RECOMMENDED PRACTICE
DNV-RP-F110

GLOBAL BUCKLING OF SUBMARINE PIPELINES

STRUCTURAL DESIGN DUE TO HIGH TEMPERATURE/HIGH PRESSURE

OCTOBER 2007

DET NORSKE VERITAS

FOREWORD

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Amendments and Corrections

This document is valid until superseded by a new revision. Minor amendments and corrections will be published in a separate document normally updated twice per year (April and October).

For a complete listing of the changes, see the “Amendments and Corrections” document located at:
http://webshop.dnv.com/global/, under category “Offshore Codes”.

The electronic web-versions of the DNV Offshore Codes will be regularly updated to include these amendments and corrections.

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Project sub-projects and sponsors					
Phase 1		*Phase 2*		*Finalisation*	
Task	*Main Contractor*	*Task*	*Main Contractor*	*Task*	*Main Contractor*
1 Pipe Capacity	Snamprogetti	1 Analysis of documents		Axial Capacity evaluation	Snamprogetti
2 Pipeline Response	Snamprogetti	2 Workshop		High strength low cycle fatigue test of Cr13	Sintef
3 Mitigation Measures	Snamprogetti	3 Capacity	Snamprogetti	Guideline finalisation	DNV
4 Design Guideline	DNV	4 Response of exposed lines on un-even seabed	Snamprogetti		
		5 Response of buried lines	DNV/Statoil		
		6 Guideline preparation	DNV		
Statoil, Norsk Agip		Statoil, Norsk Agip, Shell		Statoil, ENI Norge, Shell, BP, Hydro	

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— Acergy
— Reinertsen Engineering
— Saipem
— Xodus Subsea
— Europipe.

CONTENTS

1. GENERAL

1.1 Introduction

Global buckling of a pipeline implies buckling of the pipe as a bar in compression. The global buckling may appear either downwards (in a free span), horizontally (lateral buckling on the seabed) or vertically (as upheaval buckling of buried pipelines or on a crest of exposed pipelines normally followed by a lateral turn-down).

Local buckling on the other hand is a gross deformation of the pipe cross section.

Global buckling is a response to compressive effective axial force and global buckling reduces the axial carrying capacity. Pipelines exposed to potential global buckling are then either those with high effective axial compressive forces, or pipelines with low buckling capacity, typically light pipelines with low lateral pipe-soil resistance.

High Pressure and High Temperature (HP/HT) pipelines are, from a structural point of view, characterised by expansion due to thermal heating and internal pressure and are as such typical candidates for global buckling. Definition of high pressure or high temperature is from global buckling point of view linked with the global buckling capacity and as such irrelevant.

The integrity of pipeline with a potential for global buckling can be assured by two design concepts;

- restraining the pipeline, maintaining the large compressive forces
- releasing the expansion forces, potentially causing it to buckle globally imposing curvatures on the pipeline.

The final selection of design concept depends on a series of factors.

This Recommended Practice (RP) will in depth describe both these design concepts.

The acceptance criteria are based upon state-of-the-art design principles, aimed at providing consistent, reliable and cost effective design solutions. This implies that recognised state of the art methods must be used in the design phase including the following aspects:

- structural response modelling
- route modelling (accurate seabed vertical profiles and transverse sections)
- soil pipe interaction modelling (including site soil samples)
- engineering evaluations of associated aspects such as girth weld properties, strain concentrations and corrosion.

Traditional design approach, based on analytical formulations (Hobbs, see ref. /1/; Taylor and Gan, ref. /2/) and closed form solutions may, hence, not be sufficient or optimal for application of this Recommended Practice.

1.2 Objective

The objective of this Recommended Practice is to provide procedures and criteria to fulfil the functional requirements to global buckling in DNV-OS-F101. It allows exposed pipelines to buckle in a controlled manner but ensures that buried pipelines stay in place.

1.3 Scope and Application

This Recommended Practice applies to structural design of rigid pipelines with a potential to buckle globally. This normally implies so called HP/HT pipelines but also moderately tempered pipelines may be of relevance depending on the global buckling resistance (e.g. very light pipelines). Hence, it does not include considerations with respect to temperature and pressure effects on materials, insulation, thermal conditions and flow assurance but gives procedures and criteria to global buckling only. Consequently, there are no limitations to pressure or temperature other than those caused by structural limitations from the material properties etc.

Global buckling is a load response and not a failure mode as such. Global buckling may, however, imply an ultimate failure mode such as:

- local buckling
- fracture
- fatigue.

This Recommended Practice covers global buckling ultimately limited by local buckling. The procedures provide also best estimates and upper bound strain values that may be used in a fracture assessment. For other failure modes reference is made to DNV-OS-F101.

Three global buckling scenarios of HT/HP pipelines are covered in this Recommended Practice:

1) Exposed pipelines on even seabed. Global buckling occurs in the horizontal plane. Post buckling configuration may be allowed.
2) Exposed pipelines on un-even seabed. Global deformation occurs first in the vertical plane (feed-in and uplift) and subsequently in the horizontal plane or when this is combined with scenario I, e.g. for curves on un-even seabed. Post buckling configuration may be allowed.
3) Buried/covered pipelines. Global buckling in the vertical plane, so called upheaval buckling. Only criteria for avoiding global buckling are given.

The trawl interference evaluation in this Recommended Practice is limited to lateral buckling only. Hence, trawling in free span is not covered by this Recommended Practice but by DNV-RP-F111.

With reference to trawling applicability see also Sec.3.5.3. For other design issues, reference is made to other codes, see Sec.1.5.

The design procedures and criteria in this Recommended Practice are based on vast number of analyses. In spite of this very special combinations may give un-anticipated results and the designer shall evaluate all results critically.

The provided design procedures are considered being applicable in general. The basis for the development has been pipelines in the range from 10” to 42”.

1.4 Structure of Recommended Practice

The first part of the Recommended Practice is common for all three covered scenarios above and cover:

- Description of the different scenarios, decision flowchart and background are given in Sec.2
- Input parameters like pipe geometry, material, operational parameters, survey and trawling are given in Sec.3 including combinations of different loads
- Soil in general is discussed in Sec.4
- Analytical axial load and general requirements to the response model (FE-model) are given in Sec.5
- Detailed procedures and criteria for the three scenarios:

  I) Global buckling on even seabed, Sec.6
  II) Global buckling on un-even seabed, Sec.7
  III) Upheaval buckling of buried pipes, Sec.8.

  Each of these starts with a general design procedure, outlining the general iterations in a design process. This is followed by a more specific description of the required design steps. In case specific guidance for the specific design scenario is required on the structural modelling, this is also given here while the general discussion is given in Sec.5:

- Procedure for calculation of the condition load effect factor

for exposed pipelines (Scenario I and II) is given in Sec.9
- Complementary limit states to those in DNV-OS-F101 are given in Sec.10
- Recommendations on documentation for operations are given in Sec.11
- Bibliographic references are given in Sec.12 while code references are given in Sec.1.6
- Appendix A includes examples of mitigation measures that are applicable to exposed pipelines
- Appendix B gives more detailed guidance on pipe-soil interaction for uplift resistance (for upheaval buckling)
- Appendix C gives examples of design procedures for exposed pipelines
- Appendix D gives some information on the uncertainties in the design.

1.5 Relationships to other Codes

DNV-OS-F101 is a risk based limit state code where the pipe integrity is ensured by design criteria for each relevant failure mode. The most relevant failure modes are identified and have specific design criteria in this standard. For the global buckling limit state only functional requirements are given:

The following global buckling initiators shall be considered: (i) trawl board impact, pullover and hooking, and (ii) out of straightness.

Further:

Displacement-controlled global buckling may be allowed. This implies that global buckling may be allowed provided that:

- *pipeline integrity is maintained in post-buckling configurations (e.g. local buckling, fracture, fatigue, etc.)*
- *displacement of the pipeline is acceptable.*

This Recommended Practice complies with DNV-OS-F101 and complements the functional requirement on global buckling with specific procedures and design criteria.

