} = 90\sqrt{\frac{G_c}{N^{0.67}\cdot K}}\cdot f(M) \qquad \textbf{(3.36)}$$

$$f(M) = \left(0.58\cdot(\log M)^2 + 0.60\cdot(\log M) + 0.47\right)^{1.1} \leq 1.0$$

This formula may give large weights for high values of $G$ and if the criterion for absolute stability gives a lighter pipe, that criterion can be applied.

Minimum pipe weight required to limit maximum relative displacement $Y$ to $10 \cdot \tau/1\,000$ on clay, can be calculated by the following formula:

$$\frac{L_{10}}{(2+M)^2} = \begin{cases} C_1 + \dfrac{C_2}{K^{C_3}} & for \quad K \geq K_b \\ C_1 + \dfrac{C_2}{K_b^{C_3}} & for \quad K < K_b \end{cases} \qquad \textbf{(3.37)}$$

with the coefficients tabulated in Appendix A.

## 3.6 Absolute lateral static stability method

### 3.6.1 Introduction

This section gives an absolute static requirement for lateral on-bottom pipelines based on static equilibrium of forces that ensures that the resistance of the pipe against motion is sufficient to withstand maximum hydrodynamic loads during a sea state, i.e. the pipe will experience no lateral displacement under the design extreme single wave induced oscillatory cycle in the sea state considered.

This requirement for absolute stability may be relevant for e.g. pipe spools, pipes on narrow supports, cases dominated by current and/or on stiff clay.

This requirement of zero displacement leads to a heavy pipe, especially so for cases dominated by wave induced flow velocity with small amplitude, i.e. $K$ and $M$ are small, where force reduction effects due to relative movement are significant even for small movements and the oscillating flow will not move a slightly lighter pipe a long distance. Note also that the peak loads presented below are measured in experiments and the horizontal component thus includes both the drag term and the inertia. Furthermore, with a zero displacement requirement, one cannot take advantage of the increased passive resistance that is built up due to the penetration caused by the pipe being rugged back and forth by the wave induced flow.

### 3.6.2 Design criterion

A pipeline can be considered to satisfy the absolute static stability requirement if:

$$\gamma_{SC} \cdot \frac{F_Y^* + \mu \cdot F_Z^*}{\mu \cdot w_s + F_R} \leq 1.0 \qquad \textbf{(3.38)}$$

and

$$\gamma_{SC} \cdot \frac{F_Z^*}{w_s} \leq 1.0 \qquad \textbf{(3.39)}$$

### 3.6.3 Safety factors

The safety factors $\gamma_{SC}$ to be used for absolute stability in regular winter sea states are listed in Tables 3.5 and 3.6.

**Table 3-5 Safety factors, winter storms in North Sea**

| | *Low* | *Normal* | *High* |
|---|---|---|---|
| Sand and rock | 0.98 | 1.32 | 1.67 |
| Clay | 1.00 | 1.40 | 1.83 |

**Table 3-6 Safety factors, winter storms in Gulf of Mexico and Southern Ocean**

| | *Low* | *Normal* | *High* |
|---|---|---|---|
| Sand and rock | 0.95 | 1.41 | 1.99 |
| Clay | 0.97 | 1.50 | 2.16 |

If cyclonic cases are governing for on-bottom stability design, the safety factors $\gamma_{SC}$ to be used for absolute stability in cyclonic conditions are listed in Tables 3.7 and 3.8.

**Table 3-7 Safety factors, cyclonic conditions North West Shelf**

| | *Low* | *Normal* | *High* |
|---|---|---|---|
| Sand and rock | 0.95 | 1.50 | 2.16 |
| Clay | 0.95 | 1.56 | 2.31 |

**Table 3-8 Safety factors, cyclonic conditions Gulf of Mexico**

| | *Low* | *Normal* | *High* |
|---|---|---|---|
| Sand and rock | 0.95 | 1.64 | 2.46 |
| Clay | 0.93 | 1.64 | 2.54 |

For other areas than those mentioned above, conservative assumptions should be made for the choice of safety factors.

### 3.6.4 Loads

Peak horizontal and vertical loads are:

$$F_Y^* = r_{tot,y} \cdot \frac{1}{2} \cdot \rho_w \cdot D \cdot C_Y^* \cdot \left(U^* + V^*\right)^2 \qquad \textbf{(3.40)}$$

$$F_Z^* = r_{tot,z} \cdot \frac{1}{2} \cdot \rho_w \cdot D \cdot C_Z^* \cdot \left(U^* + V^*\right)^2 \qquad \textbf{(3.41)}$$

Maximum wave induced water particle velocity, including reduction due to directionality and spreading, $U^*$ and $T^*$ can be taken from Eqs. (3.15) and (3.16).

Current velocity, including reduction due to directionality and the boundary layer, $V^*$, can be taken from Section 3.4.2.

Peak load coefficients $C_Y^*$ and $C_Z^*$ are taken from Tables 3.9 and 3.10. Load reductions due to a permeable seabed, soil penetration and trenching can be calculated according to Section 3.4.5.

**Table 3-9 Peak horizontal load coefficients**

| $C_Y^*$ | | $K^*$ | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *2.5* | *5* | *10* | *20* | *30* | *40* | *50* | *60* | *70* | *100* | *≥140* |
| $M^*$ | 0.0 | 13.0 | 6.80 | 4.55 | 3.33 | 2.72 | 2.40 | 2.15 | 1.95 | 1.80 | 1.52 | 1.30 |
| | 0.1 | 10.7 | 5.76 | 3.72 | 2.72 | 2.20 | 1.90 | 1.71 | 1.58 | 1.49 | 1.33 | 1.22 |
| | 0.2 | 9.02 | 5.00 | 3.15 | 2.30 | 1.85 | 1.58 | 1.42 | 1.33 | 1.27 | 1.18 | 1.14 |
| | 0.3 | 7.64 | 4.32 | 2.79 | 2.01 | 1.63 | 1.44 | 1.33 | 1.26 | 1.21 | 1.14 | 1.09 |
| | 0.4 | 6.63 | 3.80 | 2.51 | 1.78 | 1.46 | 1.32 | 1.25 | 1.19 | 1.16 | 1.10 | 1.05 |
| | 0.6 | 5.07 | 3.30 | 2.27 | 1.71 | 1.43 | 1.34 | 1.29 | 1.24 | 1.18 | 1.08 | 1.00 |
| | 0.8 | 4.01 | 2.70 | 2.01 | 1.57 | 1.44 | 1.37 | 1.31 | 1.24 | 1.17 | 1.05 | 1.00 |
| | 1.0 | 3.25 | 2.30 | 1.75 | 1.49 | 1.40 | 1.34 | 1.27 | 1.20 | 1.13 | 1.01 | 1.00 |
| | 2.0 | 1.52 | 1.50 | 1.45 | 1.39 | 1.34 | 1.20 | 1.08 | 1.03 | 1.00 | 1.00 | 1.00 |
| | 5.0 | 1.11 | 1.10 | 1.07 | 1.06 | 1.04 | 1.01 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| | 10 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |

