RECOMMENDED PRACTICE

VOLUME E: PIPELINES AND RISERS
GROUP E 300: STRENGTH AND IN-PLACE STABILITY OF PIPELINES

RP E305

ON-BOTTOM STABILITY DESIGN OF SUBMARINE PIPELINES

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VERITEC

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PREFACE

Recommended Practice are guidelines for solutions, calculation methods, technical specifications (Volume A-E) and design of offshore objects (Volume O).

The **Recommended Practice** publications cover proven technology and solutions which have been experienced by Veritec to represent good practice. The publications do not cover all areas of offshore technology, but are meant to supplement the recognized codes and standards frequently used within the industry.

The **Recommended Practice** publications are divided into 6 volumes, and each volume is divided into groups. Within each group the **Recommended Practices** are issued as selfcontained booklets. See table on next page.

Volume O gives guidelines on design of offshore objects. These publications are considered as Recommended Practices related to offshore *objects*.

Volume A-E give guidelines on specific technical solutions, methods of calculations etc. These publications are considered as Recommended Practices related to *subjects*.

RP E305 On-Bottom Stability Design of Submarine Pipelines.

- General

This **Recommended Practice** replaces the following Veritec publications,
None

- Changes in this revision of **Recommended Practice**.
None.

Table 1: General view of content of Veritec's **Recommended Practices.**

RECOMMENDED PRACTICES related to offshore *objects*

Volume O		**Design of offshore objects**
Group	O 100	Design of drilling and production facilities
Group	O 200	Design of structures
Group	O 300	Design of pipeline systems
Group	O 400	Design of subsea production systems

RECOMMENDED PRACTICES related to *subjects*

Volume A		**Quality assurance methodology**
Group	A 100	Quality systems
Group	A 200	Evaluation of contractors and suppliers
Group	A 300	Quality audits
Group	A 400	Qualification of QA/QC personnel
Group	A 500	Safety assurance systems
Group	A 600	Safety and risk analysis
Group	A 700	Documentation and information systems
Volume B		**Materials technology**
Group	B 100	Materials for structural application
Group	B 200	Materials for application in drilling, completion, production and processing systems
Group	B 300	Materials for pipelines and risers
Group	B 400	Corrosion protection
Group	B 500	Sampling and testing of materials
Group	B 600	Welding and heat treatment
Group	B 700	Non-destructive examination
Volume C		**Facilities on offshore installations**
Group	C 100	General safety
Group	C 200	Production and processing systems
Group	C 300	Instrumentation
Group	C 400	Electrical systems
Group	C 500	Drilling and well completion
Group	C 600	Mechanical equipment and piping systems
Group	C 700	Fabrication of drilling, production and processing plants
Group	C 800	Hook-up and commissioning
Group	C 900	In-service inspection and maintenance of drilling, production and processing plants.
Volume D		**Structures**
Group	D 100	Risk and reliability of structures
Group	D 200	Loads and conditions
Group	D 300	Foundation
Group	D 400	Steel structures
Group	D 500	Concrete structures
Group	D 600	Aluminium structures

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CONTENTS

0. LIST OF SYMBOLS

C	-	constant
D	-	nominal outside diameter of pipe
E	-	modulus of elasticity
G	-	relative soil weight, sand
K	-	Keulegan - Carpenter number, $K = U_S T_u/D$
L	-	pipe weight parameter
M	-	current to wave velocity ratio, $M = U_c/U_S$
R	-	reduction factor due to wave directionality and wave spreading
S	-	shear strength parameter
T	-	time parameter, $T = T_l/T_u$
Y	-	total displacement
A_O	-	orbital semi-diameter of particle velocity
A_S	-	significant acceleration
C_D	-	drag coefficient
C_L	-	lift coefficient
C_M	-	inertia coefficient
D_{CC}	-	outer steel pipe diam. incl. corrosion coating
D_i	-	internal pipe diameter
D_S	-	steel pipe outer diameter
F_D	-	drag force
F_I	-	inertia force
F_L	-	lift force
F_W	-	load factor
H_S	-	significant wave height
K_b	-	equivalent sand roughness parameter
S_f	-	safety factor
$S_{\eta\eta}(\omega)$	-	wave spectrum (long-crested sea)
$S_{uu}(\omega)$	-	near-bottom horizontal velocity spectrum
S_u	-	undrained shear strength of clay soil
T_l	-	time length
T_n	-	parameter, $T_n = \sqrt{(d / g)}$
T_p	-	spectral peak period of surface wave spectrum
T_u	-	mean zero up-crossing period
U_c	-	current velocity perpendicular to the pipe
U_s	-	significant velocity perpendicular to the pipe
U_s*	-	significant velocity perpendicular to the pipe (no reduction factor included).
$U*$	-	friction velocity
U_D	-	average velocity over pipe diameter, D
U_r	-	reference steady velocity
$U(z)$	-	steady flow velocity
W_S	-	submerged pipe weight
W_{sd}	-	design weight
d	-	water depth
d_{50}	-	mean grain size
f_{ws}	-	correction factor on submerged weight
g	-	gravity constant
k	-	wave number
m_0, m_2	-	spectral moments
n	-	spreading exponent
t_s	-	steel pipe thickness
z	-	elevation above seabed
z_o	-	bottom roughness parameter
z_{oa}	-	apparent roughness

z_r	-	reference height above seabed
α	-	Phillips' constant
β	-	sub-direction around main wave direction
δ	-	scaled lateral displacement
ε,ε'	-	engineering and generalized strain
$\psi(\beta,\theta)$	-	spreading function
γ	-	peakedness parameter in Jonswap wave spectrum
κ	-	von Karman's constant
μ	-	soil friction factor
ω	-	angular frequency
ω_p	-	angular frequency of spectral peak
ρ_c	-	density of concrete coating
ρ_{cc}	-	density of corrosion coating
ρ_i	-	density of internal content
ρ_s	-	density of sand soil
ρ_{st}	-	density of steel material
ρ_w	-	mass density of water
σ	-	spectral width parameter
θ	-	main wave direction, phase angle
θ_p	-	direction perpendicular to the pipeline

1. INTRODUCTION

This Recommended Practice outlines the basic considerations with regard to the stability design of submarine pipelines.

The main objectives of this recommended practice are to make the latest state-of-the-art information on pipeline stability available for use in the design of submarine pipelines, and to provide a framework from which stability design methods can be developed further as more information becomes available. The RP is mainly based on the results from the Pipeline Stability project PIPESTAB carried out by SINTEF (1983 - 1987) and sponsored by Esso Norge A/S and Statoil, see /2/ - /8/.

Results from other research programs may be equally applicable for On-Bottom Stability of Pipelines. It is the intention, through revisions of the present RP, to incorporate other results/data as they become available and thereby extend the limits for use.

The design method presented in this Recommended Practice relates to a pipeline resting on the sea bed throughout its lifetime, or prior to some other form of stabilization (eg. trenching, burial, self-burial). The stability of the pipeline is then directly related to the submerged weight of the pipeline, the environmental forces and the resistance developed by the sea bed soil. Consequently the aim of stability design is to verify that the submerged weight of the pipeline is sufficient to meet the required stability criteria.

2. DESIGN CONDITIONS

2.1 Basic Conditions

2.1.1 The following basic conditions should be considered during the on-bottom stability design of submarine pipelines:

- Environmental conditions
- Geotechnical conditions of the sea bed
- Topographical conditions of the sea bed (eg. slope, rock outcrops, depressions)
- Bathymetry (water depth)
- Pipe data (diameter, wall thickness, concrete coating)
- Location of pipeline restraints (riser connections, crossings, etc)

2.2 Return Periods

2.2.1 The stability design is to be based on a given return period of near-bottom environmental conditions acting perpendicular to the pipe. In general, both near-bottom wave induced particle velocities and near-bottom currents will need to be considered.

2.2.2 If sufficient information is available on joint probability of waves and current, then the combined wave and steady current with 100 year recurrence interval should be used. If inadequate information is available on the joint probability of waves and current, then the following are suggested for the operational condition:

If waves dominates forces	Waves :	100 year return condition of near-bottom wave-induced particle velocity normal to the pipeline.
	Current :	10 year return condition.
If current dominates forces	Waves :	10 year return condition
	Current :	100 year return condition.

2.2.3 For temporary phases, the recurrence period should be taken as follows:
Duration less than 3 days: The environmental parameters for determination of environmental loads may be established based on reliable weather forcasts.
Duration in excess of 3 days: a) No danger for loss of human lives. A return period of 1 year for the relevant season can be applied.
b) Danger for loss of human lives. The environmental parameters should be defined with a 100 year seasonal return period.
However, the relevant season should not be taken less than 2 months.

2.3 Environmental Conditions

2.3.1 The following environmental conditions should be evaluated at a number of positions along the length of the pipeline :

- Waves
- Currents

The number of positions necessary to adequately define the environment will be dependent on the length of the pipeline and the variations in water depth, seabed soil and meteorological conditions.

2.3.2 The environmental conditions used in the stability design should be based on adequate data from the area in question. The data may be from measurements, hindcast models, or visual observations. If sufficient data on the particular area is not available, reasonably conservative estimates based on data from other nearby locations may be used.

2.3.3 Recognised methods of statistical analysis should be used to describe the random nature of the environmental conditions. Seastates will normally be defined in terms of the significant wave height (H_S), spectral peak period (T_P) and corresponding return probability.

2.3.4 The form in which the wave information is available, is dependent on the amount and quality of data available for the particular location in question. This may range from a joint distribution of H_S and T_P with directional information to an omnidirectional design value for H_S with an estimated period. The design method presented in section 3. will accept wave input of varying degrees of sophistication.

2.3.5 The peak period (T_P) will depend on fetch and depth limitations as well as duration of the seastates. If no other information is available for the peak period, then the following relationship may be used for the upper limit:

$$T_P = \sqrt{(250\, H_S/g)}$$

If a joint distribution of H_S and T_P is available, then the combination of H_S - T_P which gives the most extreme near-bottom conditions should be selected.

2.3.6 The directional distribution of the wave conditions may be accounted for when selecting the design wave-induced particle velocity. Normally extreme seastates from different directions will need to be considered. If no directional wave information is available then the extreme wave conditions should be assumed to act perpendicular to the axis of the pipeline.

2.3.7 The short crestedness of the waves may be accounted for when selecting the design wave-induced particle velocity. If no site specific information is available, then this may be taken into account by consideration of the energy spreading away from the main direction of wave propagation.

2.3.8 The wave-induced particle velocity to be used in the stability design analysis is represented by the significant value of the near-bottom velocity perpendicular to the pipeline (U_S), and the corresponding mean zero up-crossing period (T_u).

2.3.9 When calculating U_S and T_u, the most appropriate formulation for the water surface elevation spectrum should be used. For North Sea conditions the Jonswap spectral formulation is recommended. For long-crested seas, the Jonswap spectrum is given by:

$$S_{\eta\eta}(\omega) = \alpha g^2 (\omega)^{-5} \exp\{-5/4(\omega/\omega_p)^{-4}\} \gamma^a$$

$$a = \exp\left\{\frac{-(\omega-\omega_p)^2}{2\sigma^2\omega_p^2}\right\}$$

where

ω	=	angular frequency
ω_p	=	angular frequency of spectral peak
g	=	acceleration due to gravity
α	=	Phillips' constant
σ	=	spectral width parameter
		$\sigma = 0.07$ if $w \le \omega_p$
		$\sigma = 0.09$ if $w > \omega_p$
γ	=	peakedness parameter

2.3.10 U_S and T_u may be calculated by transforming the long-crested water surface elevation spectrum to the bottom and applying a reduction factor to account for wave directionality with respect to the pipe and for short-crestedness of the waves as follows:

$$S_{uu}(\omega) = (\omega / \sinh(kd))^2 \cdot S_{\eta\eta}(\omega)$$

where

$S_{\eta\eta}(\omega)$	=	water surface elevation spectrum (long-crested)
k	=	wave number ($\omega^2 = gk \tanh(kd)$)
ω	=	circular frequency

and

$$U_S = U_S * R$$

$$T_u = 2\pi(m_0/m_2)^{\frac{1}{2}}$$

where

$$U_S^* = 2\sqrt{m_0}$$

$$m_n = \int_0^\infty \omega^n S_{uu}(\omega) \cdot d\omega$$

R = reduction factor

U_S^* and T_u may be obtained through the non-dimensional curves presented in Figs. 2.1 and 2.2. The suitability of first order wave theory when approaching shallow water should be verified.