The procedures and design criteria in this Recommended Practice have been determined by means of structural reliability methods and target failure probability in compliance with DNV-OS-F101 combined with sound engineering judgement.

Global buckling is a response to effective axial load. This may subsequently cause different failures modes. In this Recommended Practice only local buckling failure will be considered. For other structural failure modes the following references apply:

- general; DNV-OS-F101
- vortex induced vibration; DNV-RP-F105
- fracture; DNV-RP-F108 and BS 7910
- on bottom Stability; DNV-RP-F109
- trawling interference in free spans; DNV-RP-F111 (see also note in Sec.3.5.3).

Referenced relevant codes are listed in 1.6 and bibliographies in Sec.12.

In case of conflict between requirements of this code and a referenced DNV Offshore Code, the requirements of the code with the latest revision date shall prevail, any conflict is intended to be removed in next revision of that document.

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

1.6 Referenced codes

1.6.1 DNV Offshore Services Specifications

The latest revision of the following documents applies:

DNV-OSS-301	Certification and Verification of Pipelines,
DNV-OSS-302	Offshore Riser Systems
DNV-OSS-401	Technology Qualification Management.

1.6.2 DNV Offshore Standards

Latest revision of the following documents applies:

DNV-OS-F101	Submarine Pipeline Systems
DNV-OS-F201	Dynamic Risers.

1.6.3 DNV Recommended Practices

The latest revision of the following documents applies:

DNV-RP-A203	Qualification Procedures for New Technology
DNV-RP-F105	Free Spanning Pipelines
DNV-RP-F108	Fracture Control for Installation Methods Introducing Cyclic Plastic Strain
DNV-RP-F109	On bottom stability design of submarine pipelines
DNV-RP-F111	Interference between Trawl gear and Pipelines.

1.6.4 Other codes

BS7910	Guide to methods for assessing the acceptability of flaws in metallic structures, British Standard Institute.

1.7 Definitions

Refer also to DNV-OS-F101 for definitions.

2D analyses	pipeline analyses with all degrees of freedom (i.e. 3D) but with the initial pipeline geometry modelled in one plane only
2½D analyses	pipeline analyses with all degrees of freedom (i.e. 3D) both modelled and analysed in three dimensions, i.e. including curves in the vertical horizontal plane, but with the seabed in the lateral direction modelled flat
3D analyses	pipeline analyses both modelled and analysed in three dimensions giving possible benefit for curves in the horizontal plane. The seabed is modelled considering real seabed, including sideways slopes, etc.
Code	means any Standard, Recommended Practice, Guideline, Classification note or similar
Cohesion less soil	type of soil material e.g. sand and rock
Cohesive soil	type of soil material e.g. clay
Cover	added material, e.g. gravel or seabed material, either in trench or on flat bottom (un-trenched).
Effective axial force	the combined axial action of the stress in the pipe wall and the internal and external pressure, see Sec.5.2.2. The effect of internal and external pressures is then included in the concept of the effective axial force
Feed-in	the expansion into an area when the resisting force has been reduced, e.g. the release of the stored energy in a pipeline into a buckle
Functional load design case	see Sec.3.5
Global buckling	on-set of transverse instability of a significant length of pipe. The transverse instability could be in the vertical plane (upheaval buckling) or horizontal plane (lateral buckling)

Interference load design case	see Sec.3.5
Lateral buckling	global buckling in the horizontal plane (the post buckling condition is sometimes also referred to as snaking)
Local maximum design temperature	is the temperature at a specific point in the pipeline corresponding to the maximum design temperature profile.is the temperature at a specific point in the pipeline corresponding to the maximum design temperature profile
Pipeline walking	is the axial accumulation of displacement
Post Global Buckling	development of the pipe configuration after the initial buckling
Propped shape	the configuration of a pipeline given by the (relevant) self weight and pipe stiffness when lifted from a horizontal plane a certain height δ
Ratcheting	in general, accumulation of cross section deformation: — A pipeline not allowed to develop additional bending, exposed to the bi-axial effect of internal pressure and axial compressive stresses may yield over the whole cross section, giving rise to diameter increase. Cyclic loading due to start-up and shutdown may cause the diameter increase as a consequence of reversal plasticity to accumulate, which is referred to as ratcheting. — A pipeline, allowed to develop additional bending may experience cyclic operating loads or frequent trawling pull-over loads so accumulating bending deformation (bending ratcheting). Pipeline walking is not referred to as ratcheting.
Snake lay	pipeline installed with global lateral imperfections.
Upheaval buckling	the consecutive deformation in the vertical plane. This can occur both for exposed (prior to developing laterally) and buried pipelines.

1.8 Abbreviations

ALS	Accidental Limit State
BE	Best Estimate
CoV	Coefficient of Variance for an stochastic variable, CoV(X) = Standard deviation(X) / Mean (X)
FE	Finite Element
HP/HT	High Pressure and/or High Temperature
KP	Kilometer Point
LB	Lower Bound
LRFD	Load and Resistance Factor Design
NA	Not Applicable
PIC	Pipe Integrity Check
ROV	Remotely Operated Vehicle
SLS	Serviceability Limit State
SMTS	Specified Minimum Tensile Strength (engineering stress) at room temperature
SMYS	Specified Minimum Yield Stress (engineering stress) at room temperature
SNCF	StraiN Concentration Factor
UB	Upper Bound
UHB	Up Heaval Buckling
ULS	Ultimate Limit State
TS	Tensile Strength (engineering stress) at room temperature.
YS	Yield Stress (engineering stress) at room temperature

1.9 Symbols

1.9.1 Latin characters

A	Pipe cross sectional area
D	Nominal outside diameter of steel pipe
D_{tot}	Nominal outside total outside diameter of pipe including external coating
E	Young's Modulus
f	Pipe-soil resistance force (lateral or vertical)
F_P	Trawl pull-over load from DNV-RP-F111
f_T	Annual trawling frequency per relevant pipeline section.
F_T	Characteristic pullover load, see. Table 3-3
f_u	Characteristic tensile strength, see Sec.3.4
$f_{u,temp}$	Tensile strength de-rating factor, see Sec.3.4
f_y	Characteristic yield strength, see Sec.3.4
$f_{y,temp}$	Yield strength de-rating factor see Sec.3.4
H	Cover height from top of pipe to soil surface
	H_1, H_2 Heights of different soil layers; ($H = H_1 + H_2$), see. Figure 4-1.
H	Residual laying tension effective force
H_{min}	The required cover on buried pipes to allow for imperfections not detected due to the accuracy of the survey equipment
H_{spec}	The required cover on buried pipes based on the actual measured imperfection
H_{survey}	The minimum measured cover height during verification
K	Stiffness (downwards) in clay
L	Buckle length
L_0	Imperfection length for an prop shape, from apex to touch down
M	Moment effect
N	True axial force (pipe wall force)
p	Pressure
R	Radius of imperfection
S	Effective (axial) force (tension is positive)
S_0	Fully restrained effective axial load, Eq. (7)
t	Time [year] or nominal thickness
t_1	Pipe minimum wall thickness adjusted for relevant corrosion, see Table 3-1
t_2	Pipe nominal wall thickness adjusted for relevant corrosion, see Table 3-1
t_{corr}	Corrosion allowance
t_{fab}	Pipe wall thickness fabrication mill tolerance
T_d	Design temperature
$T(k)$	Temperature at failure for downward soil stiffness k
T_{max}	Maximum design temperature
$T_{l,max}$	Local maximum design temperature
T_{Rd}	Design resistance equivalent failure temperature
T_{Sd}	Design load equivalent temperature
w	Submerged pipe weight
z_o	Vertical displacement of pipe in soil, see Figure 4-1

1.9.2 Greek characters

α	Thermal expansion coefficient
α_U	Material strength factor
γ_c	Condition load effect factor
γ_E	Load effect factor for environmental loads
γ_F	Load effect factor for functional loads
γ_{lift}	Safety factor (a value of 1.3 is recommended)
γ_m	Material resistance factor
γ_{SC}	Safety Class resistance factor
γ_{UF}	Maximum design temperature factor, see Sec.8.3.3
γ_{UR}	Uplift resistance safety factor, see Sec.8.3.2
δ	Imperfection height of a prop shape imperfection
δ_f	Propped shape imperfection height for minimum cover
Δp_d	Differential design pressure
Δp_i	Internal pressure difference relative to laying
ΔT	Differential temperature
ν	Poisson ratio
σ	Standard deviation (statistical)
σ_{consol}	Consolidation stress (in clay) due to a weight of the soil above
$\sigma_{configuration}$	Standard deviation on the configuration measurement accuracy, see Sec.8.3.2
σ_{cover}	Standard deviation on the pipeline cover depth measurement accuracy, see Sec.8.5.1
σ_u	Ultimate strength (average at room temperature), see Sec.3.4
σ_y	Yield stress (average at room temperature), see Sec.3.4.