**Table 3-10 Peak vertical load coefficients**

| $C_Z^*$ | | $K^*$ | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *≤2.5* | *5* | *10* | *20* | *30* | *40* | *50* | *60* | *70* | *100* | *≥140* |
| $M^*$ | 0.0 | 5.00 | 5.00 | 4.85 | 3.21 | 2.55 | 2.26 | 2.01 | 1.81 | 1.63 | 1.26 | 1.05 |
| | 0.1 | 3.87 | 4.08 | 4.23 | 2.87 | 2.15 | 1.77 | 1.55 | 1.41 | 1.31 | 1.11 | 0.97 |
| | 0.2 | 3.16 | 3.45 | 3.74 | 2.60 | 1.86 | 1.45 | 1.26 | 1.16 | 1.09 | 1.00 | 0.90 |
| | 0.3 | 3.01 | 3.25 | 3.53 | 2.14 | 1.52 | 1.26 | 1.10 | 1.01 | 0.99 | 0.95 | 0.90 |
| | 0.4 | 2.87 | 3.08 | 3.35 | 1.82 | 1.29 | 1.11 | 0.98 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 0.6 | 2.21 | 2.36 | 2.59 | 1.59 | 1.20 | 1.03 | 0.92 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 0.8 | 1.53 | 1.61 | 1.80 | 1.18 | 1.05 | 0.97 | 0.92 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 1.0 | 1.05 | 1.13 | 1.28 | 1.12 | 0.99 | 0.91 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 2.0 | 0.96 | 1.03 | 1.05 | 1.00 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 5.0 | 0.91 | 0.92 | 0.93 | 0.91 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| | 10 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |

For $K^* < 2.5$ the peak horizontal load coefficient can be taken as $C^*_{Y,K=2.5} \cdot 2.5 / K^*$ where $C^*_{Y,K=2.5}$ is the relevant value in Table 3-9 under $K^* = 2.5$.

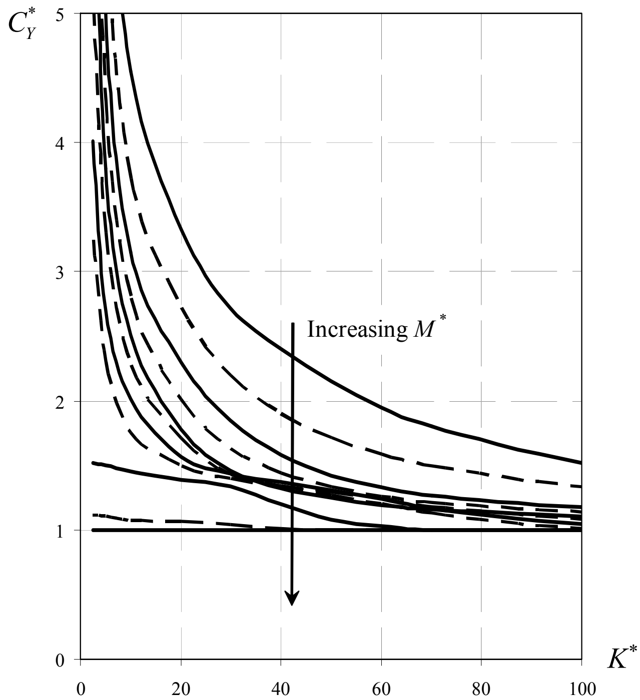


**Figure 3-13**
**Peak horizontal load coefficients**

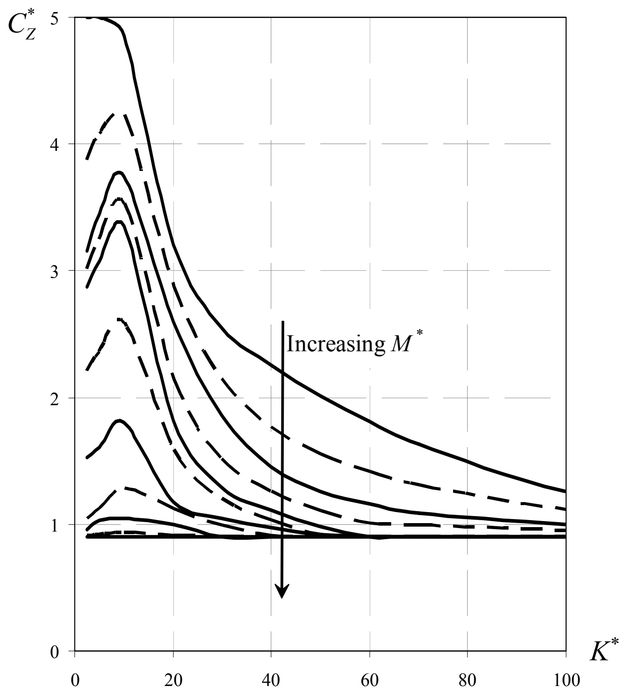


**Figure 3-14**
**Peak vertical load coefficients**

### 3.6.5 Resistance

Pipe soil interaction can be calculated with the coefficients of friction and passive resistance due to initial penetration. Maximum pipe weight prior to the design sea state (e.g. water filled during the system pressure test) and zero lift force can be assumed in the calculation of initial penetration. Reference is made to Section 3.4.6.

If only friction is accounted for on the resistance side, the required weight is given by:

$$L^* = \frac{C_Y^*}{\mu} + C_Z^* \tag{3.42}$$

This is plotted in Figure 3-15 and Figure 3-16 for $\mu = 0.6$ and 0.2, respectively. The factor $k_U$ is inserted with the value 2 for plotting purposes only.