The reduction factor due to wave directionality and wave spreading given by a cosn function, is given by:

$$R = \left\{ \int_{\theta-\pi/2}^{\theta+\pi/2} \psi(\beta,\theta)\left(\cos^2(\theta_p - \beta)\right) d\beta \right\}^{\frac{1}{2}}$$

where

θ_p	=	direction perpendicular to the pipeline
θ	=	main wave direction
β	=	sub-direction around the main wave direction
$\psi(\beta,\theta)$	=	spreading function, given by :

$$\psi(\beta,\theta) = C \cos^n(\beta - \theta)$$

n	=	spreading exponent (site specific)
C	=	constant chosen such that the integral of R over all wave directions is equal to 1.0

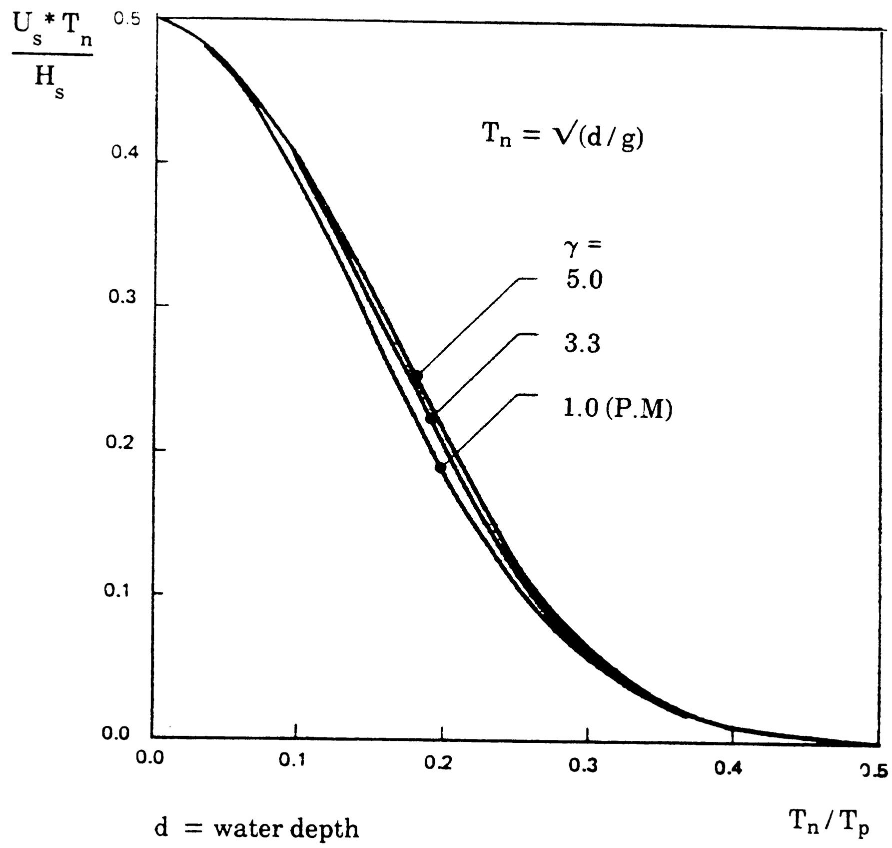
R may be obtained from Fig. 2.3.

2.3.11 The design current velocities should be based on a consideration of the various contributing components such as tidal, storm surge and circulation currents.

2.3.12 The directional distribution of the current velocity may be used in the stability design. If no such information is available, the current should be assumed to act perpendicular to the axis of pipeline.

2.3.13 The current velocity may be reduced to take account of the effect of the bottom boundary layer. This may be achieved using a suitable boundary layer model. The velocity profile in the boundary layer should be integrated over the pipeline diameter to give an effective current velocity. An approximate method of estimating a boundary layer reduction is presented in Appendix A.

2.3.14 It is not recommended to consider any boundary layer effect on wave induced velocities as such effects are normally small and are implicitly included in the applied hydrodynamic force model, which is the basis for the generalized curves. However, the effect of waves on the current boundary layer may be estimated as shown in Appendix A. In special cases of very small diameter pipes where the wave boundary layer may be important, further velocity reductions should be justified by relevant data.



d = water depth
g = gravity constant
γ = Jonswap, peakedness parameter
P.M = Pierson Moskovitz

Fig. 2.1 Significant Water Velocity, U_s*
(Linear Wave Theory. Wave Directionality and Spreading not accounted for)

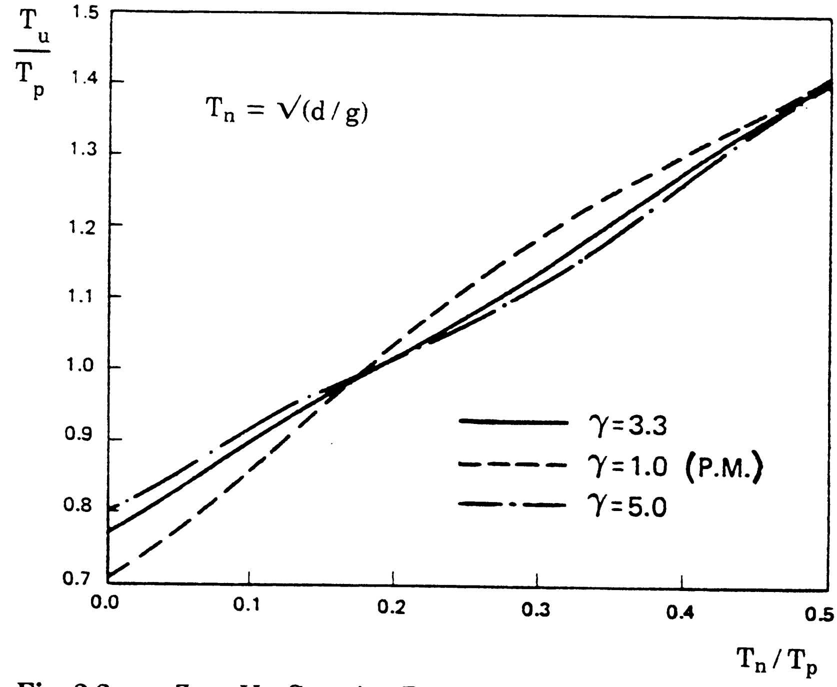


Fig. 2.2 Zero-Up-Crossing Period, T_u

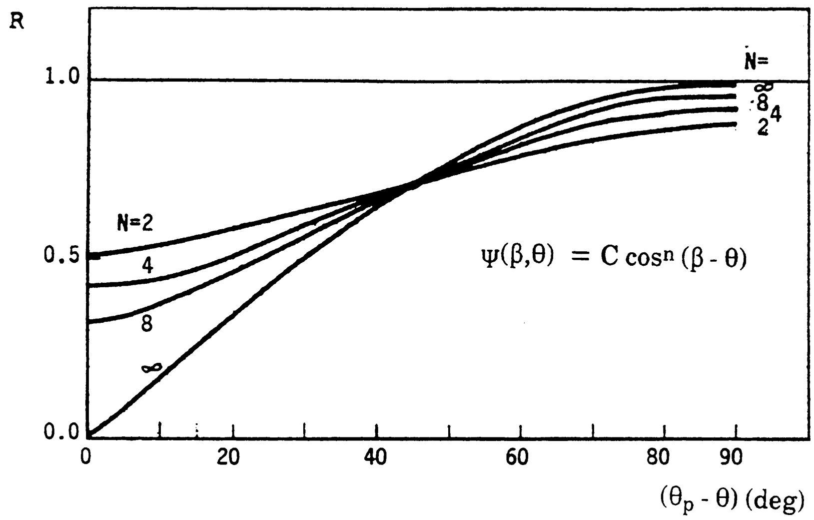


Fig. 2.3 Reduction Factor due to Wave Spreading and Directionality

2.4 Geotechnical Conditions

2.4.1 A site investigation should be performed at suitable intervals along the route of the pipeline. The number of intervals will be dependent on the length of the pipeline and the anticipated variations in geotechnical conditions. Suitable sampling techniques should be used during the site investigation.

Guidelines for site and laboratory testing may be found in Veritas' RP D301 /9/.

2.4.2 From the point of view of stability design the site investigation should provide the following information for the sea bed soil on and immediately below the surface of the sea bed :

- soil classification
- density of soil (sand only)
- strength of soil (clay only)
- possibility of soil slides or liquefaction.

2.5 Topographical and Bathymetric Conditions

2.5.1 A detailed route survey should be performed to provide suitable information on the topographical and bathymetric conditions along the length of the pipeline.
Information relevant to stability design should include :

- obstructions in the form of rock outcrops, boulders or wrecks

- topographical features such as slopes, pock marks or other items which may result in pipeline instabilities.

- variation in water depth along the length of the pipeline.

2.5.2 Further requirements for route surveys may be found in the Det norske Veritas Rules for Submarine Pipeline Systems, 1981 /1/, Sections 2.2.2 and 2.2.3.

2.6 Pipe Data

2.6.1 From the point of view of on-bottom stability, the following pipe data are required :

- outside diameter
- wall thickness
- density of contents at operating pressure
- thickness and density of any corrosion coating
- density of any weight coating
- mechanical properties of the pipeline material

2.6.2 The design method presented in Section 3. allows the pipe to undergo a certain amount of lateral displacement. As a result, areas in which the pipe-line is partly or fully restrained from moving or where the pipeline is to be designed for no movement should be identified. Such areas may include:

- pipeline/riser connections
- pipeline crossings
- subsea valves
- expansion loops
- pipeline emerging from a trench

3. DESIGN METHOD

3.1 General

3.1.1 The design method presented in this section relates to a pipeline resting on the sea bed throughout its' lifetime or prior to some other form of stabilization (eg. trenching, burial, mattresses or other point stabilisation). The stability of the pipeline is then directly related to the submerged weight of the pipeline, the environmental forces and the resistance developed by the sea bed soil. Consequently the aim of the stability design is to verify that the submerged weight of the pipeline is sufficient to meet the required stability criteria.

3.1.2 The following design criteria should be considered during the stability design :

- lateral displacement
- stress/strain in pipe wall
- interaction with lateral buckling due to axial forces
- fatigue damage
- wear and deterioration of the coating
- damage to sacrificial anodes

In general the lateral displacement and the stress/strain experienced by the pipeline will be the governing design criteria. Further consideration of the design criteria is presented in section 4.

3.2 Load Cases

3.2.1 All load cases relevant to the stability of the pipeline should be considered. In general this will normally result in two load cases namely :

- Installation Condition
- Operating Condition

3.2.2 The Installation Condition relates to the period of time after installation when the pipeline is resting on the sea bed prior to trenching or commissioning. Unless the pipeline will be water flooded immediately upon installation, the pipeline should normally be assumed air filled during this condition. For a pipeline which is to be trenched, the installation condition will normally determine the pipeline submerged weight requirements. For the Installation Condition, a minimum specific gravity $(W_S+B) / B = 1.1$ is required (W_S = submerged weight, B = buoyancy).
In general, a water absorption of 5% of concrete weight can be included.

3.2.3 Details of the design storm conditions related to the installation phase are given in 2.2.3.

3.2.4 The Operating Condition relates to the operating phases of the pipeline lifetime. In the stability analysis, the pipeline should be assumed to be filled with contents at normal operating pressure and expected lowest density.

3.2.5 During the operating condition the pipeline may be subjected to lateral displacements, stresses/strains etc. due to extreme wave and current conditions, however the pipeline should still remain serviceable after the storm situation.
The design combination of extreme wave and current should be determined so that its exceedance probability does not exceed 10^{-2}/year (100 year return period).

3.2.6 Details of the environmental conditions to be applied during the operational condition are given in 2.2.2.

3.3 Analysis Methods

3.3.1 There are several analysis methods available on which to base pipeline stability design. Three different methods are considered in this Recommended Practice, namely :

(i) Dynamic Analysis
(ii) Generalized Stability Analysis
(iii) Simplified Stability Analysis

The choice of the above analysis methods is dependent on the degree of detail required in results of the design analysis.

3.3.2 Dynamic Analysis involves a full dynamic simulation of a pipeline resting on the seabed, including modelling of soil resistance, hydrodynamic forces, boundary conditions and dynamic response. Dynamic analysis forms the reference base for the generalized method. It may be used for detailed analysis of critical areas along a pipeline, such as pipeline crossings, riser connections etc., where a high level of detail is required on pipeline response, or for reanalysis of a critical existing line.

3.3.3 The Generalized Stability Analysis is based on a set of non-dimensional stability curves which have been derived from a series of runs with a dynamic response model. This method can be used in either detailed design calculations or preliminary design calculations. The Generalized Stability Analysis method may be used on the sections of the pipeline where potential pipeline movement and strain may be important. The main assumptions of the method are given in section 5.2.

3.3.4 The Simplified Stability Analysis is based on a quasi-static balance of forces acting on the pipe, but has been calibrated with results from the generalized stability analysis. The method generally gives pipe weights that form a conservative envelope of those obtained from the generalized stability analysis.

This method may be used for the vast majority of stability calculations, where the required submerged weight is the only parameter of interest. The method is based on simplified models, consequently it is recommended that this method should not be modified in any way without a full consideration of all the relevant factors, i.e. checking with one of the above two analysis methods.

3.3.5 If the partial burial of a pipeline (implying stable pipe) is to be taken account of in the stability design, then the following should be considered in the stability calculations :

- method to be based on static considerations only, i.e. the pipe should not break out, i.e. be pulled out of the partially buried condition.

- the most probable maximum 100 year near-bottom wave-induced velocity and acceleration normal to the pipeline should be used in the calculation of hydrodynamic forces.

- realistic hydrodynamic force models should be used.

- a soil resistance model which realistically represents the pipe-soil interaction should be used.