1.9.3 Subscripts

a	Axial
c	Characteristic
e	External
E	Environmental
F	Functional
i	Internal
l	Lateral
ld	Local design (pressure)
li	Local incidental (pressure)
Rd	Design (factorised) resistance
s	Steel
Sd	Design (factorised) load
T	Trawl

1.10 Units

The use of metric units is strongly recommended. However, any set of consistent units may be applied with the following exceptions; the uplift resistance factor, γ_{UR}, must be expressed in meters.

2. Design Scenarios

2.1 Global Buckling

Temperature and pressure effects create expansion effective forces which may cause a pipeline to buckle globally. Pipelines installed on the seabed and left exposed have a potential to buckle globally and change configuration while a buried pipeline is designed to stay in place being restricted by the surrounding soil reaction forces.

The driving force for global buckling of the pipeline is the effective axial force, *S,* which represents the combined action of pipe wall force, *N,* and internal and external pressures, see Sec.5.2.2.The effective force for a restraint straight pipe, S_0, constitute an upper bound axial load and is discussed in Sec.5.3.1.

For a certain expansion force, the pipeline will buckle globally. For a partially displacement controlled condition, this implies that it will find a new equilibrium by moving perpendicular to the pipe axis at the same time as the pipe will move axially, *feed-in,* from both sides towards the buckle. The level of axial force to initiate this global buckling depends on:

- pipe cross section properties
- lateral resistance
- out-of-straightness in the pipeline
- lateral triggering force (e.g. trawling).

A straight column will buckle according to the classical Euler buckling formulation. As the out-of-straightness in the column increases, the level of axial force required to buckle it will be reduces. This effect, away from the buckle, is illustrated in Figure 2-1.

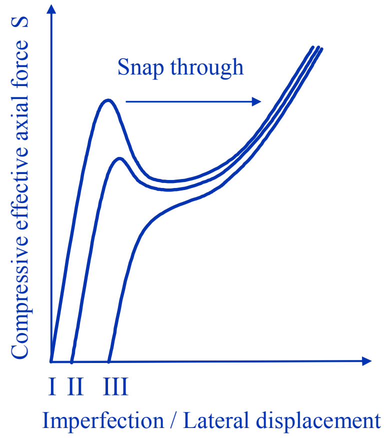


Figure 2-1
Load response of a globally buckling pipeline

The out-of-straightness may be caused by:

- small imperfections on the seabed like the pipeline resting on rocks
- global imperfections as uneven seabed
- curvature in the horizontal plane purposely made or random from installation.

To illustrate the global buckling of a section in a pipeline, the following idealised sequence of a pipeline with free end expansion can be used :

a) Prior to applying pressure and temperature, the effective force will be limited to the residual lay tension. The effective force variation will be tri-linear; from zero at the pipeline ends with a linear increase proportional to the axial resistance to the soil, until it reach the residual lay tension H. It will then be constant until it reaches the decay from the other side, see lower curve of Figure 2-2.

b) When the temperature or pressure increase the compressive effective force will increase to maximum S_0, This will vary along the pipeline as the temperature and pressure decrease along the line. At the pipe ends, the load will still be zero, see upper curve of Figure 2-2. A snap shot from a short section is now selected for closer examination in Figure 2-3.

The buckling development is described in Figure 2-3.

Note also that the post-buckling load, point B above, may not be reached directly but through a continuous development. This may imply that higher force close to the buckle is achieved prior to reaching B, that may trigger another buckle.

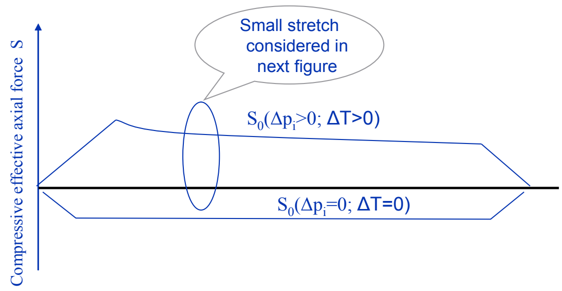


Figure 2-2
Effective force after lay-down and slightly after start-up, prior to buckling

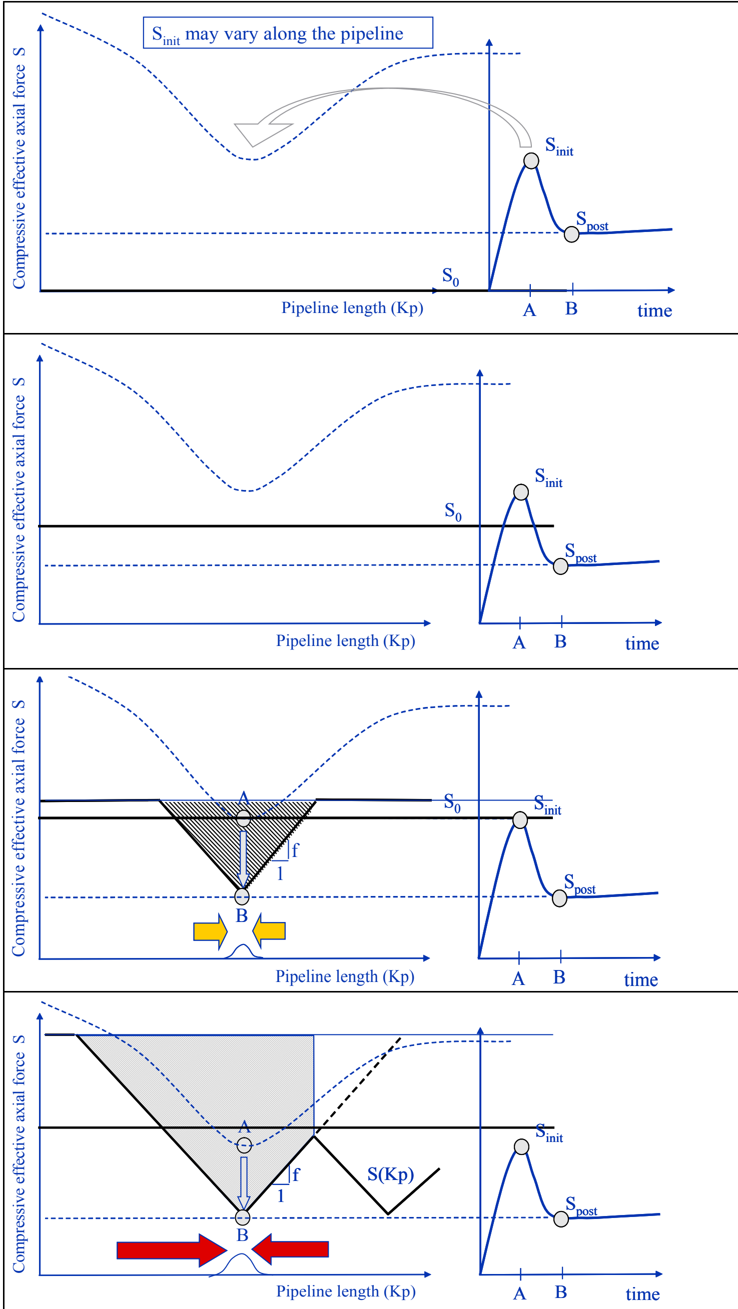


Figure 2-3
General behaviour during global buckling

a Buckling capacity along the pipeline

The pipeline has an imperfection. At this point the global buckling capacity, S_{init} has its lowest point.