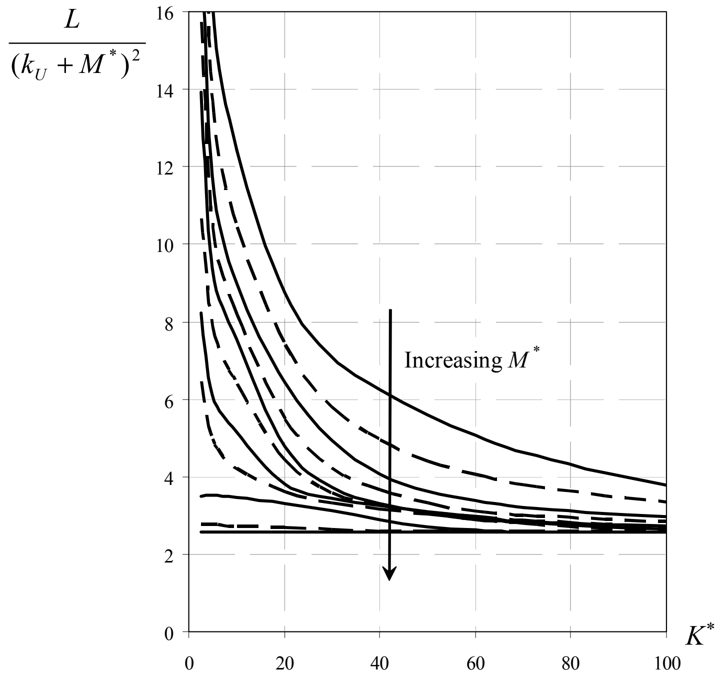


**Figure 3-15**
**Required weight for $\mu = 0.6$ and $F_R = 0$**

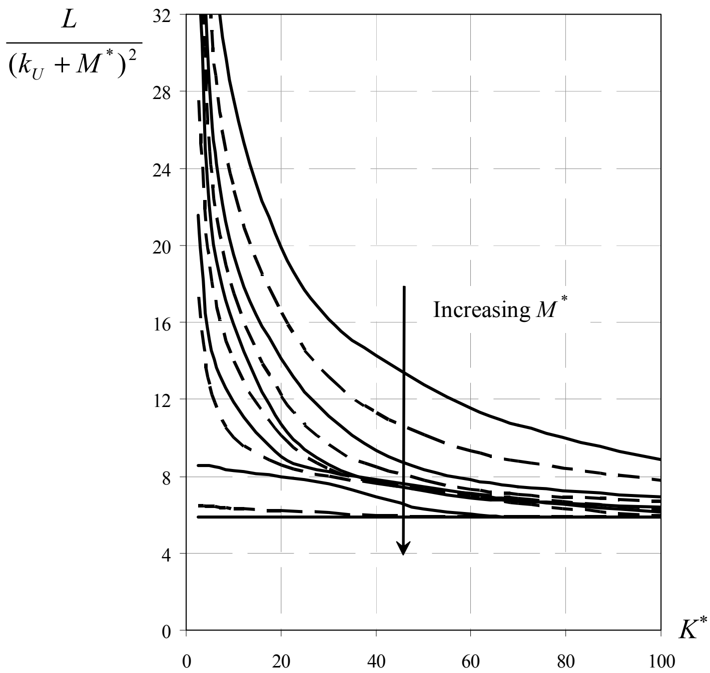


**Figure 3-16**
**Required weight for $\mu = 0.2$ and $F_R = 0$**

# 4. Miscellaneous

## 4.1 Free spans

Free spans affect lateral stability in at least three ways:

- Hydrodynamic loads, in particular lift, are reduced in spans due to water flow between seabed and pipe.
- Current flow, and thus the hydrodynamic loads, are increased slightly because the pipe in the spans lies higher in the current flow boundary layer.
- Soil resistance is increased because of more concentrated vertical pipe to soil (weight) forces on the span shoulders.

It is deemed that the first effect will dominate and that free spans consequently will lead to a more laterally stable pipe.

It is appropriate to account for these effects by using average values for hydrodynamic loads due to the gap, increased current boundary flow and increased penetration.

Note that with the on-set of vortex induced cross-flow vibrations, the drag force will increase significantly. See Ref. /2/ for details.

## 4.2 Mitigating measures

Stability may be obtained by e.g.:

- weight coating
- trenching
- burial
- mattresses
- structural anchors
- intermittent rock berms.

It must be documented that these measures are properly dimensioned for all relevant loads, e.g. hydrodynamic loads and force from pipe throughout the design life of the pipeline.

Where intermittent support is applied, one must consider bending strain and fatigue.

Where intermittent support is applied, one should be aware that scour may lead to free spans.

DNV-OS-F101 presents requirements to concrete weight coating, e.g. minimum thickness, minimum density and reinforcement.

## 4.3 Curved laying

During pipe laying in a curve, with lay radius $R_c$, the horizontal tension $H$ should be kept below:

$$H \leq R_c \cdot (\mu \cdot w_s + F_R) \quad \textbf{(4.1)}$$

in order to avoid that the curved pipe slides in the radial direction. Margin of safety should reflect the consequence of potential sliding. Normally, hydrodynamic forces can be neglected in this assessment.

## 4.4 Seabed stability

The Shields parameter $\theta$ expressing the ratio between the shear stress $\tau_s$ exerted on a grain and the stabilising gravitational force acting the grain is given as:

$$\theta = \frac{\tau_s}{\rho_w \cdot g \cdot (s_s - 1) \cdot d_{50}} \quad \textbf{(4.2)}$$

When the Shields parameter exceeds a critical value, i.e. 0.04, the sediments in a non-cohesive soil will start moving, i.e. the seabed becomes unstable.

The shear stress can be expressed through the water particle velocity through:

$$\tau_s = \frac{1}{2} \rho_w \cdot f_w \cdot U^2 \quad \textbf{(4.3)}$$

Where $U$ may denote water particle velocity.

The critical water particle velocity with respect to seabed instability is thus:

$$U_{cr} = \sqrt{0.08 \cdot \frac{g \cdot (s_s - 1) \cdot d_{50}}{f_w}} \qquad \textbf{(4.4)}$$

The friction factor $f_w$ may be taken as:

Waves only $f_w = 0.04 \cdot \left( \frac{A_w}{K_b} \right)^{-0.25}$ **(4.5)**

Current only $f_w = 0.005$

The two formulae yield the same results when the orbital semi-diameter of the water particles is about $10^4$ times the main grain size $d_{50}$. There exists no simple way of combining wave and current.

By these formulae, it may be shown that non-cohesive soil will in many cases become unstable for water velocities significantly less than the velocity that causes an unstable pipe. However, there exists no straight forward method to include this in pipeline stability design.

### 4.5 Soil liquefaction

Soil liquefaction denotes the phenomenon that the soil loses a significant part of – or all – its shear strength. Liquefaction may occur due to cyclic shear stresses, imposed by waves or earth quakes, that generate excessive pore pressure until the soil loses a significant part of its shear strength (residual liquefaction) or if a steep wave travels over a loose soil inducing a upward-directed pressure gradient under the wave through (instantaneous liquefaction).

Obviously, soil liquefaction will affect both vertical stability (sinking and floatation) and lateral stability.

Depending on the specific gravity of the pipe, soil liquefaction may make a heavy pipe laid on the seabed to sink into the soil and bury itself, or make a light (and buried) pipe to float up through the soil.

Models do exist to predict possible liquefaction, however some rather sophisticated (in situ and laboratory) soil tests are required in order to quantify the parameters that enter these models. Reference is made to Refs. /17/ and /18/.

## 5. References


| | |
|---|---|
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| /3/ | DNV Recommended Practice DNV-RP-C205 *Environmental Conditions and Environmental Loads*, 2007. |
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| /8/ | V. Jacobsen, M. B. Bryndum and D. T. Tshalis: *Hydrodynamic Forces on Pipelines: Model Tests*, OMAE, Houston, Texas, 1988. |
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| | |
|---|---|
| /15/ | R. L. P. Verley: *Effect of Wave Short-Crestedness for Pipelines*, International Journal of Offshore and Polar Engineering – Vol. 3, No. 2, June 1993. |
| /16/ | D. Myrhaug and L. E. *Holmedal: Bottom Friction Caused by Boundary Layer Streaming Beneath Random Waves for Laminar and Smooth Turbulent Flow*, Ocean Engineering 32 (2005) 195 – 222. |
| /17/ | ASCE Journal of Waterway, Port, Coastal and Ocean Engineering, Volume 132, Number 4 (2006). Special Issue:<br>*Liquefaction Around Marine Structures. Processes and Benchmark Cases*. Editor: D. M. Sumer. |
| /18/ | J. S. Damsgaard and A. C. Palmer:<br>*Pipeline Stability on a Mobile and Liquefied Seabed:*<br>*A Discussion of Magnitudes and Engineering Implications*. OMAE2001/PIPE-4030, June 3-8, 2001. |

# APPENDIX A
# STABILITY CURVES FOR CLAY

Minimum pipe weight required to limit maximum relative displacement $Y$ to $10 \cdot \tau/1000$ on clay, can be calculated by Eq. (3.37) with the coefficients listed below. The value of K should not be taken less than 5. Linear interpolation can be applied in the region 0.003 < N < 0.006.