3.4 Sinking/Floatation

3.4.1 Buried lines should be checked for possible sinking or floatation. For both liquid and gas lines, sinking should be considered assuming the pipe to be water filled and floatation should be considered assuming the pipe to be gas or air filled.

3.4.2 If the specific weight of the water filled 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 in soils having low shear strength, a consideration of the soil stress may be necessary. If the soil is, or is likely to be liquified, the depth of sinking should be limited to a satisfactory value, by consideration of the depth of liquifaction or the build up of resistance during sinking.

3.4.3 If the specific gravity of the gas or air filled 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 liquified, the specific weight of the gas or air filled pipe should not be less than that of the soil (if burial is required).

3.4.4 Exposed lines resting directly on the sea bed should be checked for possible sinking in the same manner as explained for buried lines, in section 3.4.2 above.

3.5 Overview of the Design Method

3.5.1 The flow diagram presented in Figure 3.1 shows an overview of the design method outlined above.

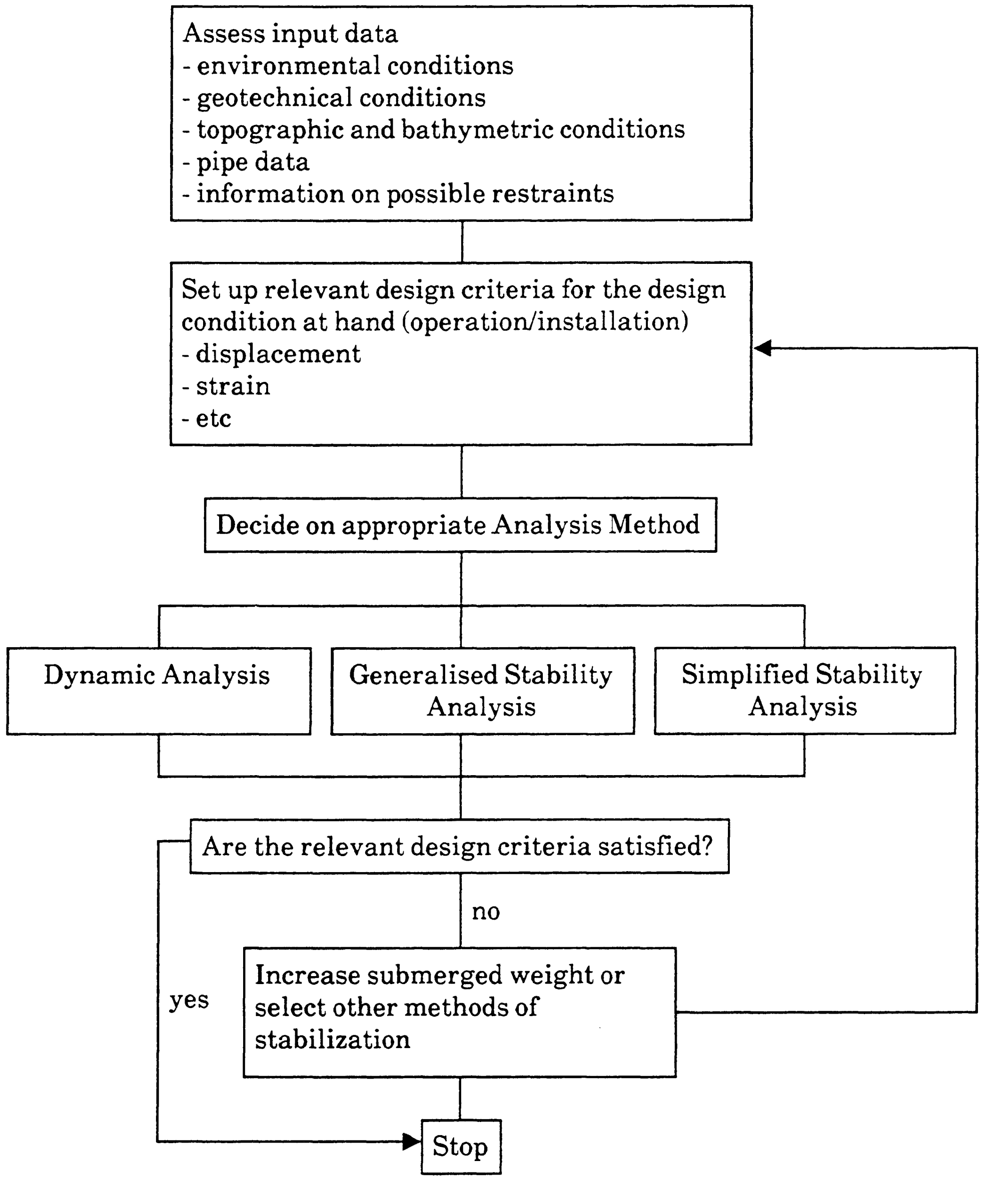


Figure 3.1 Overview of Design Method

4. DESIGN CRITERIA

4.1 General

4.1.1 The criteria to be used in the stability design method outlined in Section 3, will vary depending on the situation under consideration. Careful evaluation of possible failure mechanisms is recommended in each case.

4.1.2 The information given below with regard to design criteria should be viewed as general recommendations. Specific criteria should be considered on a case by case basis.

4.1.3 The design criteria presented below have been related to the design conditions described in Section 3, and also the pipeline zone system used in /1/. The following definitions of the pipeline zones are used :

Zone 1 : the part of the sea bed located more than a certain distance away from the platform or subsea template, normally taken as 500m

Zone 2 : the part of the seabed located close to a platform or subsea template, normally taken as 500m.

4.1.4 For the purposes of these stability guidelines, points on the pipeline such as valve connections, pipeline crossings, Y- or T-connections, expansion loops, etc should in general be considered as Zone 2 pipelines. However, the zone 2 definition normally applies in connection with potential danger for human life, significant pollution or considerable economic consequences.

4.2 Potential Lateral Displacement

4.2.1 The allowable lateral displacement <u>if any</u> will be dependent on several factors, such as :

- national regulations
- sea bed obstructions
- width of surveyed corridor
- distance from platform or other restraint

4.2.2 The specified allowable lateral displacement should be limited to a value not greater than half the width of the surveyed corridor in which the pipe is laid. This implies that the pipeline should not move beyond the allowable corridor.

4.2.3 If no further information is available, then the following may be used for the allowable maximum lateral displacement in the operational condition :

Zone 1	20 m
Zone 2	0 m

This criteria can be relaxed if other relevant data are available.The pipe must also be able to satisfy the other relevant design criteria at the above allowable displacement. For most situations the lateral displacement will be the governing criteria. In general, the strain requirement will also be satisfied when limiting the movement to maximum 20 m. The sensitivity to variations in environmental parameters (wave height/period) should be checked. The allowable displacement criteria refer to a seastate duration of 3 hours at maximum storm intensity.

4.2.4 For Zone 2 pipelines the allowable lateral displacement may be increased above zero if the effect of the displacement can be acceptably accommodated by the pipeline and the supporting structure (eg. riser connection)

4.2.5 The allowable lateral displacement for the installation condition is dependent on the time period between laying and commissioning, and should be decided on a case by case basis. However, if the recommendations with respect to environmental conditions given in 2.2.3 are followed an allowable displacement of 5 m is suggested.

4.3 Bending Strain

4.3.1 Due to the development of bending moments at points of fixity along the pipeline, as a result of the lateral displacement, the bending strains experienced by the pipe should be evaluated during the stability design.

4.3.2 For known points of fixity, such as riser connections, subsea valves, subsea templates etc, the effect of the lateral pipe displacement should be evaluated for both the pipeline and the restraining structure.

4.3.3 Any part of the pipeline may bend as a result of local variations in seabed and pipe properties, and the bending strain criterion should be satisfied at any point assuming a fixed end restraint. This applies to the generalized method.

4.3.4 When evaluating the bending effects resulting from the lateral displacement of the pipeline, consideration should be given to the following :

- excessive straining
- ovalization
- buckling

Reference is made to /1/ for the limiting criteria for the above.

4.3.5 If no further information is available, the limiting strain criteria may be taken as $7.5/(D/t)^2$, with a maximum strain limit of 1 percent, see Fig. 4.1. The limiting strain values relate to total (static + dynamic) accumulated elasto/plastic strain, not elastic strain. Consequently, when using this strain criteria the ductility of the pipeline material should be taken into account. The limiting strain values may only be used if a full dynamic analysis applying nonlinear elasto/plastic elements is used. If nonlinear strain is used in design some check of pipe behaviour in the ductility level events is required.

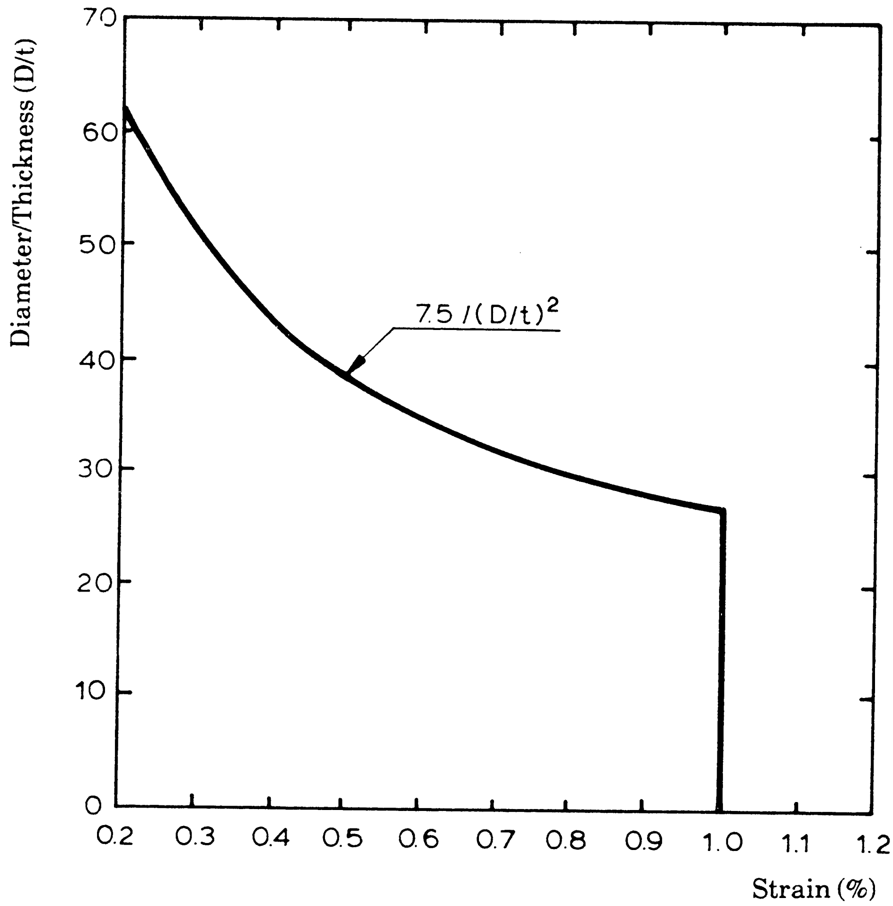


Fig. 4.1 Limiting Strain

4.3.6 If a large number of strain cycles are anticipated due to the lateral movement of the pipeline, these should be included in the fatigue assessment of the pipeline, as outlined in section 4.2.4 of /1/.

4.4 Other Relevant Criteria

4.4.1 The lateral movement of the pipeline should not result in significant damage to the external pipeline coating, as a result of abrasion from the seabed.

4.4.2 The lateral movement of the pipeline should not result in damage to sacrificial anodes attached to the pipeline.

4.4.3 The lateral movement of the pipeline should not interfere with other pipelines or other subsea installations.

5. ANALYSIS METHODS

5.1 Dynamic Analysis

5.1.1 Dynamic analysis involves the dynamic simulation of a section of pipeline under the action of waves and current. The full dynamic analysis will only be used in specialized circumstances. Due to the nonlinear behaviour of the pipeline, a time domain solution is recommended.

5.1.2 The following should be accurately modelled :

- wave spectrum and corresponding realistic time series
- current velocity at the sea bed
- structural behaviour of pipe
- hydrodynamic forces
- soil resistance forces
- restraints (eg riser connections, etc)

5.1.3 The dynamic simulation should be performed for a complete sea state. If no information is available on the duration of sea states then a sea state duration of 3 hours is recommended.

5.1.4 The length of pipeline modelled should be sufficient to adequately represent the real situation. This implies that different lengths (e.g. 250-1000 m) should be analyzed to determine sensitivity of results.

5.1.5 If the strain response of the pipeline is critical, i.e., above the proportionality limit then it is recommended to model the non-linear stress/strain behaviour of the pipe material.

5.1.6 A method of realistically representing the hydrodynamic forces experienced by the pipeline should be used. Two such methods are those presented in /10/ and /12/.

5.1.7 It is recommended that the method of modelling the soil resistance forceincludes both the effect of friction between the pipe and the soil, and the resistance due to the penetration of the pipe into the soil. One such model is that developed by Wagner et. al. /5/.