b Prior to global buckling

When the temperature or pressure increase the compressive effective force will increase to maximum S_0.

c First buckle

When the effective axial force S_0 reach S_{init} (point A) the pipeline will buckle and the effective axial force at the apex will drop to S_{post} (point B). The maximum change in the effective force will be equal to the axial pipe-soil restraining force, giving the slope of the effective force diagram. The effective axial force will then be as the solid line. The feed-in to the buckle will be proportional to the shaded area between the solid line and the potential effective force line.

d Second buckle

If the pressure or temperature is further increased, a neighbouring imperfection may buckle and change the force diagram as shown in Figure 2-3d. From this point, the force will remain constant but the feed-in will increase, proportional to the shaded area. Hence, the slope of the feed-in describes which way the pipe is moving and what the feed-in will be.

Where:

f	axial friction [N/m]
S_0	restrained effective axial force – maximum potential force along the pipeline for a given pressure and temperature
S_{init}	effective axial force initiating global buckling in the pipeline
S_{post}	effective axial force close to apex a buckle after the pipeline has buckled
S(kp)	effective axial force along the pipeline (kilometre post).

2.2 Design Flow

2.2.1 General

In (structural) pipeline design, different design tasks interact to a varying degree but may be split into; *material selection/wall thickness design, installation design* and *design for operation* giving the outputs as shown in Figure 2-4.

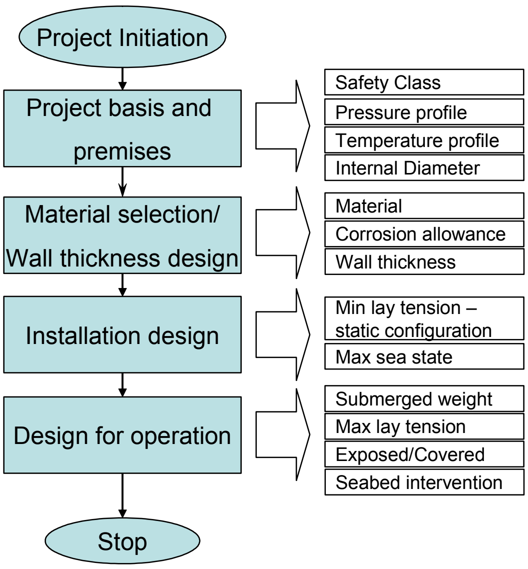


Figure 2-4
General design tasks

The design for operation includes several aspects that may influence if the pipeline must be buried or not, see Figure 2-5.

Global buckling design is often referred to as expansion design and is a part of the above *operation design*. Note that there may be a strong link between free-span design and expansion design since release of the compressive forces in a free span may significantly change the eigen frequencies, see ref. /3/.

Based on the cover considerations in Figure 2-5 for burying a pipeline, global buckling analyses needs to be assessed, either in an exposed scenario or a buried scenario. The following scenarios are covered in this Recommended Practice:

I. pipeline exposed on even seabed
II. pipeline exposed on un-even seabed
III. buried pipeline.

A design based on releasing the expansion forces has particular challenges during operation with respect to evaluation of inspection results. Therefore clear documentation for this purpose shall be provided, see Sec.11.

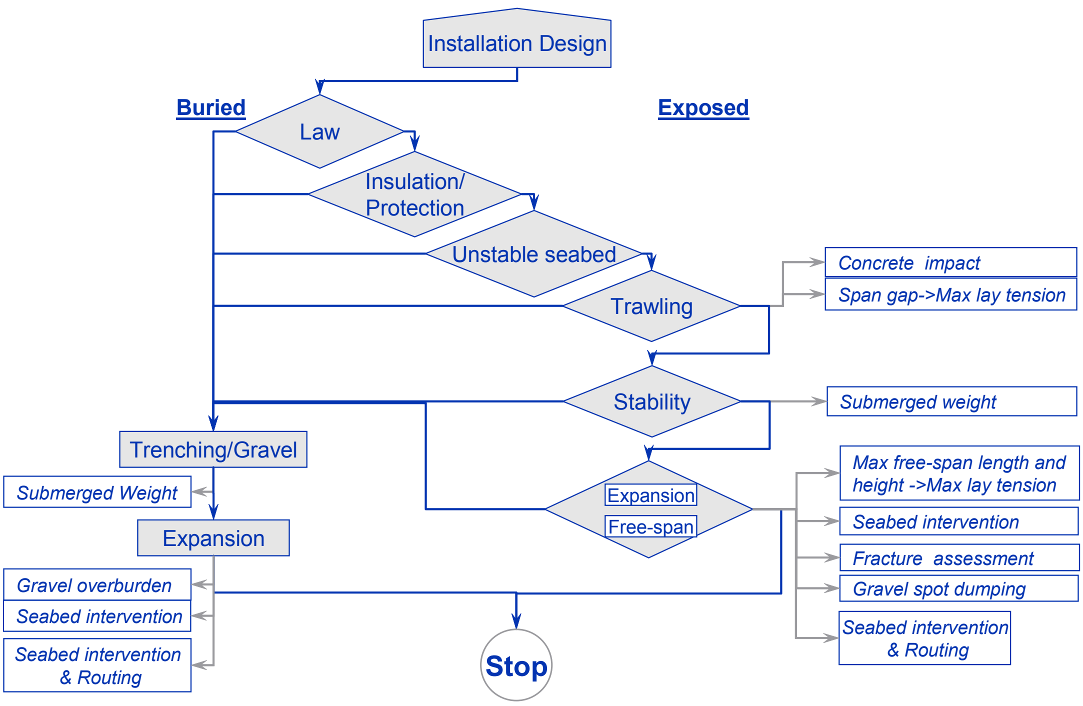


Figure 2-5
In place structural design tasks

2.2.2 Scenario I. Pipeline exposed on even seabed

This scenario applies when the governing deformation will take place on the seabed, i.e. in the horizontal plane. The deformation will take place either due to natural out of straightness or by purpose made out of straightness, see Figure 2-6.

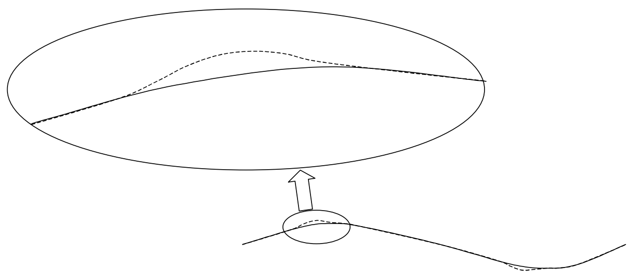


Figure 2-6
Exposed pipeline on even seabed with purpose made lateral imperfection

Design of the exposed pipeline on even seabed include the following design steps:

— **Global buckling Assessment**: Determination of the susceptibility of the pipeline to experience lateral buckling, upheaval or upheaval combined with lateral buckling due to temperature and pressure.
— **Pipe Integrity check**: The resulting bending moment/longitudinal strain in the post-buckled configuration must be shown to be acceptable. Subsequent over-trawling must be considered, if relevant.
— **Mitigation Measure check**: If buckling due to imperfections or external loads results in too high local bending moment/longitudinal strains, i.e. that the pipe integrity check is not fulfilled, mitigating measures shall be considered, see Appendix A. The mitigation may either be to control the buckling so that the moments/longitudinal strains are within allowable limits or to prevent development of buckling.