**Table A-1 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c = 0.0556$**

| $G_c = 0.0556$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M$ | $N \leq 0.003$ | | | | $0.006 \leq N \leq 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 0 | 9 | 0.6 | 10 | 0.2 | 5 | 0.5 | 15 |
| 0.4 | 0 | 8 | 0.6 | 10 | 0.2 | 5 | 0.5 | 15 |
| 0.5 | 0.1 | 7 | 0.6 | 10 | 0.4 | 4 | 0.5 | 15 |
| 0.6 | 0.1 | 7 | 0.6 | 10 | 0.4 | 4 | 0.5 | 15 |
| 0.8 | 0.1 | 7 | 0.6 | 10 | 0.7 | 3 | 0.5 | 15 |
| 1.0 | 0.4 | 5 | 0.6 | 5 | 0.7 | 3 | 0.5 | 15 |
| 1.5 | 0.4 | 5 | 0.6 | 5 | 1.1 | 2 | 0.5 | 15 |
| 2.0 | 0.7 | 3 | 0.6 | 5 | 1.6 | 0 | 0.5 | 15 |
| ≥ 4.0 | 1.4 | 1 | 0.6 | 5 | 1.9 | 0 | 0.5 | 15 |

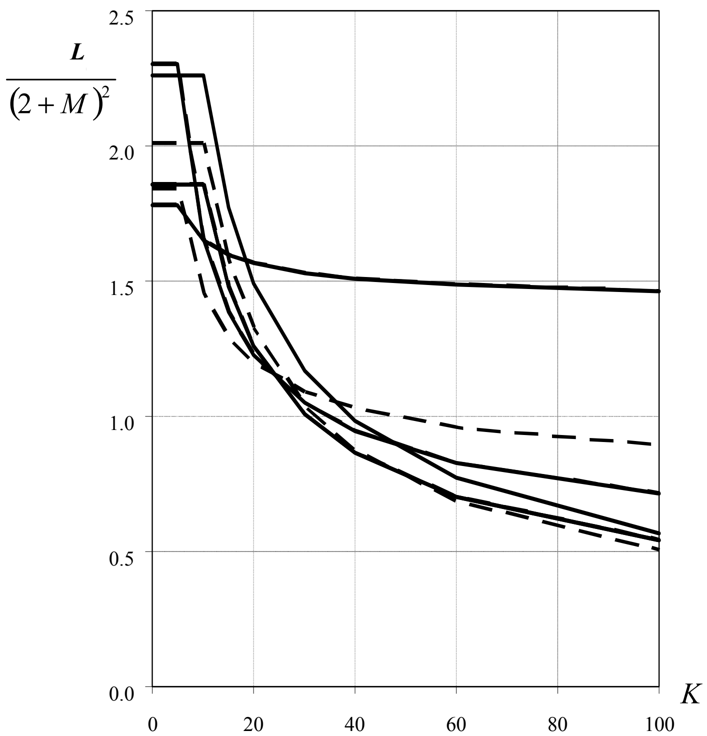


**Figure A-1**
**Minimum weight for a pipe on clay, $Y = 10 \cdot \tau/1\,000$, $N \leq 0.003$, $G_c = 0.0556$**

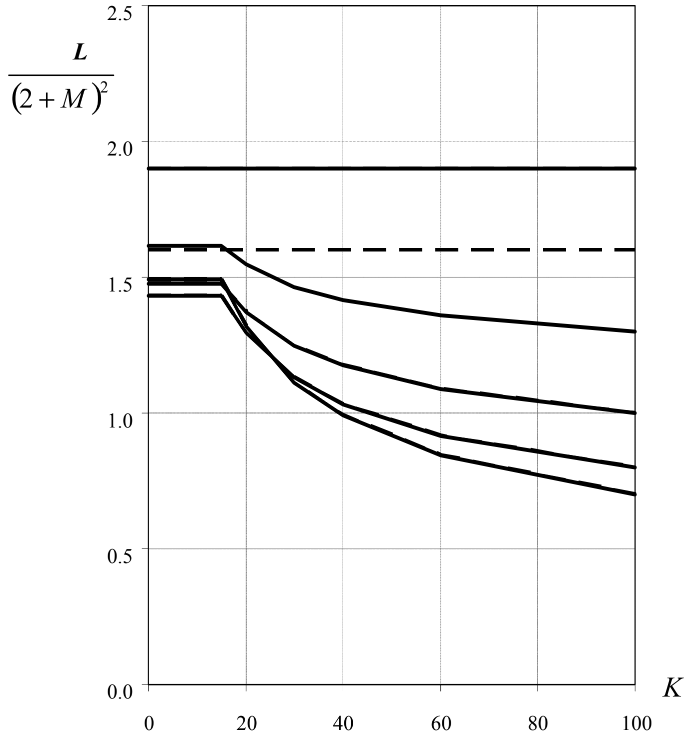


**Figure A-2**
**Minimum weight for a pipe on clay, $Y = 10 \cdot \tau / 1\,000$, $0.006 \leq N \leq 0.024$, $G_c = 0.0556$**

| Table A-2 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c = 0.111$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $G_c = 0.111$ | | | | | | | | |
| $M$ | $N \leq 0.003$ | | | | $0.006 \leq N \leq 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 0.1 | 9 | 0.6 | 10 | 0.1 | 7 | 0.6 | 10 |
| 0.4 | 0.1 | 8 | 0.6 | 10 | 0.1 | 7 | 0.6 | 10 |
| 0.5 | 0.1 | 8 | 0.6 | 10 | 0.1 | 7 | 0.6 | 10 |
| 0.6 | 0.2 | 8 | 0.6 | 10 | 0.2 | 6 | 0.6 | 10 |
| 0.8 | 0.4 | 7 | 0.6 | 5 | 0.3 | 6 | 0.6 | 10 |
| 1.0 | 0.4 | 7 | 0.6 | 5 | 0.4 | 6 | 0.6 | 10 |
| 1.5 | 0.4 | 5 | 0.6 | 5 | 0.8 | 4 | 0.6 | 10 |
| 2.0 | 0.7 | 3 | 0.6 | 5 | 1.5 | 0 | 0.6 | 10 |
| ≥ 4.0 | 1.4 | 1 | 0.6 | 5 | 1.5 | 0 | 0.6 | 10 |