5.2 Generalized Stability Analysis

5.2.1 This method of pipeline stability analysis is based on generalization of the results from a Dynamic Analysis, through the use of a set of non-dimensional parameters and for particular end conditions.
The limitations of the method are given in section 5.2.5.
In Appendix B a calculation example is given.
The method is based on the work published in /8/ and /11/.

The major assumptions are as follows:

- hydrodynamic forces modified for wake effects
- no initial embedment
- no prior load history
- rough pipe
- passive soil resistance due to partial penetration of the pipe into the soil under cycle loading is included.

- medium sand soil
- JONSWAP wave spectrum
- no reduction of hydrodynamic forces due to pipe penetration

5.2.2 The generalized response of the pipeline in a given sea state is principally controlled by the following non-dimensional parameters.

Load parameter (significant KC-number)	$K = U_S T_u / D$
Pipe weight parameter	$L = W_S / 0.5\rho_W D U^2_S$
Current to wave velocity ratio	$M = U_C / U_S$
Relative soil weight (for sand soil)	$G = (\rho_S - \rho_W) / \rho_W = \rho_S / \rho_W - 1$
Shear strength parameter (for clay soil)	$S = W_S / (DS_u)$
Time parameter	$T = T_l / T_u$

where:

- U_S and T_u are the near bottom significant velocity normal to the pipeline and zero up-crossing period, respectively, due to a given surface sea state.

- U_C is the steady current component in the boundary layer normal to the pipeline. An average value integrated over the diameter of the pipeline is used.

- W_S and D are the submerged pipe weight and outer diameter, respectively.

- ρ_W and ρ_S are the mass density of sea water and sand soil material, respectively.

- S_u is the undrained shear strength of a clay soil.

- T_l is the duration of the sea state in seconds.

5.2.3 Pipeline on sand soil

For a pipeline on sand soil the generalized response is given in terms of lateral displacement for a free section and bending strain corresponding to a fixed point along the pipeline. The displacement includes the expected net displacement plus one standard deviation plus the maximum amplitude of displacement in a single wave. The Design Method determines the pipe weight that satisfies the given criteria for displacement and strain in the design sea state.

5.2.3.1 Figures 5.1 to 5.6 give the generalized weight parameter L, versus K for specific M values, solid lines. Figures are given for values of the scaled lateral pipe displacement, $\delta = Y / D$, of 10, 20 and 40 and based on sea states with 500 and 1000 wave periods, i.e. $T = T_l / T_u = 500, 1000$. For a given pipeline with a specified design wave environment, interpolation within these figures will give the necessary submerged pipe weight to satisfy the design criteria for lateral displacement given in Section 4.2. A few iterations on the curves may be necessary to give satisfactorily accuracy in the design weight.

Net movement predictions may be sensitive to small changes in input parameters, thus sensitivity of results to each parameter should be checked.

5.2.3.2 In Fig. 5.7 is also given the generalized weight parameter L for a complete stable pipe ($\delta = 0$) on sand soil.

5.2.3.3 The bending strain in the pipe at a fixed point along the pipeline section is also found from Figures 5.1 to 5.6 (dotted lines). The engineering strain is calculated from the generalized strain under the assumption of a thinned walled pipe:

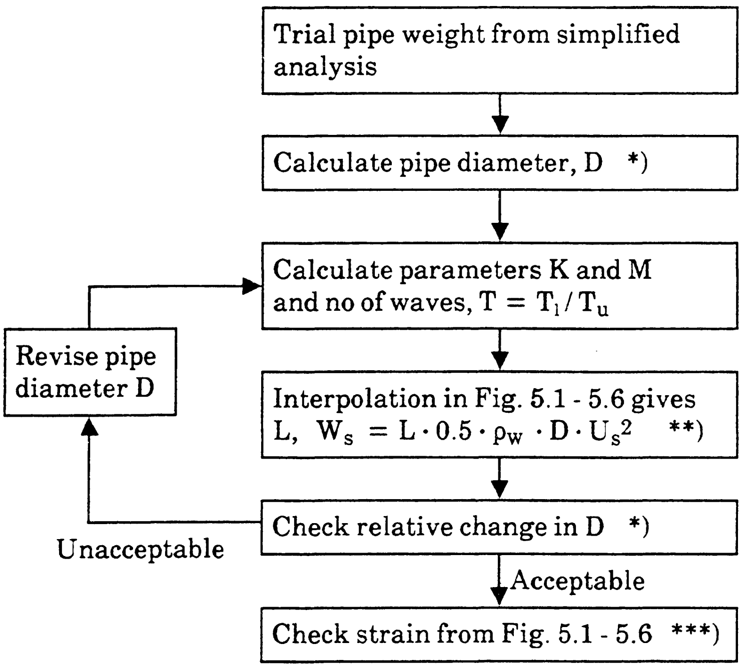
$$\varepsilon = \left(\frac{8\, W_s\, D}{\pi\, E\, t_s\, D_s} \right)^{\frac{1}{2}} \varepsilon'$$

where

ε	-	strain
W_s	-	submerged weight
D_s	-	steel pipe diameter
t_s	-	steel pipe thickness
E	-	modulus of elasticity
D	-	pipeline outside diameter

The maximum allowed strain that can be accepted as a result from the Generalized Stability Method is 0.2 %. This is due to the use of linear elastic material properties during the development of this method. If the proportionality level (ε = 0.2 %) is exceeded a more refined analysis is recommended to study the bending strain based on a nonlinear material modelling. The use of 20 m maximum movement will generally ensure that the strain criteria is also met.

5.2.3.4 The Generalized Stability Analysis Method for sand soil is illustrated in the following flow chart:



*) $$D^2 = \frac{1}{\rho_c - \rho_w} \cdot \left[\frac{W_s}{0.25\pi g} + D_i^2 \cdot (\rho_{st} - \rho_i) + D_s^2 \cdot (\rho_{cc} - \rho_{st}) + D_{cc}^2 \cdot (\rho_c - \rho_{cc}) \right]$$

**) 5% water absorption can be assumed when calculating concrete weight

***) If strain limit (ε = 0.2 %) is exceeded:

1. Increase lateral pipe weight or
2. Apply more refined analysis (non-linear material model)

Where

ρ_c	-	density of concrete coating
ρ_{cc}	-	density of corrosion coating
ρ_{st}	-	density of steel material
ρ_i	-	density of internal content
ρ_w	-	density of sea water
D_i	-	internal pipe diameter
D_s	-	outer steel pipe diameter
D_{cc}	-	outer steel pipe diameter incl. corrosion coating.

5.2.3.5 The design curves given in Figures 5.1 to 5.6 relate to a pipeline resting on a medium sand soil ($\rho_s = 1860$ kg/m^3). For sand soil with different density, the calculated submerged weight, W_s, should be multiplied by a correction factor according to Fig. 5.8 given as a function of the relative soil weight, $G = \rho_s / \rho_w - 1$.

Note that the curves may be used with any consistent set of units.