2.2.3 Scenario II. Pipeline exposed on Un-even seabed

This scenario applies when the deformations initially occur in the vertical plane, and subsequently in the horizontal plane, Figure 2-7. This scenario also applies for combination of un-even seabed and Scenario I, e.g. for curves on un-even seabed. Typically it includes the following three phases:

1) expansion into free-span
2) lift-off at the crests:
   — limited lift-off
   — maximum lift-off
3) lateral instability, causing the pipeline to expand sideways.

Alternatively, purpose built crossing of existing pipelines may constitute the imperfection in which the laid pipeline develops a considerable initial bending in the vertical plane, Figure 2-8.

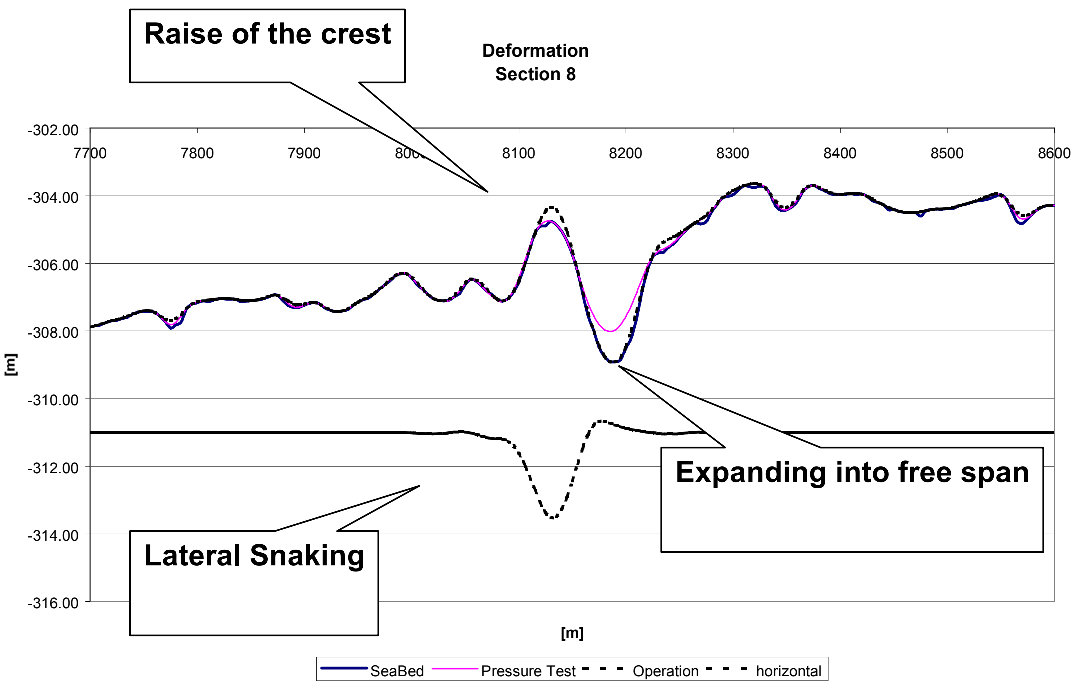


Figure 2-7
Exposed pipeline laid on un-even seabed

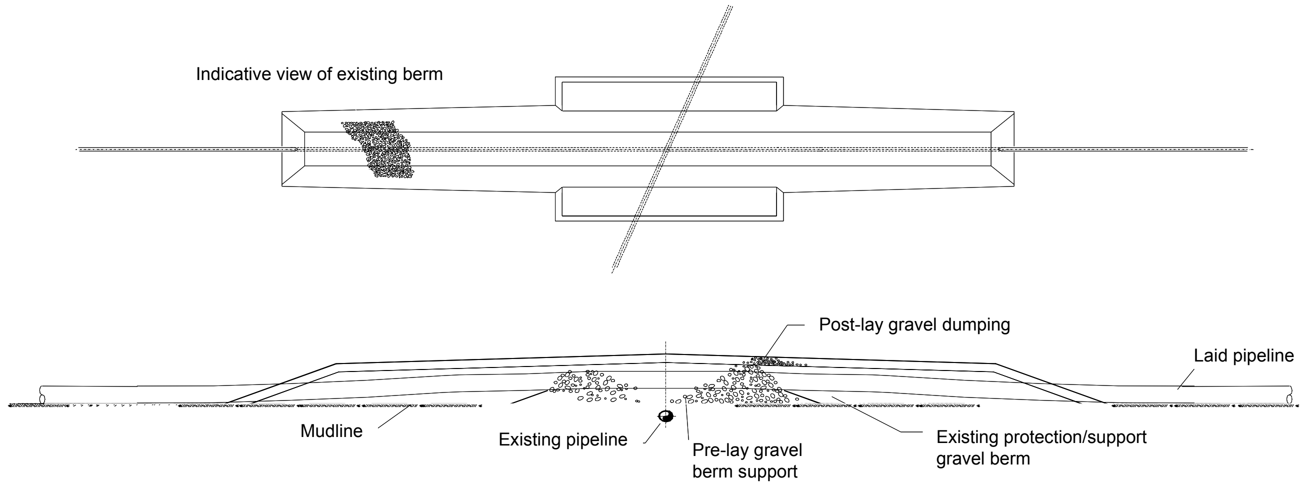


Figure 2-8
Purpose built crossing of existing pipeline in which the laid pipeline develops considerable initial bending in the vertical plane.

Similar design steps as for scenario I apply. The major difference is the triggering mechanism of global buckling which is likely to be vertical imperfections caused by the un-even seabed.

2.2.4 Scenario III. Buried pipeline

If the pipeline must be covered, the cover/lateral restraint shall be designed to avoid global buckling of the pipeline. This may be done by trenching and covering it by naturally back-fill or artificially back-fill, see Figure 2-9 a and b. Soil nature, pipe properties and trenching technology influence the evenness of the trench bottom, and a reference bottom roughness for cover height requirements must be anticipated.

An un-trenched pipeline may be restrained in its configuration e.g. by covering with continuous gravel dumping, Figure 2-9c. This may be a preferred choice in some cases. Soil nature, pipe properties and dumping technology influence the shape and height of pipe cover, which is implemented generally using crushed rock.

Design of buried pipelines is often split into two stages:

— pre-installed phase
— as-installed phase.

The purpose of the first is to get a cost and gravel estimate while the purpose of the second is to ensure the integrity of the pipeline.

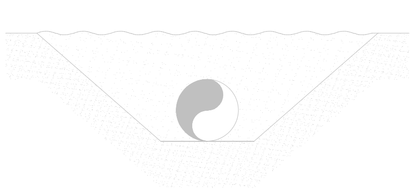
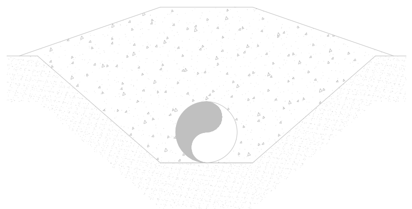
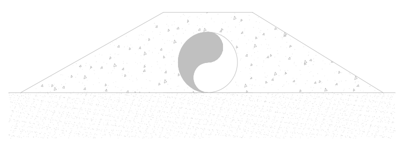
 a	 b	 c
Pipeline trenched and naturally covered	Pipeline trenched and covered with gravel (or a mixture of natural and gravel)	Pipeline covered with gravel dumping

Figure 2-9
Possible scenarios for covered/restrained pipelines

3. Basis for Design

3.1 General

This section defines the characteristic design parameters and load combinations to be used with the design procedures and criteria in this Recommended Practice.

3.2 Uncertainties

All properties like loads, geometries and material strengths include uncertainties. These uncertainties can be split into four groups:

— natural variability (physical)
— statistical uncertainty
— measurement uncertainty
— model uncertainty.

Natural variability is of random nature and characterised by that more studies will not necessarily reduce the uncertainty. One example is wall thickness; it will vary independent on how many measurements are taken. Another example is waves. Note that limited measurements (statistical uncertainty) and accuracy of measurements (measurement uncertainty) sometimes are included in the natural variability.

Statistical uncertainty relates to the uncertainty in predicting the statistical variables. Increasing number of samples reduces the statistical uncertainty.