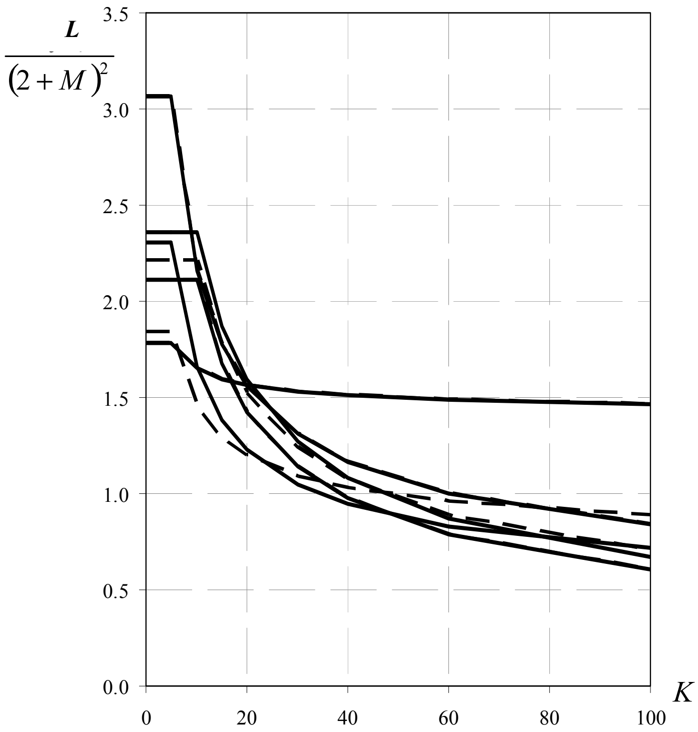


**Figure A-3**
**Minimum weight for a pipe on clay, Y = 10· τ/1 000, $N \leq 0.003$, $G_c$ = 0.111**

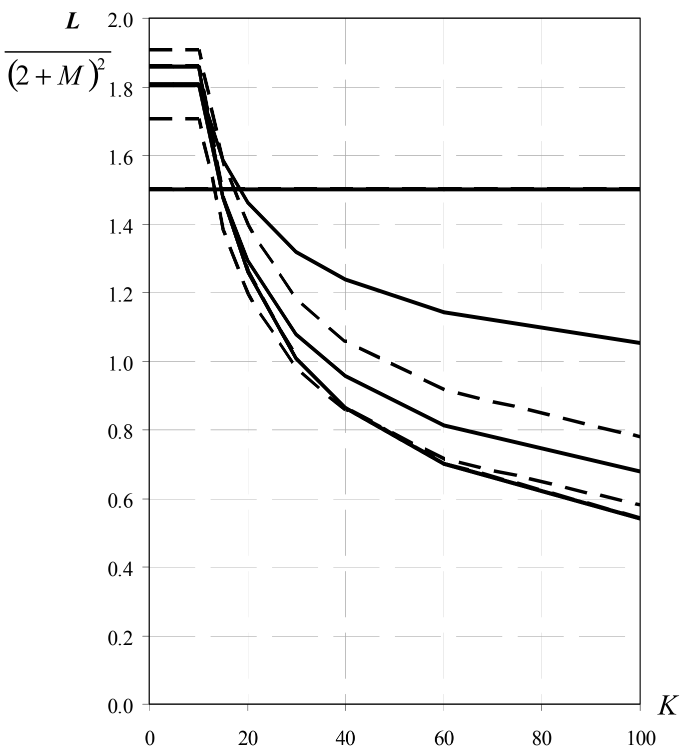


**Figure A-4**
**Minimum weight for a pipe on clay, Y = 10· τ/1000, $0.006 \leq N \leq 0.024$, $G_c$ = 0.111**

| Table A-3 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c = 0.222$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $G_c = 0.222$ | | | | | | | | |
| $M$ | $N \le 0.003$ | | | | $0.006 \le N \le 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 0.1 | 8 | 0.5 | 15 | 0.1 | 8 | 0.5 | 10 |
| 0.4 | 0.1 | 7 | 0.5 | 10 | -0.3 | 8 | 0.5 | 10 |
| 0.5 | 0.1 | 7 | 0.5 | 10 | -0.1 | 7 | 0.5 | 10 |
| 0.6 | 0.1 | 7 | 0.5 | 10 | 0.0 | 7 | 0.5 | 10 |
| 0.8 | 0.1 | 7 | 0.5 | 5 | 0.1 | 6 | 0.5 | 5 |
| 1.0 | 0.1 | 7 | 0.5 | 5 | 0.1 | 6 | 0.5 | 5 |
| 1.5 | 0.1 | 7 | 0.5 | 5 | 0.5 | 3 | 0.5 | 5 |
| 2.0 | 0.1 | 7 | 0.5 | 5 | 0.9 | 2 | 0.5 | 5 |
| 4.0 | 0.1 | 7 | 0.5 | 5 | 1.7 | 0 | 0.5 | 5 |
| ≥ 10 | 0.1 | 7 | 0.5 | 5 | 1.7 | 0 | 0.5 | 5 |