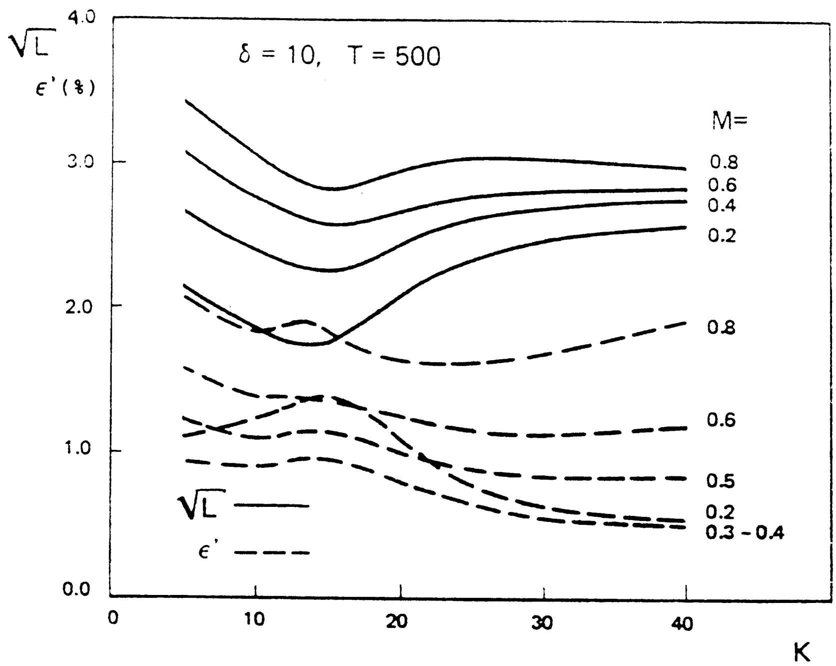


Fig. 5.1 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

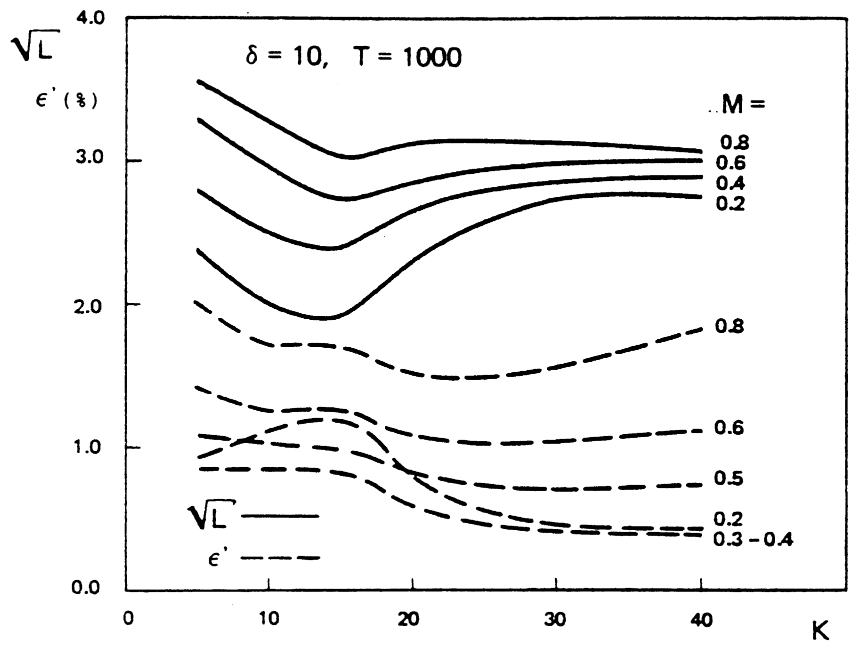


Fig. 5.2 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

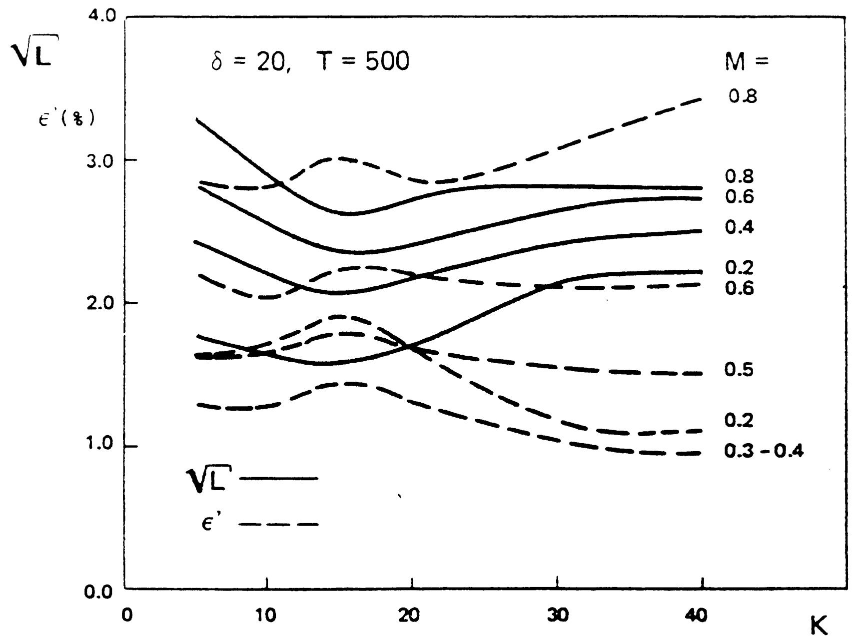


Fig. 5.3 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

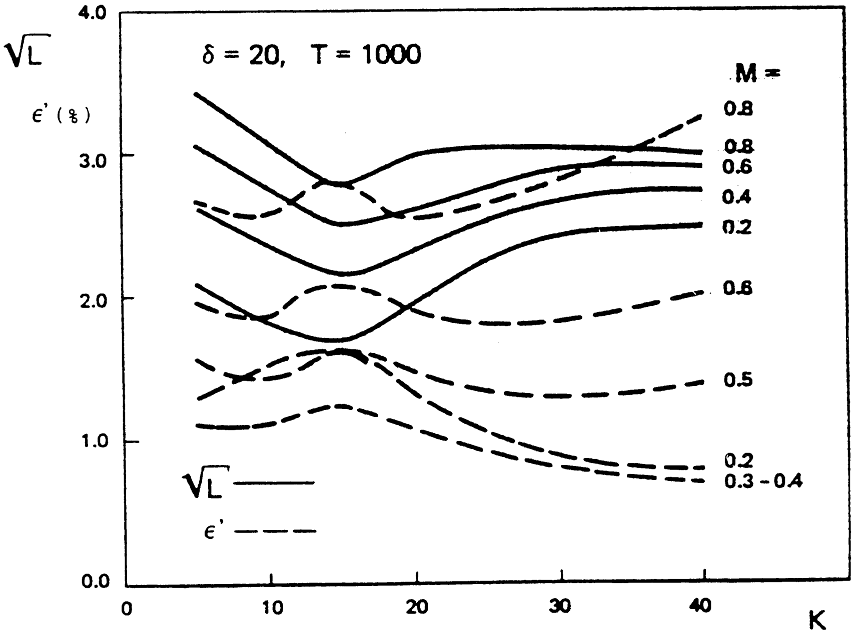


Fig. 5.4 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

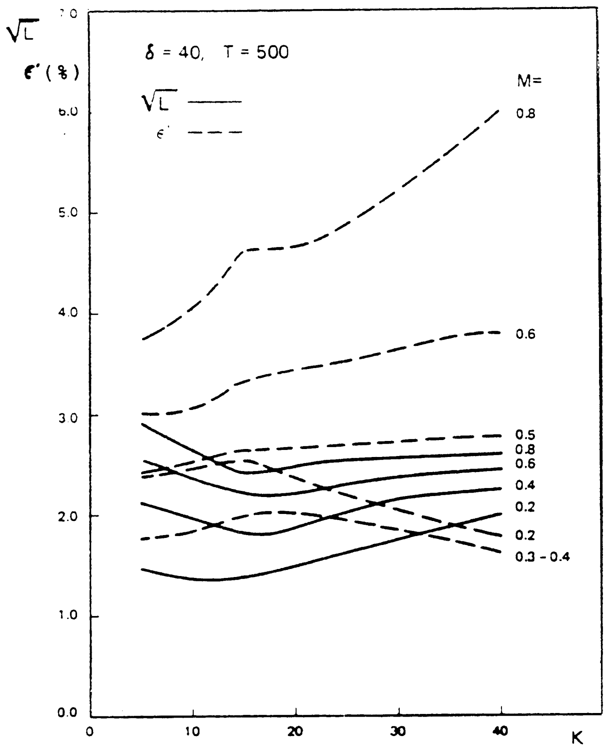


Fig. 5.5 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

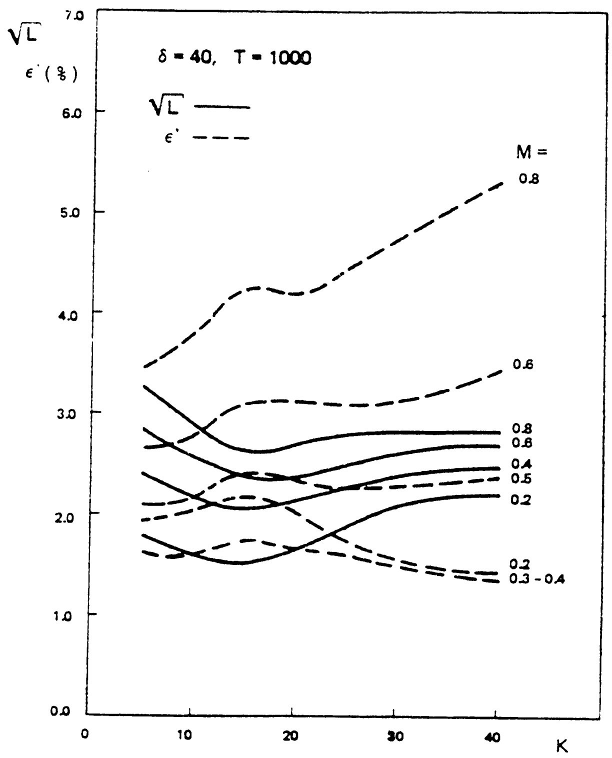


Fig. 5.6 Generalized Weight Parameter L and Bending Strain versus K for various M Values.

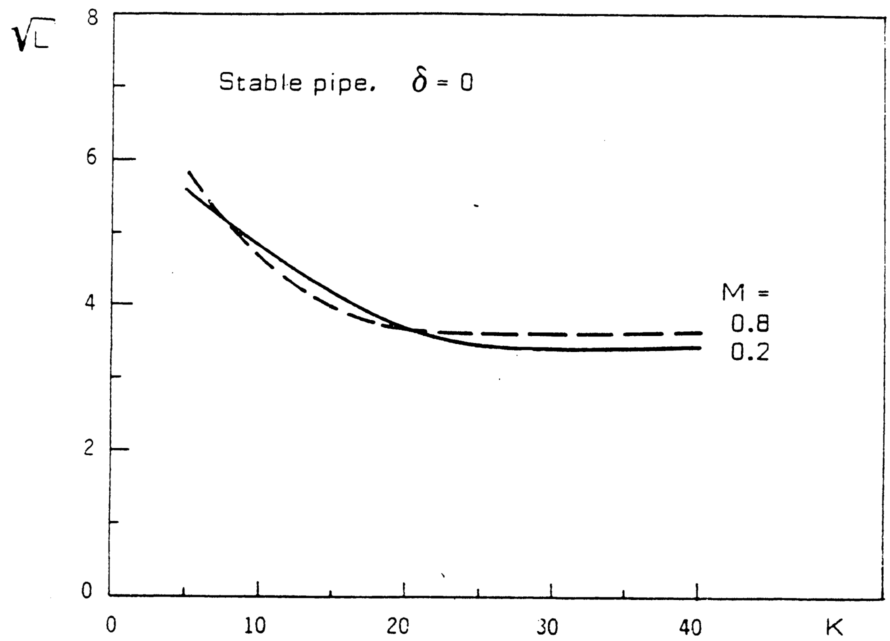


Fig. 5.7 Generalized Weight Parameter L for a Stable Pipe (δ = 0)

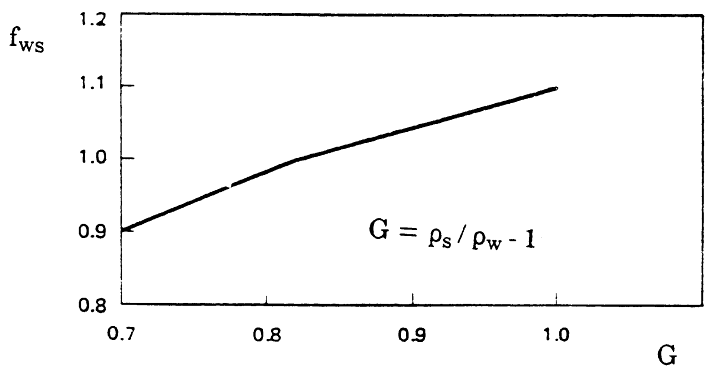


Fig. 5.8 Correction Factor on Weight W_s versus Soil Density.

5.2.4 Pipeline on clay soil
For a pipeline on clay soil the Design Method determines the pipe weight that satisfies absolute stability (no breakout) for the extreme wave in the design sea state.

5.2.4.1 Figure 5.9 and 5.10 give the basis for the stability calculations for a pipeline on clay soil. Design according to this figure will ensure that the pipe is stable for the extreme wave combination in the specified design sea state. The figure gives the critical weight parameter, L_{cr}, as a function of dimensionless soil strength parameter, S/L, as a function of K and M. A few iterations on the curves are necessary to give satisfactory accuracy in the design weight.

5.2.4.2 A safety factor of 1.1 should be applied on the calculated stable weight as shown in 5.2.4.3.

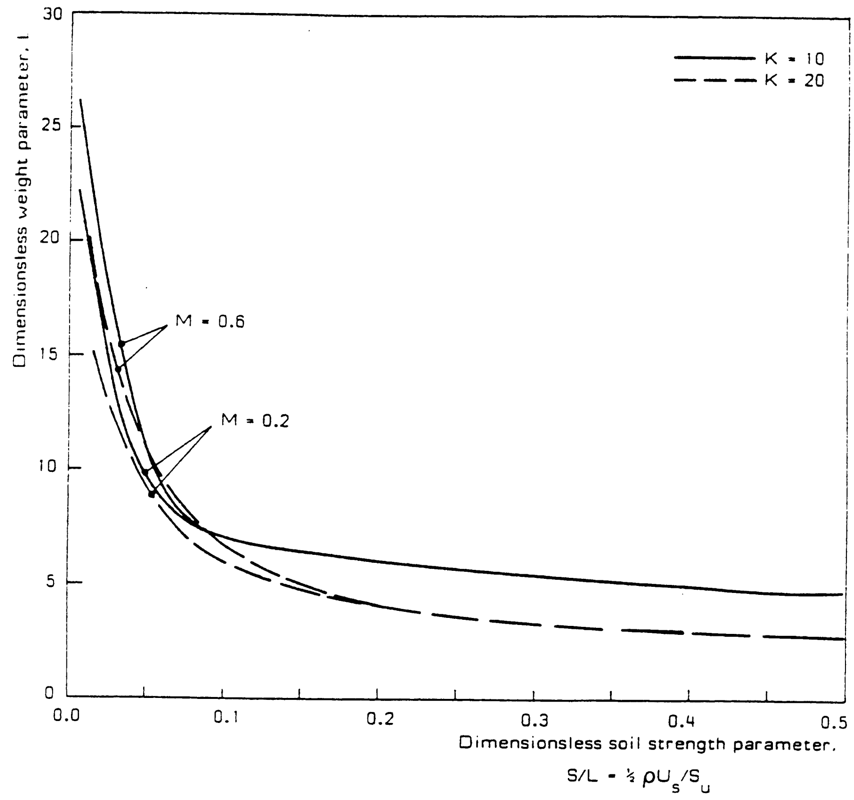


Fig. 5.9 Stability Curves for Clay (K = 10 and 20)

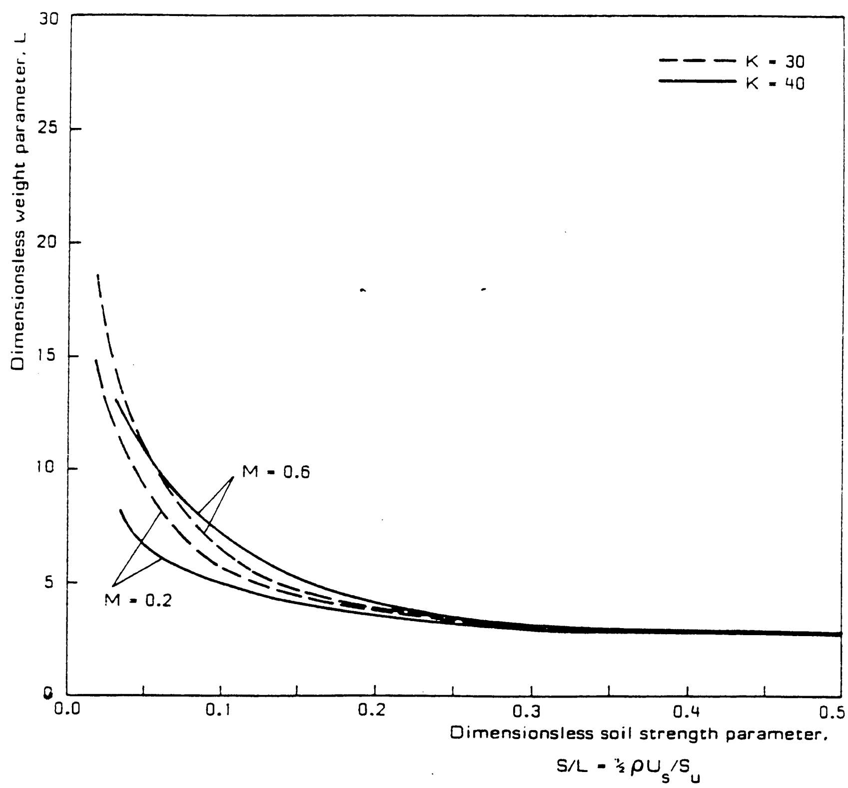
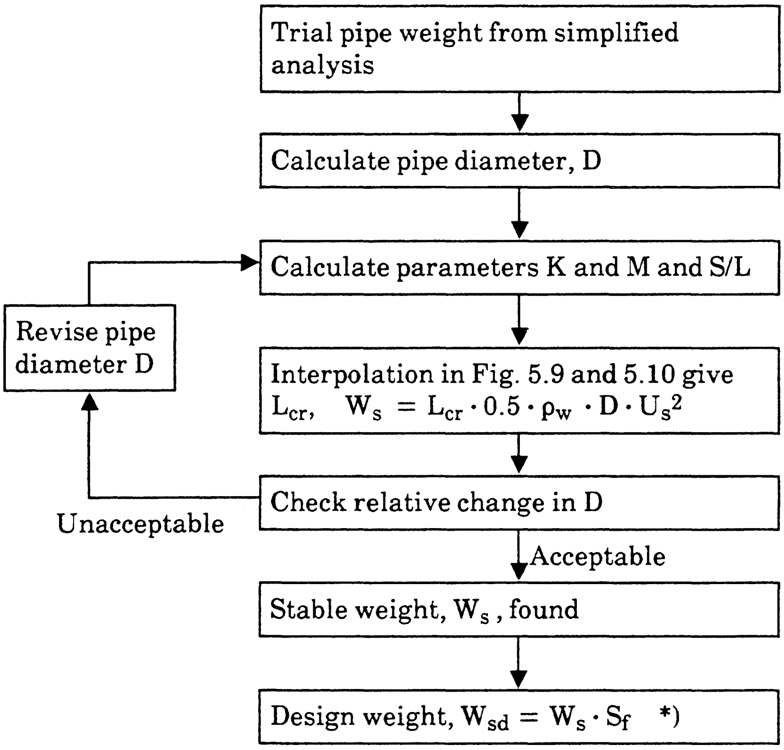


Fig. 5.10 Stability Curves for Clay (K = 30 and 40)

5.2.4.3 The Generalized Stability Analysis Method for clay soil is illustrated in the following flow chart.



*) The stable weight, W_s, calculated based on Figs. 5.9 and 5.10 must be multiplied with a safety factor $S_f = 1.1$ to arrive at design weight.