Measurement uncertainty relates to the accuracy in the measurement of each sample.

Model uncertainty is characterised by limited knowledge or idealisation of stochastical /physical model. More research will typically reduce the model uncertainty.

In order to correctly reflect the uncertainty in the property, it can be expressed as a probability density function, see Figure 3-1.

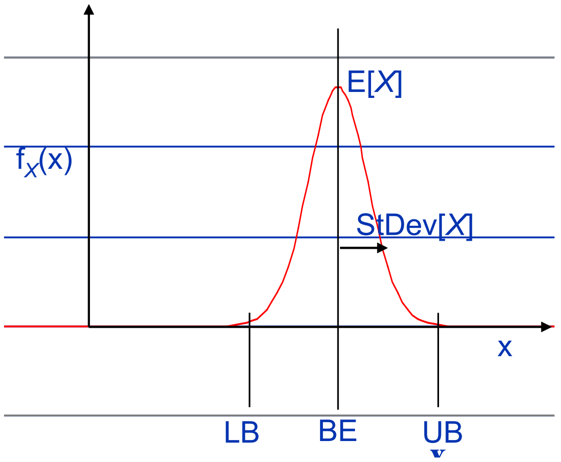


Figure 3-1
Example of a symmetric density distribution function (Normal)

The distribution function is normally characterised by a mean value, μ, and a standard deviation, σ. When using properties with significant uncertainty, first, it has to be checked if variation in this property will affect the results (i.e. if the result is sensitive to this property). If it is, a “conservative value” of this property is normally used. This may be a lower or higher value than the mean value. A value representing a distribution in a certain application is called a “characteristic value” and only means that it is defined in some way, often as mean ± a certain number of standard deviations. Two standard deviations are often used, corresponding to that the probability of that the property has a lower or higher value than this characteristic value is equal to 2.275% if the distribution is normal distributed.

In Figure 3-1 the lower bound (LB) and upper bound (UB) may be such characteristic values while the best estimate (BE) may be used when the result is insensitive to the property.

Sometimes, a non-symmetric distribution may be a better representation of the uncertainty. For such distributions there are several properties that may represent “Best Estimate”, see Figure 3-2 (where all of them have the same value in a symmetric distribution).

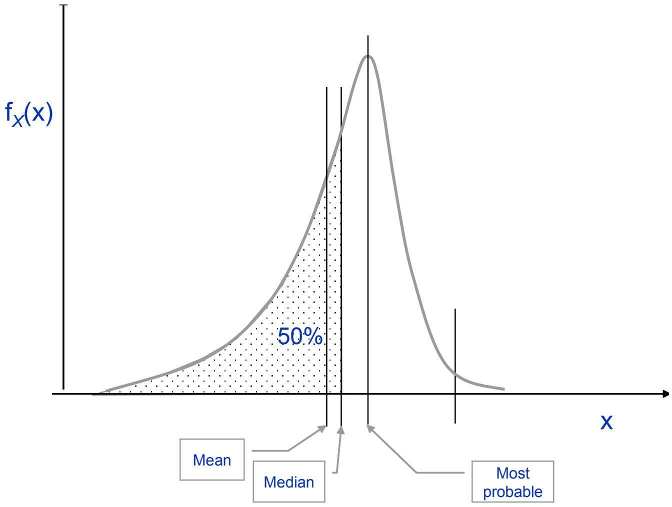


Figure 3-2
Common values in a density distribution function

For such a non-symmetric distribution, it may be more representative to define the upper and lower bound values as a “fractile”, see Figure 3-3. This may be a value corresponding to a normal distribution function with two standard deviation, i.e. in the order of 2%-5%.

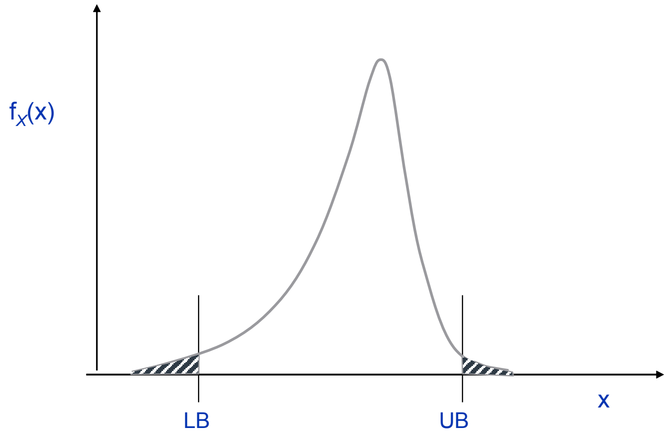


Figure 3-3
Typical upper and lower bound values for a non-symmetric distribution

In application of this Recommended Practice, it is not known if a low or high value representing the pipe-soil interaction is conservative and a procedure, including all; LB, BE and UB is used. These shall typically be defined as mean ±two standard deviation or with a fractile in the order of 2%-5%.

As the distribution of the pipe-soil capacity is normally not well known, the estimates have to rely on engineering judgement.

3.3 Pipe Geometry

The compressive forces in the pipeline are mainly due to temperature and pressure differences to the as-laid condition, see Eq.(7). For the thermal expansion this implies that the larger the steel cross sectional area is the larger the restrained axial forces will be. A thicker wall may therefore be detrimental, triggering global buckling. A thicker wall will, on the other hand, be beneficial in the post-buckling condition.

The load effect analyses shall be based on the most unfavourable combinations of loads. This implies that nominal wall thickness normally apply while the corroded section is used, if relevant, for the resistance. A robust design will often be based on that the buckles occurs early (i.e. first time when design values are reached) and full cross section properties are then normally used.

If it can be documented that the corrosion for a substantial pipe stretch is expected to be uniform, the load effect could be calculated for half the corrosion.

Table 3-1 Characteristic pipe geometry properties

Parameters	*Limit State*	*Resistance*		*Load effect calculation*[3]
		Symbol	*Value*	
Diameter	All	D	Nominal	D (Nominal)
Wall thickness	Pressure containment	t_1	$t_1 = t - t_{fab} - t_{corr}$	-
	Local buckling[1]	t_2	$t_2 = t - t_{corr}$	t (Nominal)
	Local buckling[2]	t_2	$t_2 = t$	t (Nominal)
		t_2	$t_2 = t - t_{corr}$	Min $t - 0.5 \cdot t_{corr}$

1) If it can be documented that the corrosion is expected to be uniform, local buckling[2] can replace this check.
2) If it can be documented that the corrosion is expected to be uniform, load effect could be calculated for maximum half the corrosion combined with full corrosion for the capacity check on the load effect calculation. In addition, the scenario of no corrosion shall be checked.
3) For local buckling, the load effect calculation is equivalent to the global buckling calculation.

3.4 Pipe Material

The pipe material discussed in this section applies to both the load effect calculation and the capacity check.

The material parameters shall be based on the nominal values except for the yield stress and ultimate strength. The stress-strain curve based on yield stress and ultimate strength shall be based on the specified minimum values, f_y and f_u, as per DNV-OS-F101, except for when the mean value is explicitly required by the procedure Eq. (3) and (4).