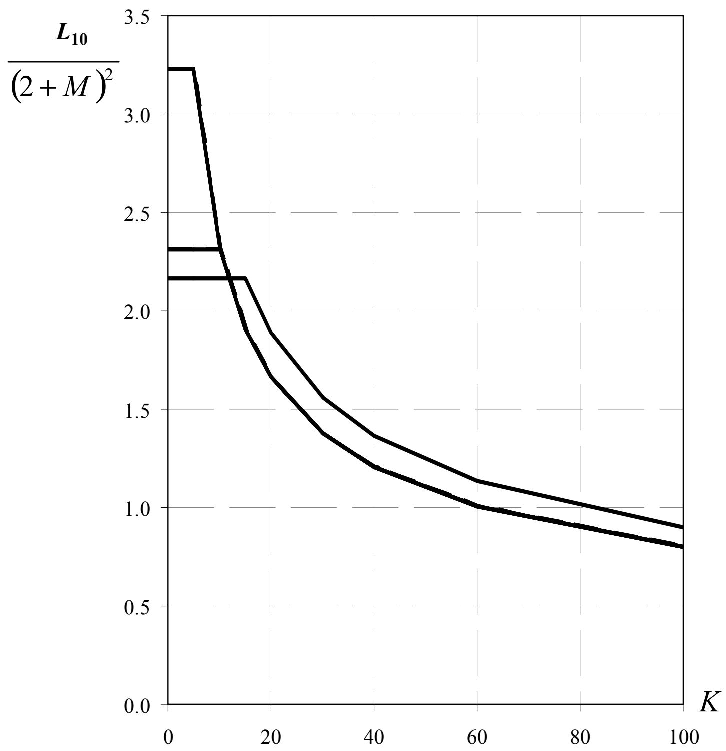


**Figure A-5**
**Minimum weight for a pipe on clay, Y = 10· τ/1000, N ≤ 0.003, $G_c$ = 0.222**

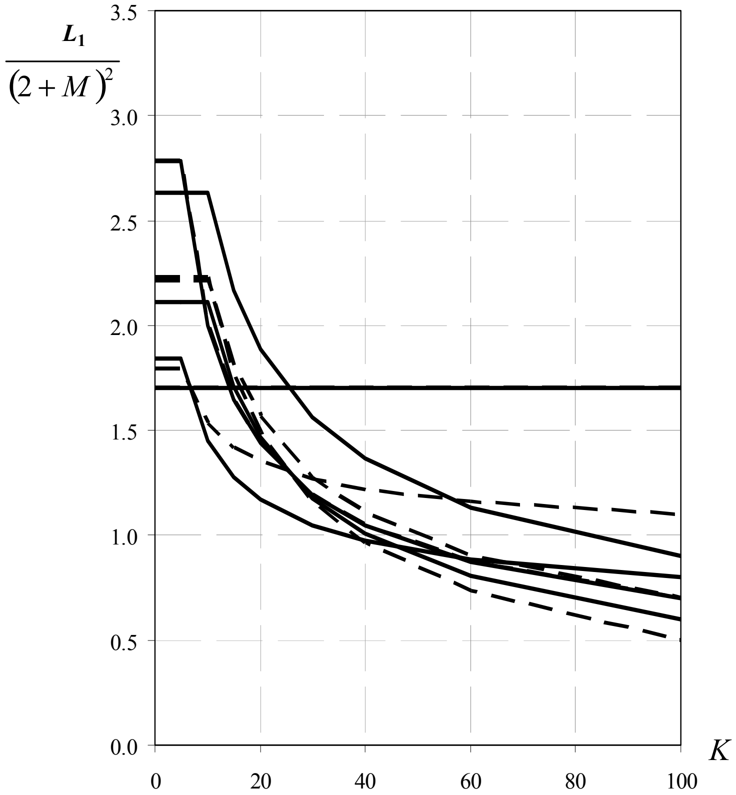


**Figure A-6**
**Minimum weight for a pipe on clay, Y = 10· τ/1 000, 0.006 ≤ $N$ ≤ 0.024, $G_c$ = 0.222**

**Table A-4 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c$ = 0.556**

| $G_c$ = 0.556 | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M$ | $N \leq 0.003$ | | | | $0.006 \leq N \leq 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 1.4 | 3 | 0.5 | 15 | 0.0 | 8 | 0.5 | 10 |
| 0.4 | 0.5 | 6 | 0.5 | 5 | 0.3 | 6 | 0.5 | 5 |
| 0.5 | 0.5 | 6 | 0.5 | 5 | 0.3 | 6 | 0.5 | 5 |
| 0.6 | 0.5 | 6 | 0.5 | 5 | 0.3 | 6 | 0.5 | 5 |
| 0.8 | 1.1 | 4 | 0.5 | 5 | 0.4 | 7 | 0.5 | 5 |
| 1.0 | 1.3 | 4 | 0.5 | 10 | 0.4 | 7 | 0.5 | 5 |
| 1.5 | 1.2 | 7 | 0.5 | 10 | 0.8 | 6 | 0.5 | 10 |
| 2.0 | 1.2 | 7 | 0.5 | 10 | 0.8 | 6 | 0.5 | 10 |
| 4.0 | 1.2 | 7 | 0.5 | 10 | 0.8 | 6 | 0.5 | 10 |
| ≥ 10 | 1.4 | 6 | 0.5 | 10 | 0.8 | 6 | 0.5 | 10 |

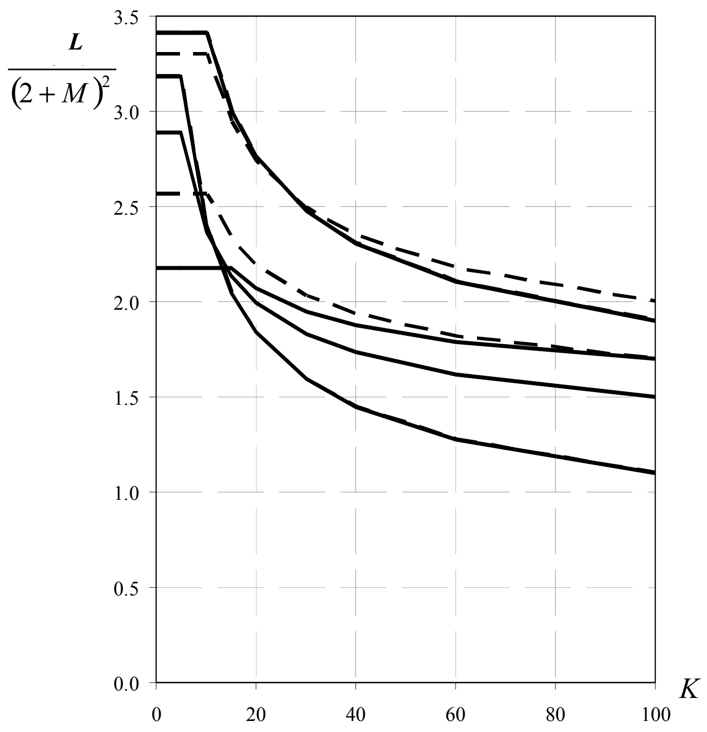


**Figure A-7**
**Minimum weight for a pipe on clay, Y = 10· $\tau$/1 000, $N \leq 0.003$, $G_c = 0.556$**

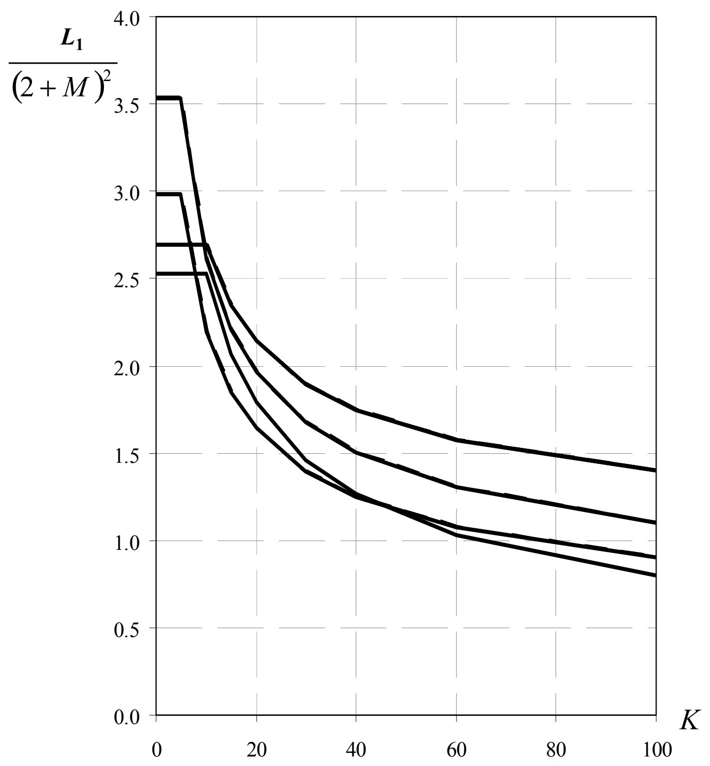


**Figure A-8**
**Minimum weight for a pipe on clay, Y = 10· $\tau$/1 000, $0.006 \leq N \leq 0.024$, $G_c = 0.556$**

**Table A-5 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c = 1.11$**

| $G_c = 1.11$ | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M$ | $N \leq 0.003$ | | | | $0.006 \leq N \leq 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 2.1 | 1 | 0.5 | 15 | 1.4 | 4 | 0.5 | 15 |
| 0.4 | 2.4 | 2 | 0.5 | 15 | 1.1 | 7 | 0.5 | 15 |
| 0.5 | 2.4 | 2 | 0.5 | 15 | 1.5 | 5 | 0.5 | 15 |
| 0.6 | 1.9 | 6 | 0.5 | 15 | 1.6 | 5 | 0.5 | 15 |
| 0.8 | 2.2 | 8 | 0.5 | 15 | 1.9 | 6 | 0.5 | 15 |
| 1.0 | 2.2 | 8 | 0.5 | 15 | 2.2 | 6 | 0.5 | 15 |
| ≥ 1.5 | 2.4 | 8 | 0.5 | 15 | 2.0 | 8 | 0.5 | 15 |