5.2.5 The Generalized Stability Analysis presented above is valid for the following range of parameters:

4	$< K < 40$	
0	$< M < 0.8$	
0.7	$< G < 1.0$	(for sand soil)
0.05	$< S < 8.0$	(for clay soil)
	$D \geq 0.4$ m	

The reason for the above validity in K and M is related to the use of the wake force model /10/ in the dynamic simulation program from which the method was derived. The sand and clay soil models have been tested within the above specified ranges. The method presented above should be limited to pipeline diameters (outer) ≥ 0.4 m, because the calibration has been performed for larger diameters.
For conditions outside the above range, the use of the simplified Analysis Method outlined in section 5.3 is recommended.

5.3 Simplified Static Stability Analysis

5.3.1 The purpose of this section is to outline a simple method of stability design suitable for checking stability in all normal design situations.In Appendix B a calculation example is given.

5.3.2 The method is based on a static stability approach, which ties the classical static design approach to the generalized stability method through a calibration of the classical method with generalized stability results. A calibration factor (F_w) is included, which has been developed from pipelines designed with a lateral displacement of up to 20 m. The results are thus brought into agreement even though the forces calculated for any given case are not necessarily physically realistic (ref. e.g. constant C_D = 0.7 instead of as function of R_e, K, roughness etc.).

5.3.3 The soil friction factors to be used in conjunction with the simple design method are to be based on soil classification as follows :

Soil Type	Friction Calibration Factor
Sand	0.7
Clay	given in Figure 5.11

5.3.4 The friction factors presented for clay soils in Figure 5.11 were developed as part of the simplified method and consequently must only be used in conjunction with the simplified design method.

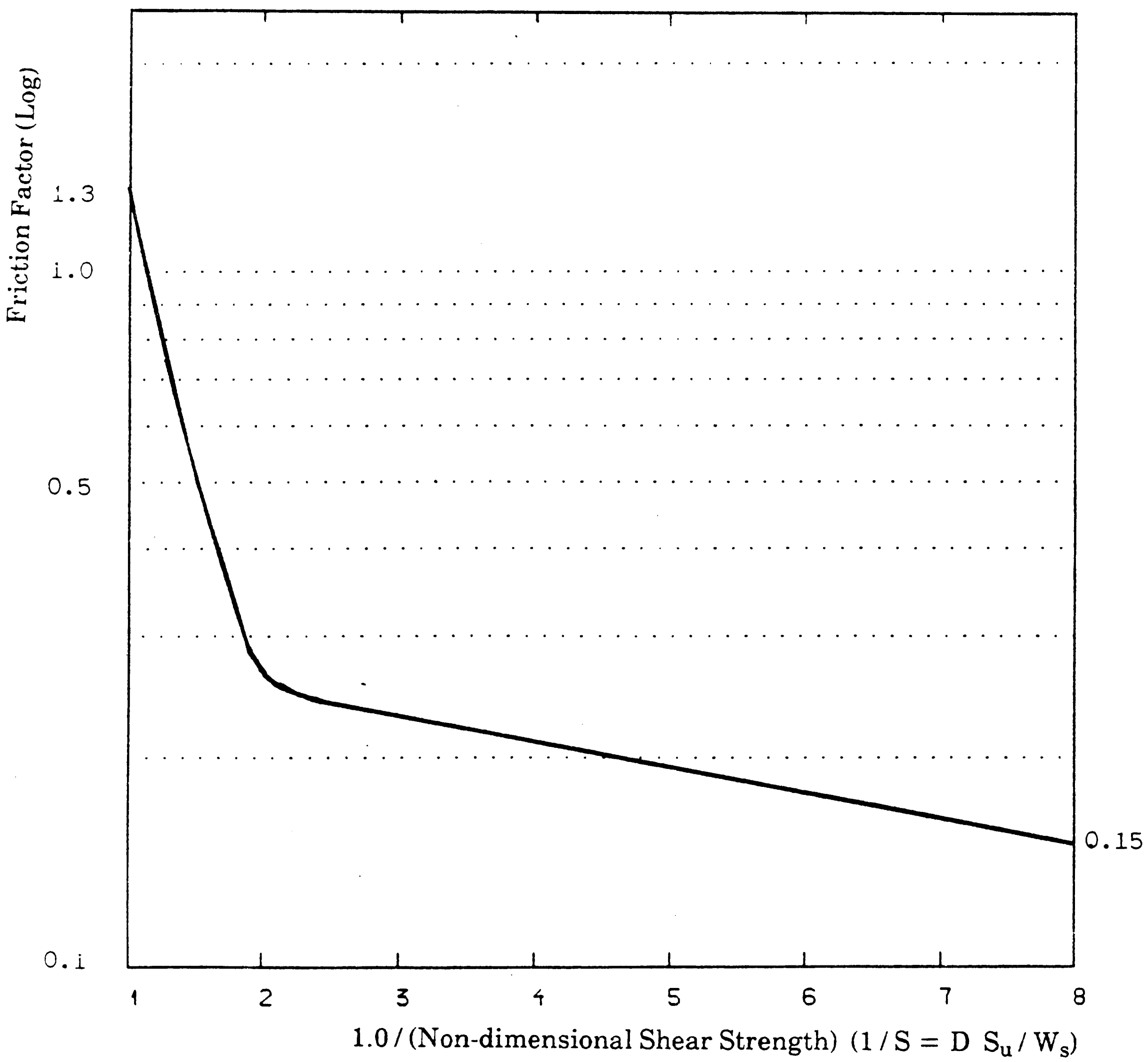


Fig. 5.11 Recommended Friction Factors for Clay (Simplified Design Method)

5.3.5 Stability in this quasi-static method is given by the following expression :

$$[\,W_S/F_W - F_L\,]\,\mu \geq F_D + F_I$$

where

W_S = submerged weight of the pipe
F_W = calibration factor
μ = soil friction factor
F_L = lift force
F_D = drag force
F_I = inertia force

5.3.6 The limiting value of submerged weight can then be found from :

$$W_S = \left[\frac{(F_D + F_I) + \mu \cdot F_L}{\mu}\right]_{max} \cdot F_W$$

5.3.7 The variation of the calibration factor, F_W with K and M is shown in Figure 5.12. A safety factor of 1.1 is inherent in the calibration factor F_W.

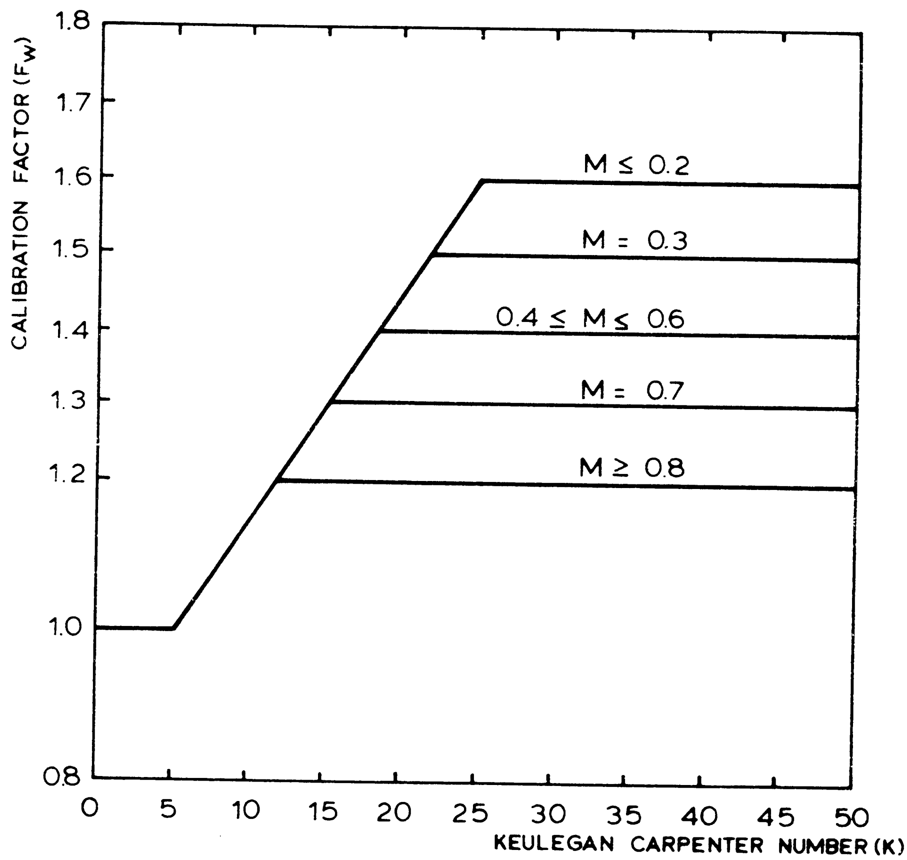


Fig. 5.12 Calibration Factor, F_W as Function of K and M

5.3.8 When using the calibration factor F_w, to calculate W_S, the hydrodynamic forces acting on the pipe (F_L, F_D and F_I) may be estimated from the following expressions :

$$F_L = \tfrac{1}{2} \cdot \rho_w \cdot D \cdot C_L \cdot (U_S \cdot \cos\theta + U_C)^2$$

$$F_D = \tfrac{1}{2} \cdot \rho_w \cdot D \cdot C_D \cdot |(U_S \cdot \cos\theta + U_C)| \ (U_S \cdot \cos\theta + U_C)$$

$$F_I = (\pi \cdot D^2) / 4 \cdot \rho_w \cdot C_M \cdot A_S \cdot \sin\theta$$

where

ρ_w	=	mass density of water
D	=	total outside diameter of the pipe
C_L	=	lift force coefficient (C_L = 0.9)
C_D	=	drag force coefficient (C_D = 0.7)
C_M	=	inertia force coefficient (C_M = 3.29)
U_S	=	significant near-bottom velocity amplitude perpendicular to the pipeline
U_C	=	current velocity perpendicular to the pipeline
A_S	=	significant acceleration perpendicular to the pipeline (= $2\pi\, U_S / T_u$)
θ	=	phase angle of the hydrodynamic force in the wave cycle.

5.3.9 Information on the estimation of the water particle characteristics is given in section 2.

5.3.10 Values for the soil friction factor are based on the soil classification of the seabed. Recommended soil friction factors are given in 5.3.3.

5.3.11 For $K > 50$ and $M \geq 0.8$, (i.e. approaching stationary current), a constant calibration factor F_w = 1.2 may be applied.

5.3.12 For subcritical and critical flow regime, i.e $R_e < 3.10^5$, and $M \geq 0.8$, realistic hydrodynamic coefficients, valid for stationary current (C_D = 1.2, C_L = 0.9), should be applied to determine hydrodynamic forces for the stability calculations.

6. REFERENCES

/1/ Rules for Submarine Pipeline Systems, Det norske Veritas, 1981

/2/ Wolfram Jr., W.R., Getz, J.R. and Verley, R.L.P.: "PIPESTAB Project: Improved Design Basis for Submarine Pipeline Stability", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5501, Houston, 1987.

/3/ Holthe, K., Sotberg, T. and Chao, J.C.: "An Efficient Computer Model for Predicting Submarine Pipeline Response to Waves and Current", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5502, Houston, 1987.

/4/ Verley, R., Lambrakos, K.F. and Reed, K.: "Prediction of Hydrodynamic Forces on Seabed Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5503, Houston, 1987.

/5/ Wagner, D., Murff, D., Brennodden, H. and Sveggen, O.: "Pipe-Soil Interaction Model", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5504, Houston, 1987.

/6/ Slaattelid, O.H., Myrhaug, D. and Lambrakos, K.F.: "North Sea Bottom Boundary Layer Study for Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5505, Houston, 1987.

/7/ Tryggestad, S., Kollstad, T., Nilsen, J. and Selanger, K.: "Measuring Technique and Field Data for Pipeline Stability Studies", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5506, Houston, 1987.

/8/ Lambrakos, K.F., Remseth, S., Sotberg, T. and Verley, R.: "Generalized Response of Marine Pipelines", Proc. of Nineteenth Offshore Technology Conference, Paper No. 5507, Houston, 1987.