It is important to include the temperature effect on the material parameters, not limited to the yield stress and ultimate strength only but also to the temperature expansion coefficient, α, and to Young's modulus, E. Note that the thermal expansion coefficient will increase with temperature and neglecting this effect will give non-conservative results. The characteristic material strength factors are defined below and a summary of the material parameter definition is given in Table 3-2.

$$f_y = \left(SMYS - f_{y,temp}\right) \cdot \alpha_U \quad \text{DNV-OS-F101:2007 Eq. (5.5)} \tag{1}$$

$$f_u = \left(SMTS - f_{u,temp}\right) \cdot \alpha_U \quad \text{DNV-OS-F101:2007 Eq. (5.6)} \tag{2}$$

$$\sigma_y = YS - f_{y,temp} \tag{3}$$

$$\sigma_u = TS - f_{u,temp} \tag{4}$$

Table 3-2 Material parameters

Parameter	*Symbol*	*Value*	*Temperature effect*[4]
Young's modulus	E	Nominal/Mean	Above 50°C[3]
Temperature expansion coefficient	α	Nominal/Mean	Above 50°C[3]
Yield stress, specified minimum	SMYS	Minimum specified[1]	At room temperature
Yield stress, mean	YS	Mean	At room temperature
Ultimate strength, specified minimum	SMTS	Minimum, specified[2]	At room temperature
Ultimate strength, mean	TS	Mean	At room temperature
Poisson ratio	ν	Mean	Negligible
Reduction in yield stress due to elevated temperature[5]	$f_{y,temp}$	Mean	Above 50°C, Local design temperature
Reduction in ultimate strength due to elevated temperature	$f_{u,temp}$	Mean	Above 50°C, Local design temperature

1) When supplementary requirement U is specified, the minimum specified stress shall be at least 2 standard deviations below the mean value.
2) When supplementary requirement U is specified, the minimum specified strength shall be at least 3 standard deviations below the mean value.
3) If a constant value is used in the load effect calculations, this shall be an equivalent value representing the total effect at the local design temperature.
4) For Duplex and Super Duplex, the de-rating shall be considered from 20°C.
5) See DNV-OS-F101:2007 Fig. 5-2.

Unless no data exist on the material de-rating effects, the conservative estimate in Figure 3-4 can be used. The same de-rating can also conservatively be used for the ultimate strength de-rating.

Unless no data exist on the material expansion coefficient temperature dependency indications are given in Figure 3-5 from ref. /5/.

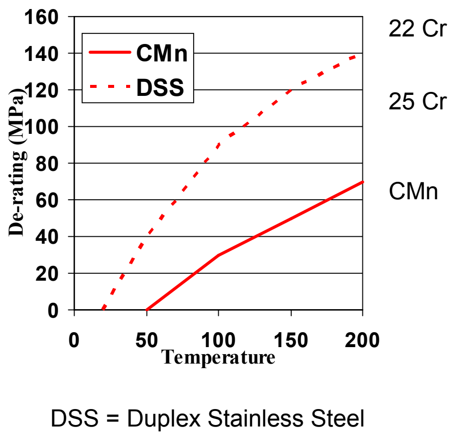


Figure 3-4
Proposed yield stress de-rating if no other data exist DNV-OS-F101:2007 Figure 5-2

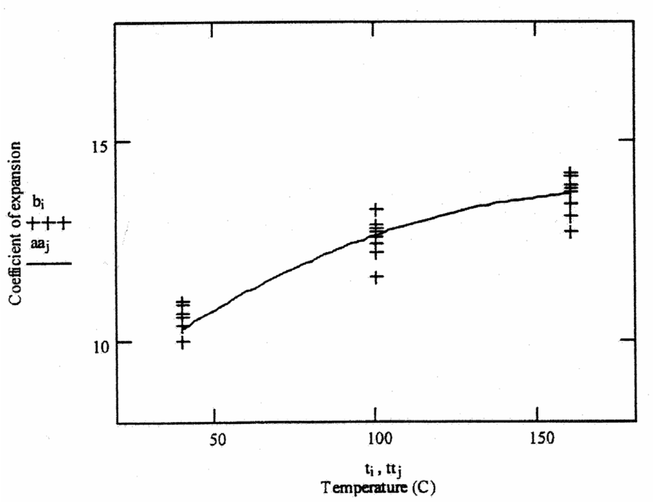


Figure 3-5
Measured expansion coefficient dependency with temperature, see ref. /5/

Note that the thermal expansion coefficient in Figure 3-5 is the incremental expansion and in order to get an equivalent, constant, thermal expansion coefficient, it has to be integrated as shown in Eq. (5).

$$\alpha_{eq} \cdot (T_2 - T_1) = \int_{T1}^{T_2} \alpha(t) \cdot dt \qquad (5)$$

Where T_1, and T_2 are the ambient (or temperature during installation) and design temperature respectively.

3.5 Loads

3.5.1 General

Global buckling shall be checked for the most critical 100-year return period load effects. Different load combinations for 100-year return period may be governing and must, hence, be checked. Typically, the following combinations load effects should be checked:

— functional design case; extreme functional load effect (100-year) with associated interference and environmental loads effects

— interference design case; extreme interference load effect with associated functional and environmental load effects

— environmental design case; extreme environmental load effect (100-year) with associated functional and interference load effects.

The value of the load effects shall be in accordance with the different limit states. Some guidance is given in Table 3-4.

3.5.2 Operational data (functional load)

The analyses shall be performed with relevant operational parameters (pressure and temperature). For the functional design case this shall represent the 100-year return value, normally the local incidental pressure, unless otherwise is stated in the limit state used.

Note:

DNV-OS-F101:2000 use Δp equal to $1.05(p_{ld}\text{-}p_e)$ in the local buckling limit state. DNV-OS-F101:2007 use $(p_{li}\text{-}p_e)$.

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For the interference design case, the value of the operational loads will depend on the probability of occurring simultaneously with the interference load. This implies that the operational load for the interference (trawl) design case will depend on the trawling frequency as given in Table 3-4.

The temperature profile corresponding to the relevant temperature shall be used (the local temperature). The insulation shall include conservative assumptions in order to ensure that these temperatures not will be exceeded corresponding to an annual probability of exceedance equal to 10^{-2}.

3.5.3 Trawling loads and frequencies (interference load)

The evaluation of trawling is based on the principles in DNV-RP-F111.

Note:

The trawl interference evaluation in this Recommended Practice is limited to lateral buckling only. Hence, trawling in free span is not covered by this Recommended Practice but by DNV-RP-F111.

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Note:

The effect of trawling in this Recommended Practice is included as a sensitivity study on the overall moment. Since the global buckling moment is mostly displacement controlled, the load controlled trawl moment will not be “added” but to a large extent “replacing” the functional moment from global buckling. If the contribution from the trawl is dominating, special evaluations are required in order to determine a higher γ_c than resulting from this Recommended Practice.

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The trawling frequency is annual per relevant pipeline section. For global buckling assessment where the trawl load acts as a triggering mechanism, the section relates to the part of the pipeline in a trawl area (order of kilometres) that has a potential for buckling.

For trawling assessment in a buckle, i.e. after it has buckled globally, it is assumed that a trawl board will hit the buckle near the apex in an unfavourable manner. The section length refers then to the length of the buckle or sum of buckles if more than one buckle is anticipated. The length of the relevant section is typically less than 100 metres per buckle.

The trawl pull-over load F_T depends on the trawl frequency f_T and trawl board type among other parameters. If detailed information is not available the values in Table 3-3 may be applied.

Table 3-3 Definition of characteristic trawl pull-over loads, F_T

Pull-over load	$f_T > 1$	$10^{-4} < f_T < 1$	$f_T < 10^{-4}$
F_T^{UB}	1.3 F_p	1.0 F_p	NA
F_T^{BE}	1.0 F_p	0.8 F_p	NA
F_T^{LB}	0.4 F_p	0.3 F_p	NA
F_p is the trawl pull-over load according to Sec.4 in DNV-RP-F111			

Table 3-3 shall both be used to evaluate the trigging of a pipe to buckle and to be used for the integrity check of a buckled pipe.

3.5.4 Environmental loads

If no information on most critical 100-year return period condition exists, the following combinations are proposed for pipelines on seabed, normally conservative:

— 100-year return period bottom current and 1-year return period wave induced flow
— 1-year return period bottom current and 100-year return period waved induce flow.

If no information on most critical 1-year return period condition exist, the following conservative combinations are proposed:

— 1-year return period bottom current and 1-year return wave induced flow.

Note:

The Environmental design case is normally not a governing design case for global buckling. One exception may be triggering of lateral buckle on even seabed.

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3.5.5 Load combinations

In general, all relevant load combinations shall be checked for a 100-year return period load effects. Table 3-4 gives guidance what will be the governing conditions in practice. For intermediate trawling intensity, this implies that two combinations have to be checked while one is sufficient for frequent trawling as well as for no trawling.