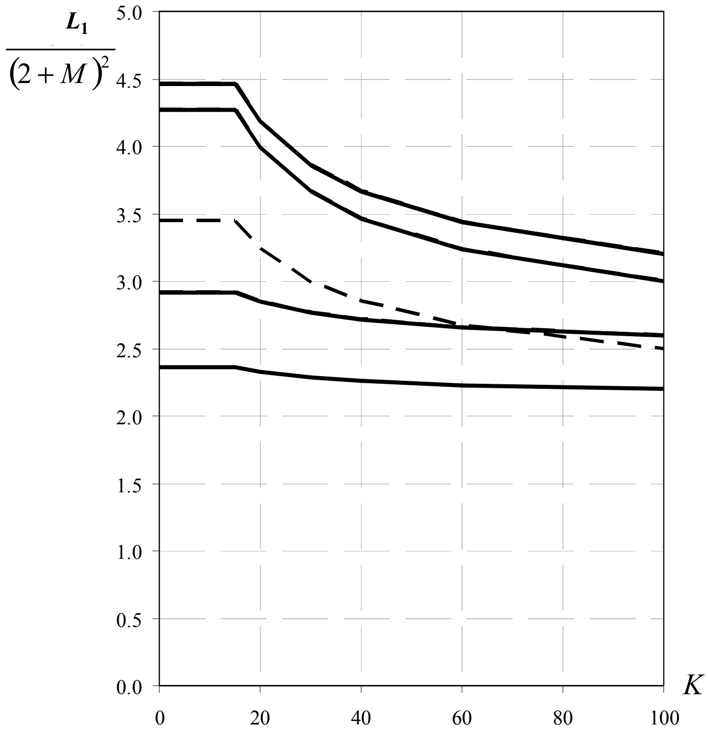


**Figure A-9**
**Minimum weight for a pipe on clay, Y = 10· τ/1 000, $N \leq 0.003$, $G_c = 1.11$**

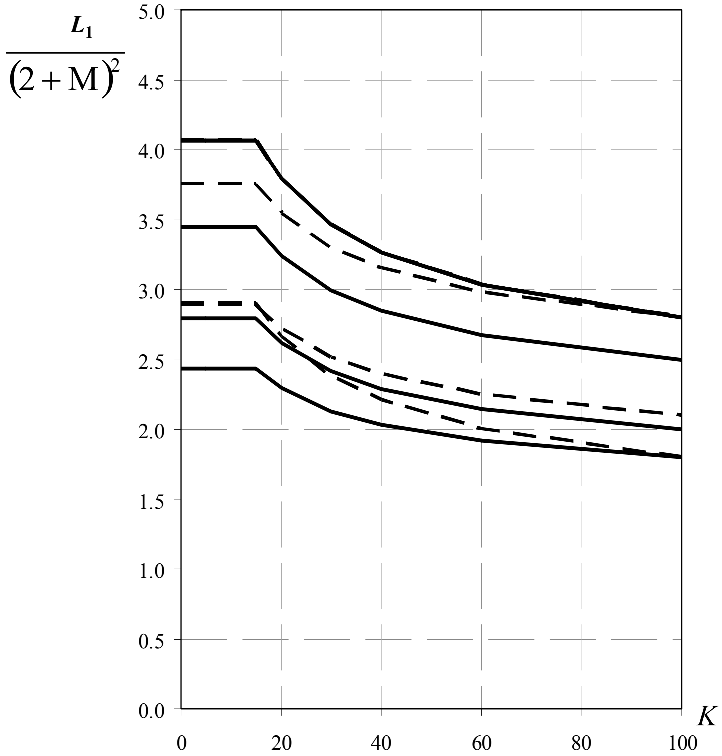


**Figure A-10**
**Minimum weight for a pipe on clay, Y = 10· τ/1000, 0.006 ≤ $N$ ≤ 0.024, $G_c$ = 1.11**

**Table A-6 Parameters for calculating minimum weight, $L_{10}/(2 + M)^2$, for pipe on clay, $G_c$ = 2.78**

| | $G_c$ = 2.78 | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $M$ | $N \le 0.003$ | | | | $0.006 \le N \le 0.024$ | | | |
| | $C_1$ | $C_2$ | $C_3$ | $K_b$ | $C_1$ | $C_2$ | $C_3$ | $K_b$ |
| ≤ 0.2 | 3.4 | 1 | 0.5 | 20 | 2.7 | 3 | 0.5 | 20 |
| 0.4 | 3.4 | 1 | 0.5 | 20 | 2.4 | 4 | 0.5 | 20 |
| 0.5 | 3.0 | 4 | 0.5 | 20 | 2.2 | 7 | 0.5 | 20 |
| 0.6 | 3.2 | 6 | 0.5 | 15 | 1.9 | 9 | 0.5 | 15 |
| 0.8 | 2.4 | 12 | 0.5 | 15 | 1.9 | 12 | 0.5 | 15 |
| 1.0 | 2.3 | 12 | 0.5 | 15 | 1.5 | 14 | 0.5 | 15 |
| 1.5 | 2.3 | 12 | 0.5 | 15 | 1.5 | 14 | 0.5 | 15 |
| 2.0 | 2.3 | 12 | 0.5 | 15 | 1.5 | 14 | 0.5 | 15 |
| 4.0 | 2.3 | 12 | 0.5 | 15 | 1.5 | 14 | 0.5 | 15 |
| ≥ 4.0 | 2.3 | 12 | 0.5 | 15 | 1.5 | 14 | 0.5 | 15 |