/9/ RP D301: Site Investigation for Offshore Structures, Veritec, 1987.

/10/ Lambrakos, K.F., Chao, J.C., Beckman, H. and Brannon, H.R. "Wake Model of Hydrodynamic Forces on Pipelines," Ocean Engineering, Vol. 14, No. 2, pp 117-137

/11/ Sotberg T., Lambrakos, K.F., Remseth, S., Verley, R.L.P., and Wolfram, W.R., "Stability Design of Marine Pipelines", BOSS' 1988.

/12/ Jacobsen, V., Bryndum, M.B., and Tsahalis, D.T., "Prediction of irregular wave forces on submarine pipelines". Proc. 7th OMAE, Houston, Febr. 1988.

APPENDIX A

APPROXIMATE METHOD TO CALCULATE BOUNDARY LAYER REDUCTION

A.1 INTRODUCTION

This Appendix presents an approximate method for calculating a boundary layer reduction factor which may be applied to the steady current velocity used in the calculation of pipeline stability.

The method can be applied to both steady current and combined wave/steady current flow conditions over fixed bottoms of granular materials. The effect of the seabed roughness and wave/current interaction are accounted for in this simplified procedure. However other effects such as sediment transport and ripple formation are not included. These effects will in general lead to greater velocity reductions.

The calculation procedure outlined below is based on work reported in /A1/.

A.2 VELOCITY PROFILE

The steady flow is described as a logarithmic velocity profile of the form:

$$U(z) = \frac{U^*}{\kappa} \ln\left[\frac{z+z_o}{z_o}\right] \tag{A.1}$$

where

U^*	=	friction velocity
κ	=	von Karman's constant (= 0.4)
z	=	elevation above the seabed
z_0	=	bottom roughness parameter

The average steady velocity acting over the pipe is appropriate for use in determining the hydrodynamic forces on the pipe.

The average velocity acting over a pipe of diameter D, is given by :

$$U_D = \frac{1}{D}\int_o^D U(z)\,d(z) \tag{A.2}$$

The ratio between this average velocity and a known reference steady velocity, U_r, at some height z_r above the seabed is given by :

$$\frac{U_D}{U_r} = \frac{1}{ln(z_r/z_o+1)} \cdot \frac{1}{D} \cdot \int_o^D ln\left[\frac{z+z_o}{z_o}\right] dz$$

$$= \frac{1}{ln(z_r/z_o+1)} \cdot \left\{\left[1+z_o/D\right] ln\left[D/z_o+1\right] - 1\right\} \tag{A.3}$$

z_r may be taken as 3 m if no other information is available.

A.3 CURRENT FLOW

For the case of steady current flow acting alone the effect of the seabed roughness (grain size) may be accounted for in the boundary layer velocity estimation.

The mean grain size, d_{50}, is related to Nikuradse's equivalent sand roughness parameter, K_b, and to the bottom roughness by :

$$K_b = 2.5\, d_{50} \tag{A.4}$$

$$z_o = \frac{K_b}{30}$$

The mean grain size may be estimated from Table A.1.

The following procedure may then be followed to estimate the boundary layer reduction.

/1/ Estimate the mean grain size (d_{50}) from soil samples or from Table A.1

/2/ Calculate K_b, z_o, D/z_o and z_r/z_o

/3/ Calculate the velocity reduction factor, U_D/U_r from equation (A.3)

A.4 COMBINED WAVE AND CURRENT FLOW

The non-linear interaction between the wave and the current flow results in a modification of the steady velocity profile. This modification of the steady flow component is attributed to an apparent increase in the seabed roughness.

The apparent roughness (z_{oa}) is dependant on the ratio between the wave induced velocity and the steady current velocity, given by :

$$U_s/U_r$$

where
U_s = significant horizontal wave induced velocity at the reference distance (z_r) above the seabed.

The apparent roughness is also dependant on the relative roughness parameter given by :

$$A_o/K_b$$

where
A_o = orbital semi-diameter of the water particles associated with U_s, i.e. $A_o = (U_s T_p/2\pi)$

The determination of the boundary layer reduction factor is based on similar assumptions to the steady flow case. In addition it is assummed that the bottom material does not form into ripples, and that the steady current and the wave flow are co-directional. These are both conservative assumptions. The apparent roughness (z_{oa}) can be obtained from Figures A.1 to A.7 and the boundary layer reduction factor then obtained from equation (A.3), with z_{oa} substituted for z_o.

This method is valid provided the following are satisfied:

$$z_r > 0.2\,A_o \left[\frac{A_o}{K_b} \right]^{-0.25}$$

$$\frac{A_o}{K_b} \geq 30$$

$$\frac{U_s}{U_r} \geq 1$$

The following procedure may be adopted to estimate the boundary layer reduction factor for combined wave and current flows :

1. Estimate the mean grain size (d_{50}) from soil samples or from Table A.1

2. Calculate K_b, z_o, z_r/K_b and K_b/A_o

3. Check that the parameters Z_r, A_o/K_b and U_s/U_r are within the ranges of validity.

4. From Figures A.1 to A.5 determine the appropriate value of z_{oa}/z_o and hence z_{oa}. This may require interpolation between figures for various values of z_r/K_b.

5. Calculate the velocity reduction factor, U_D/U_r from equation (A.3) substituting z_{oa} for z_o.

Table A.1 Grain size for seabed materials

Seabed	Grain Size d_{50} (mm)	Roughness z_0 (m)
Silt	0,0625	5.21 E-6
V. Fine Sand	0,125	1.04 E-5
Fine Sand	0,25	2.08 E-5
Medium Sand	0,5	4.17 E-5
Coarse Sand	1,0	8.33 E-5
V. Coarse Sand	2,0	1.67 E-4
Gravel	4,0	3.33 E-4
Pebble	10,0	8.33 E-4
	25,0	2.08 E-3
	50,0	4.17 E-3
Cobble	100,0	8.33 E-3
	250,0	2.08 E-2
Boulder	500,0	4.17 E-2

A.5 EXAMPLES

/1/ Current Flow

A pipeline with an external diameter of 0.5m is to be placed in a tidal stream with a velocity of 1m/s measured at 5m above the seabed. The seabed material is coarse sand. Find the average velocity acting across the diameter of the pipeline.

From the problem formulation the following are given:

$D = 0.5m$
$U_r = 1m/s$
$z_r = 5m$

For coarse sand the following can be extracted from Table A.1:

$d_{50} = 1mm$
$z_0 = 8.33\text{ E-5 m}$

This gives:

$D/z_0 = 6000$
$z_r/z_0 = 60000$

and substituting into equation (A.3) gives :

$U_D/U_r = 0.7$ giving $U_D = 0.7m/s$

The average velocity across the pipe diameter is 0.7m/s

/2/ Combined Wave and Current Flow

A pipeline with an external diameter of 0.5m is to placed on the seabed with a water depth of 30m. The design wave conditions for the area show a significant wave height of 8m with a peak period of 13s. The design current velocity is 1m/s measured at 5m above the seabed. The seabed material is coarse sand.
Find the average steady velocity acting on the pipe .

From the problem formulation the following are given :

$D = 0.5m$
$U_r = 1m/s$
$z_r = 5m$
$H_s = 8m$
$T_p = 13s$
$d = 30m$
$R = 1.0$

Calculating the near-bed significant wave induced particle velocity and associated period at the seabed from Figs. 2.1 and 2.2.

$U_s^* = 1.55m/s$ $T_u = 12.35s$

The amplitude of the horizontal water particle displacement is estimated as:

$$A_o = \frac{U_s * T_u}{2\pi} = 3.05 \text{ m}$$

For coarse sand the following can be extracted from Table A.1:

d_{50} = 1mm
K_b = 2.5 E-3 m
z_0 = 8.33 E-5 m

giving

$z_r / K_b = 2000$

$A_o / K_b = 1220$

Checking the regions of validity :

$$0.2A_o \left[\frac{A_o}{K_b} \right]^{-0.25} = 1.03 \text{ E} - 1 \qquad \text{OK}$$

$A_o / K_b = 1220$ OK

$U_s / U_r = 1.55$ OK

From Figure A.4 for $Z_r / K_b = 2000$

$z_{oa} / z_o = 17.5$ giving z_{oa} = 1.46 E-3 m

The velocity reduction factor is then found from equation (A.3), giving :

$U_D / U_r = 0.6$ and thus U_D = 0.6m/s

<u>The average steady velocity across the pipe is then 0.6m/s</u>

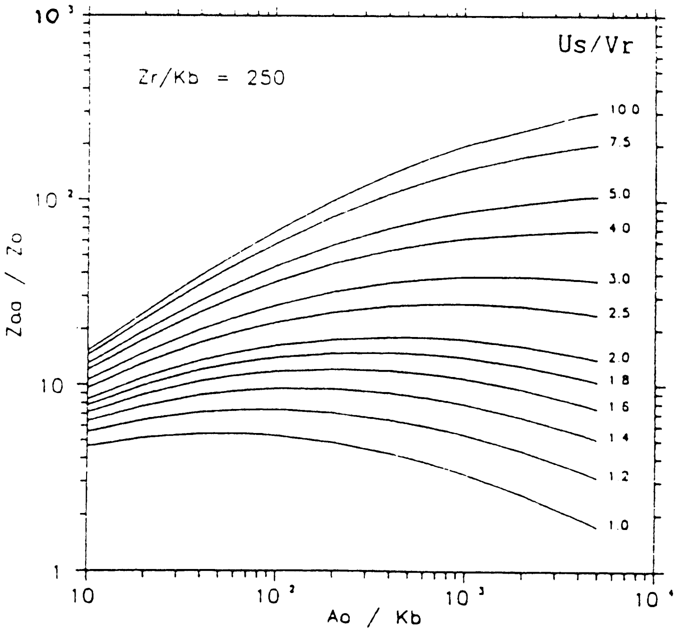


Fig A.1 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 250$)

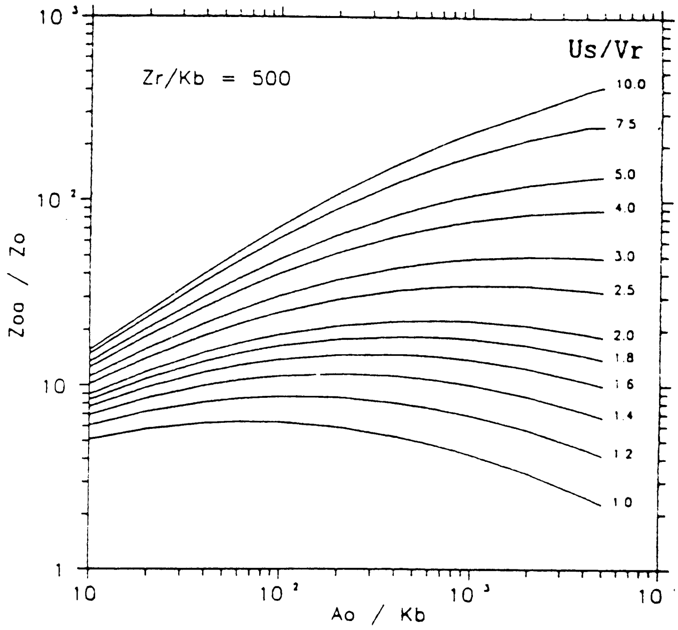


Fig A.2 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 500$)

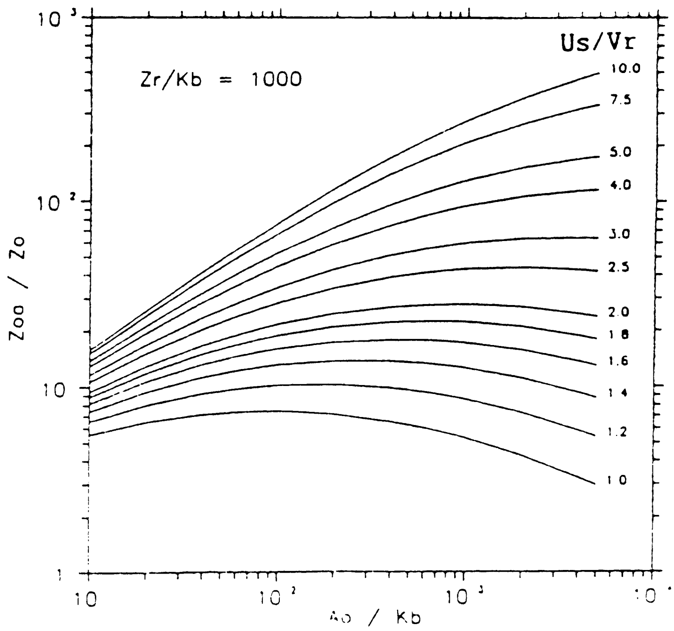


Fig A.3 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 1000$)

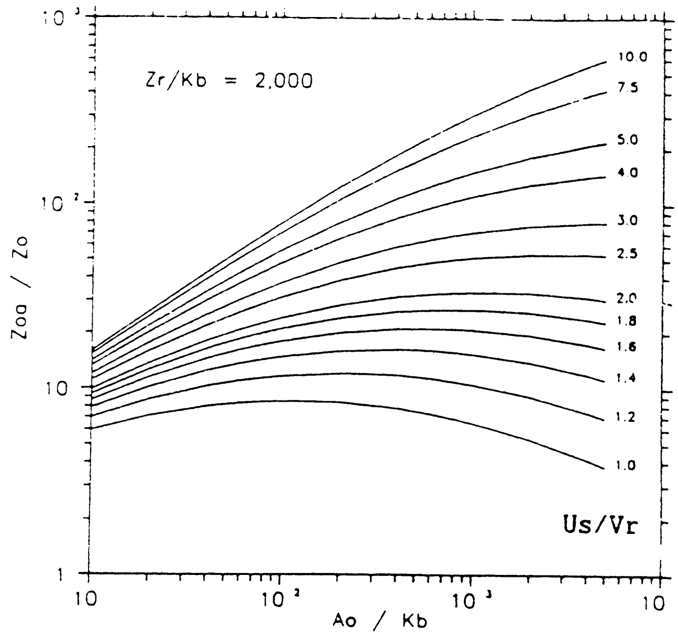


Fig A.4 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 2000$)

Fig A.5 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 5000$)

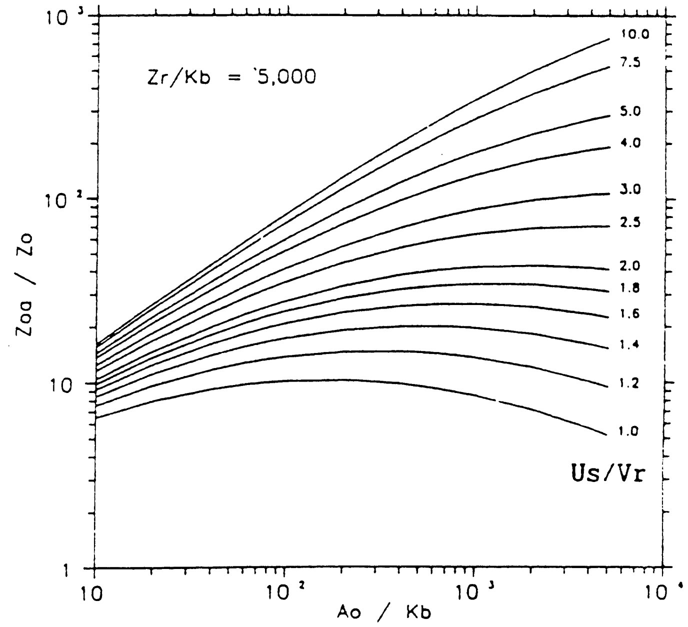


Fig A.6 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 10\,000$)

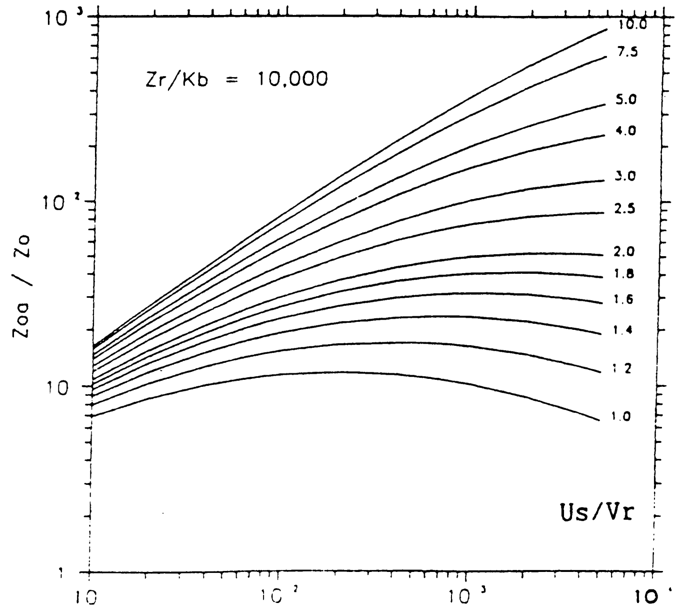
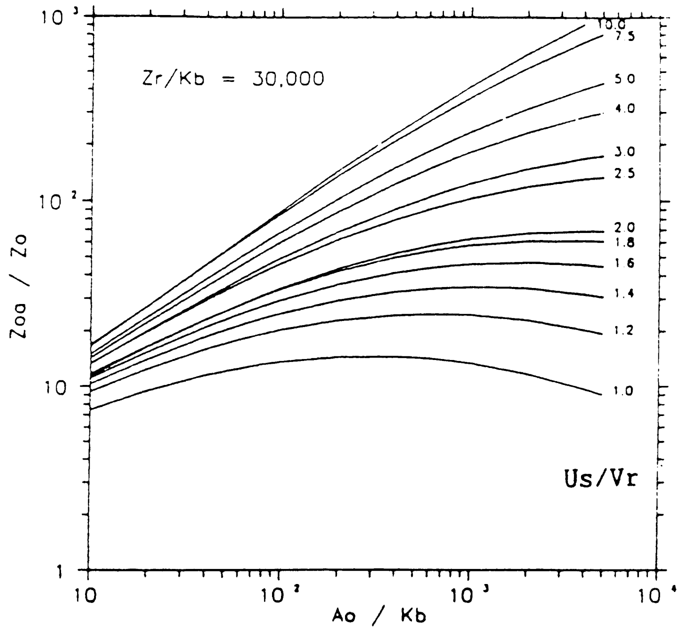


Fig A. 7 Z_{oa}/Z_o versus A_o/K_b (for $Z_r/K_b = 30\,000$)



A.6 REFERENCES

/A1/ Slaattelid,O.H., Myrhaug,D. and Lambrakos,K.F. North Sea boundary layer study for pipelines, OTC 5505, 1987.

APPENDIX B

CALCULATION EXAMPLES

B.1 INTRODUCTION

This Appendix presents some calculation examples of the simplified and generalized methods. The examples are for the following design case:

Pipeline design parameters:

- Steel pipe outer diameter, $D_s = 0.4064$ m
- Wall thickness, $t_s = 0.0127$ m
- Internal diameter, $D_i = 0.3810$ m
- Corrosion coating thickness, $t_{cc} = 0.005$ m
- Density of corrosion coating, $\rho_{cc} = 1300$ kg/m^3
- Density of concrete coating, $\rho_c = 2400$ kg/m^3
- Density of internal content, $\rho_i = 10$ kg/m^3 (gas)
- Density of seawater, $\rho_w = 1025$ kg/m^3
- Density of steel, $\rho_{st} = 7850$ kg/m^3

Soil type: Medium sand of density, $\rho_s = 1860$ kg/m^3

Environmental data:

- significant wave height, $H_s = 14.5$ m
- spectral peak period, $T_p = 15$ s
- water depth, $d = 110$ m
- current 3 m above bottom, $U_r = 0.6$ m/s

B.2 SIMPLIFIED METHOD

1. Find water particle velocities:

For wave, using Fig. 2.1 - 2.3.

$$T_n = \sqrt{(d/g)} = \sqrt{(110/9.81)} = 3.348$$

$$T_n/T_p = 3.348/15 = 0.223$$

From graph, Fig. 2.1 (Pierson Moskovitz, PM): $(U_s{}^*T_n)/H_s = 0.14$

$$U_s{}^* = (H_s/T_n)\cdot 0.14 = (14.5/3.348)\cdot 0.14 = \underline{0.606\ \text{m/s}}$$

Zero-up-crossing period, T_u - using Fig. 2.2

$$T_u/T_p = 1.07 \rightarrow T_u = 1.06\cdot T_p = 16.05\ \text{sec.}$$

Directional and spreading factor assumed to be

$R = 1.0$ - no reduction.

$\underline{U_s = U_s{}^*\cdot R = 0.606\ \text{m/s}}$

$\underline{T_u = 16.05\ \text{sec.}}$

Current velocity:

The current velocity 3 m above seabed ($Z_r = 3$).

$U_r = 0.6$ m/s

To calculate average velocity across the pipe assuming an approximate pipe diameter of 0.5 m (i.e. including corrosion coating plus 40 mm of concrete coating).

Medium sand assumed, from Table A1,

$d_{50} = 0.5$ mm
$Z_0 = 4.17 \cdot 10^{-5}$ m

which gives:

$D / Z_0 = 11990$

$Z_r / Z_0 = 3.0 / 4.17 \cdot 10^{-5} = 71942$

Substituting in equation A.3:

$$\frac{U_D}{U_r} = \frac{1}{\ln(71942+1)} \cdot \left\{ \left[1 + \frac{1}{11990}\right] \ln(11990+1) - 1 \right\}$$

$U_D / U_r = 0.7504$

$U_D = 0.7504 \cdot U_r = 0.6 \cdot 0.7504 = 0.45$ m/s

2. Using simplified static stability method:

Medium sand has been assumed, $\mu = 0.7$.

$C_L = 0.9, C_D = 0.7, C_M = 3.29$

An approximate diameter, $D \simeq 0.5$ m

$$A_s = 2\pi \cdot \frac{U_s}{T_u} = 2\pi \cdot \frac{0.606}{16.05} = 0.2372 \text{ m/s}^2$$

$$M = \frac{U_D}{U_s} = \frac{0.45}{0.606} = 0.75$$

$$K = \frac{U_s \cdot T_p}{D} = \frac{0.606 \cdot 16.05}{0.5} = 19.45$$

From Fig. 5.12, $F_w = 1.25$

Computing hydrodynamic forces and iterating to find the phase angle (θ) giving maximum submerged weight requirement (W_S).

For $\theta = 21$ degrees, max W_S is found:

$$\left.\begin{aligned} F_L &= 237.9 \text{ N/m} \\ F_D &= 185.1 \text{ N/m} \\ F_I &= 56.4 \text{ N/m} \end{aligned}\right\} W_S = \left[\frac{(185.1+56.4)+0.7\cdot 237.9}{0.7}\right]\cdot 1.25 \text{ [N/m]}$$

$$W_S = 728.75 \text{ N/m}$$

<u>A minimum submerged weight of 728.75 N/m is required.</u>

(Calculate concrete density required to achieve the above submerged weight with the estimated concrete thickness. Revise concrete thickness and density as necessary and repeat until a satisfactory combination of density and thickness is achieved).

B.3 GENERALIZED METHOD

From simplified static analysis, we have determined the following start values:

$W_S = 728.75$ N/m
D = 0.5 m (initial approximate outer pipe diameter)

Using the flowchart, section 5.2.3.4, assume thicknesses in first trial to be as for Simplified Method above.

Check diameter against formula:

$$D = \left\{ \frac{1}{2400-1025} \left[\frac{728.75}{0.25\cdot\pi\cdot 9.81} + 0.3810^2(7850-10) + 0.4064^2(1300-7850) + 0.4184^2(2400-1300) \right] \right\}^{\frac{1}{2}} \text{ [m]}$$

D = 0.5 m → required outer diameter.

Calculate parameters: (environmental data from simplified static stability method).

$$K = \frac{U_s \cdot T_u}{D} = \frac{0.606 \cdot 16.05}{0.5} = 19.45$$

$$M = \frac{U_D}{U_s} = \frac{0.45}{0.606} = 0.75$$

$$T = \frac{T_l}{T_u} = \frac{3\cdot 60\cdot 60}{16.05} = 672.90 \text{ (3 hours storm duration)}$$

$$\text{Target displacement} = 10\ \text{m};\ \delta = \frac{\text{displacement}}{D} = \frac{10}{0.5} = 20$$

Using Fig. 5.1 to 5.6 to determine L by interpolating with respect to values for δ and T as necessary:

$$\left.\begin{array}{l}\delta = 20, T = 500 \text{ give } \sqrt{L} = 2.65 \\ \delta = 20, T = 1000 \text{ give } \sqrt{L} = 2.85\end{array}\right\} \text{interpolating, } \sqrt{L} = 2.72$$

$$\rightarrow L = 7.40$$

Computing new $W_s = L \cdot 0.5 \cdot \rho_w \cdot D \cdot U_s^2$

$= 7.40 \cdot 0.5 \cdot 1025 \cdot 0.500 \cdot 0.606^2$ N/m

$W_s = 696.4$ N/m

Compute new D:

$$D = \left\{ \frac{1}{2400-1025} \left[\frac{696.4}{0.25 \cdot \pi \cdot 9.81} + 0.3810^2(7850-10) + 0.4064^2(1300-7850) + 0.4184^2(2400-1300) \right] \right\}^{\frac{1}{2}} [\text{m}]$$

D = 0.497 m (i.e. 0.6% difference from trial figure of 0.500 m, therefore acceptable).

Check strain level:

From Fig. 5.1 - 5.4, by interpolation $\varepsilon' = 2.6\%$

Engineering strain, section 5.2.3.3:

$$\varepsilon = \left(\frac{8 \cdot 666.3 \cdot 0.500}{\pi \cdot 2.1 \cdot 10^{11} \cdot 0.0127 \cdot 0.4064} \right)^{\frac{1}{2}} \cdot 2.6 = 0.0023\ \%: \text{OK (i.e. } < 0.2\ \%)$$