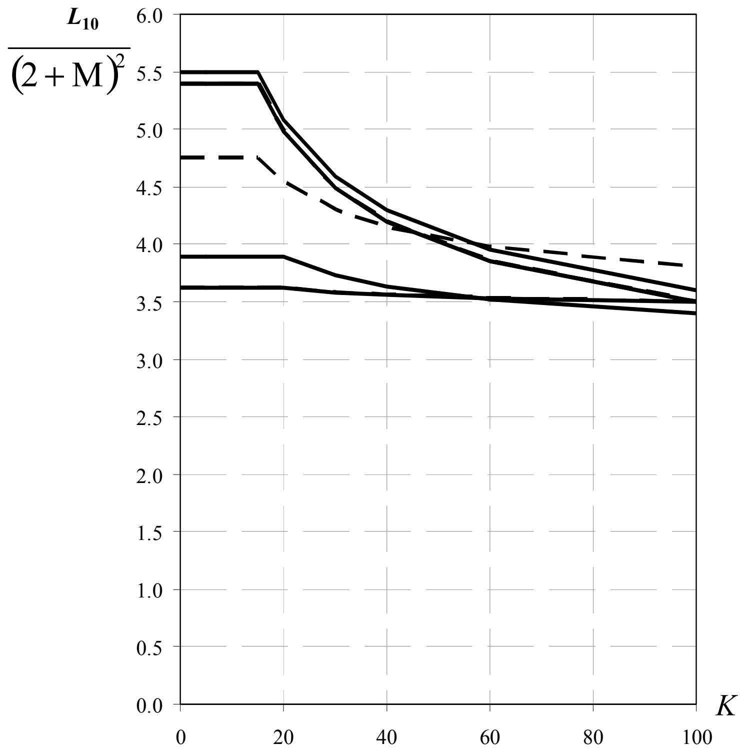


**Figure A-11**
**Minimum weight for a pipe on clay, Y = 10· $\tau$/1000, $N \leq 0.003$, $G_c$ = 2.78**

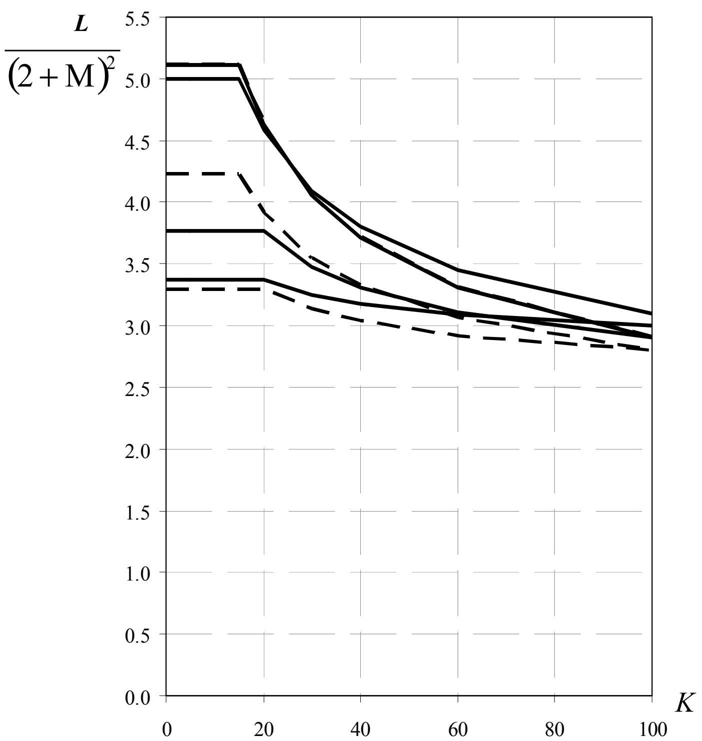


**Figure A-12**
**Minimum weight for a pipe on clay, Y = 10· $\tau$/1000, $0.006 \leq N \leq 0.024$, $G_c$ = 2.78**

# APPENDIX B
# CARBONATE SOILS

## B.1 Definition

A soil is classified as carbonate soil if it contains more than 50% of calcium carbonate [1]. Most carbonate soils are composed of large accumulations of the skeletal remains of marine organisms, such as coralline algae, coccoliths, foraminifera and echinoderms, although they also exist as non-skeletal material in the form of oolites, pellets and grape-stone. Carbonate deposits are abundant in the warm, shallow tropical waters, such as offshore Africa, Australia, Brazil and the Middle East.

## B.2 Characteristic features

Carbonate soils have important characteristic features that distinguish them from terrigenous silica seabed materials. Particles of skeletal carbonate material can be highly angular with rough surfaces and intra-particle voids, leading to a soil fabric that is very open and compressible. Non-skeletal carbonate particles may be rounded and solid, but are still susceptible to crushing or fracturing due to the low hardness that calcium carbonate has compared with quartz.

A typical granular carbonate soil will be characterized by a high void ratio (and low density), high compressibility due to the high initial void ratio and the crushable nature of the individual particles, and a high friction angle due to angularity, roughness and interlocking of the particles.

Carbonate sediments are susceptible to transformation by biological and chemical processes over time. Older sedimentary deposits that may have been sub-aerially exposed at some stage in geological life are prone to cementation, which completely alters the mechanical properties of the sediment. The cementation may occur over a flat horizon, forming a caprock layer near the seabed, or may occur in irregular discontinuous lenses.

The engineering properties of uncemented carbonate soils differ from silica rich soils in the following key ways:

- Calcium carbonate has a low hardness value compared to quartz. This leads to relatively high crushability of carbonate soils at relatively low stress levels.
- Carbonate soils often have large porosity, resulting in high void ratio and low density, and are therefore more compressible than soils from silica deposits. However they generally exhibit higher friction angles than equivalent silica soils.
- The undrained cyclic strength of carbonate soils is generally lower than for most silica soils and permeabilities also tend to be lower. Consequently, carbonate soils are generally more susceptible to liquefaction under the action of cyclic loading.
- More detailed description of these features and experimental data can be found in [1] to [7].

## B.3 Pipeline response on carbonate soils

A pipeline may be viewed as a special type of strip footing that has a circular cross-section. Therefore its behaviour in carbonate sediments is in many ways similar to that of shallow foundations in the same soil. Experimental evidence indicates that pipeline response in carbonate soils is characterised by the following features:

- Approximately linear vertical load-displacement response.
- Relatively large lateral displacement to achieve the ultimate resistance. A pipeline on low density calcareous sand may typically move laterally two or more diameters before developing ultimate soil resistance, compared with typically about half to one diameter for a pipeline on silica sands.
- Cyclic loading induces larger embedment than for pipelines in silica sand.
- Load-displacement response typically exhibits a ductile strain hardening response (except where the pipe is embedded below its equilibrium depth), unlike in silica sands that generally exhibit post-peak strain softening behaviour.
- Significant excess pore pressure may accumulate under cyclic environmental loads acting on the pipe, as well as wave pressure loading directly on the seabed, compared to silica soils. This is because such sediments are more prone to degradation and compaction under cyclic loading and tend to have larger coefficients of consolidation than typical silica sands, and consequently, carbonate soils have a higher propensity for liquefaction than silica soils.

Details of the above-mentioned experimental evidence are presented in [8] to [11].

## B.4 Pipe soil interaction model

Theoretical models have been developed for drained pipe-soil interaction in carbonate sands within the framework of soil plasticity. These models link the displacement increments of a pipeline with the load increments through a non-linear stiffness or flexibility matrix. This approach enables straightforward incorporation of the pipe-soil interaction model into the full structural analysis of a pipeline. Simplified expressions for calculating the ultimate lateral soil resistance of pipelines have also been derived for basic pipeline stability assessments. Detailed descriptions of these models are given in [10] and [12] to [14].

In most real situations, pipeline-seabed interaction in carbonate soils is much more complex than assumed in a model for drained pipe-soil interaction. Local pipe-soil interaction and large scale wave-soil interaction must both be considered, and this is further complicated by partially drained/ undrained behaviour of the seabed soil in many cases. Therefore a comprehensive model is required to describe pipe-soil interaction for offshore pipelines under realistic wave and soil conditions. An appropriate model would be able to perform the following functions:

— Predict the accumulation of excess pore pressure in the seabed soil due to wave actions acting directly on the seabed and due to cyclic loading on the pipeline.
— Predict the resulting strength/ stiffness degradation of the seabed soil in the vicinity of the pipeline.
— Calculate the pipeline embedment, i.e., self-burial of the pipeline.
— Evaluate the increase in lateral resistance due to self-burial.

Such a model has not been presented within the public domain. As a general guide, local properties of the carbonate soils should be determined, which allows development of a model for local use.

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