Table 3-4 Load combinations to be considered in the design

Trawling frequency 1)	*Scenario*	*Functional load*		*Trawl load 3)*	*Environmental load*
		Pressure load 2)	*Temperature load*		
$f_T < 10^{-4}$ and buried pipelines	Functional design	Local Incidental	Local Design	No	-
	Interference design	NA			
	Environmental design 4)	Local Operating	Local operating	No	100 yr
$10^{-4} < f_T < 1$	Functional design	Local Incidental	Local Design	No	-
	Interference design	Local Operating	Local operating	$F_T^{BE} = 0.8$	-
	Environmental design 4)	Local Operating	Local operating	No	100 yr
$1 < f_T$	Functional design	Local Incidental	Local Design	$F_T^{BE} = 1.0$	-
	Interference design	Identical with above			
	Environmental design 4)	Local Operating	Local operating	No	100 yr

1) Trawling frequency is defined in Sec.3.5.3.
2) See Note of Sec.3.5.2.
3) In addition to sensitivity analyses as required by the procedures.
4) This will normally not be a governing design case except for onset of global buckling on even seabed.

3.6 Time Effects

Properties may change with time, in particular pressure, temperature and wall thickness (corrosion). The same applies to the temperature profile.

In line with most pipeline design codes it is allowed to split the design life in different phases, e.g. combining the anticipated corrosion with corresponding pressure at that time. This requires adjustment of the associated control system for pressure and temperature. A combination of more corrosion and a lower temperature after a certain time can be highly beneficial.

In case the corrosion can be considered to be uniform, it is allowed to use a reasonably conservative estimate (i.e. little corrosion) on the wall thickness when calculating the load effect combined with capacity based on corroded cross section. This requires also that non-corroded section is acceptable when considering this in both load effect calculation and in the capacity. The reduced wall thickness due to corrosion for the load effect calculation is limited to maximum half the corrosion allowance. This is mentioned in the section of geometry above and load effect calculation in Sec.5.

4. Pipe-Soil Interaction

4.1 General

This sub-section gives an introduction to pipe-soil requirements while detailed formulations of soil resistance for scenario III are given in Appendix B.

In case other soil resistance models are applied these should be established in compliance with the described formulations in order to ensure safety level consistency.

Soil data for pipeline engineering is related to the upper layer of the seabed, often the upper 0.5 m and seldom below 2 m. This implies that the soil samplings performed with the objective to design load carrying structures may be of limited value. Normally, specific samplings are needed for the pipeline design.

Global buckling behaviour is strongly linked to the pipe-soil interaction. The pipe-soil interaction includes large uncertainties both due to variation and uncertainty in characterisation and is the most vital aspect of global buckling or expansion design.

The uncertainties related to pipe-soil interaction are often hard to quantify and a fair amount of engineering judgement is required.

The design procedures in this Recommended Practice are based on that safety factors will be determined based on the sensitivity to input parameters, in particular, the soil properties. A limited amount of survey data shall result in larger soil property ranges (upper bound, best estimate and lower bound). If the response sensitivity to these is large, it will give higher safety factors and more sampling may be of advantage. On the other hand, if the response sensitivity is small, further samples may be of limited value.

The components of the pipe-soil interaction involved in the potential buckling modes of a pipeline are the following:

— *Downward*
The downward stiffness is important for smoothening of survey data and for upheaval buckling design
— *Lateral*
For an exposed pipeline free to buckle laterally, the lateral pipe-soil interaction is the key parameter for the lateral buckling as it influences both mobilisation load (break-out resistance) and pipeline post buckling configuration (residual soil resistance after break-out).
— *Axial*
The axial pipe-soil interaction is relevant when any buckling mode is triggered as it affects the post-buckling configuration. The axial feed-in of the straight sections into the buckled region is determined by the mobilised axial reaction (of the natural soil and/or of the gravel/rock cover).
The axial pipe-soil interaction is also important for the axial load build-up; either at the pipeline ends or after a buckle has occurred.
— *Upward*
The vertical pipe-soil interaction during up-lift is relevant when upheaval buckling is of major concern, as it affects the mobilisation load. A multi-linear interaction model is normally required.

The selection of the most suitable formula/parameters in buckling analysis must therefore be guided by engineering judgement supported by experience on the specific problem and, where possible, by correlation/bench marketing with field measurements. In addition sensitivity analyses are always recommended, aimed at determining the criticality of project conditions with respect of modelling assumptions.

Simplifications of the pipe-soil interaction may be considered in the assessment. Emphasis should then be to make this simplification representative for the relevant condition; e.g. the model will be different when the breakout force shall be determined compared to determination of the post-buckling configuration.

4.2 Vertical Stiffness for Lay-Down

For vertical stiffness, reference is made to the DNV-RP-F105.

4.3 Exposed Pipes

For a pipeline laid on the seabed the axial and lateral resistance is mainly affected by:

— pipe embedment due to its submerged weight and installation loads (particularly the water filling and pressure test which may change the pipe penetration, alignment, smoothing the local curvatures, etc.)
— pipe load history
— time of consolidation between installation, gravel dumping, if any, and initial start-up
— axial and lateral displacements
— scouring.

In addition, the following aspects have to be accounted for when the pipeline is covered with spot or continuous gravel dumping to define the total resistance:

— additional embedment due to rock overburden
— rock penetration into the seabed
— geometry of cover
— additional embedment due to erosion and sediment of seabed soil.

The formulae for calculating the total resistance reported in literature vary due to three initial hypotheses/models. Generally speaking this is due to:

— differences inherent to the assumptions on which the proposed analytical models are based (for example, regarding soil failure mechanisms)
— extrapolation of experimental data with reference to limited case records
— simplifications which are the basis of numerical models.

The design of exposed pipelines will aim for; either to document that the pipeline will not buckle laterally, or, that the pipeline will buckle laterally and the post-buckling condition is acceptable. The two cases require different values, for small displacement or for large displacement. The amount of values depends on the design scenario and is given in Table 4-1.

Table 4-1 Required friction factors for pipelines left exposed on the seabed

Design scenario	*Axial* 1)	*Lateral*
No global buckling	BE	LB
Maybe global buckling	LB	LB
		BE
Global buckling	LB	LB
	BE	BE
	UB	UB

1) See also Sec.5.2.3.
LB Lower Bound
BE Best Estimate
UB Upper Bound.

Indices UB and LB indicate upper and lower bound value normally specified as mean value +/– 2 standard deviation (in accordance with normal interpretation), and index BE indicates a “best estimate”. The range in terms of standard deviations may be altered as it is included in the design procedure.

Pipe-soil resistance curve should be considered including peak resistance (break-out) and residual resistance.

4.4 Buried Pipelines

Buried pipelines are covered by soil, either with the seabed soil, trenched soil or with additional cover material (e.g. gravel). Soil properties will be required for each of the three materials as relevant in the design:

— in-situ soil conditions
— trench material (remoulded / fluidised and reconsolidated)
— added cover material.

The in-situ soil conditions are used for the downward resistance of the pipeline. This may be either on the seabed, for pipelines resting on the seabed and covered with gravel or at the bottom of the trench for a pipeline in a trench.

A trenched pipeline may be covered by natural seabed material or by gravel.

Natural seabed material can be put in place, either by ploughing back the material from the sides or by jetting where the material has been flushed backwards along the pipeline. When the soil is ploughed back, some parts of the soil will maintain its original strength while there may be water pockets in between these parts. The jetting trenching will, on the other hand, liquefy the soil, giving a very homogenous soil, and remould the strength. Attention should be paid to the characterisation of the back-filling material in the short and long term, as a variation of the characteristic soil parameters may be expected.

The properties of the gravel will depend on the source for the material and need to be defined correspondingly.

The modelling of pipe-soil interaction generally supplies analytical relationships to describe the ultimate soil capacity and the relative displacements at mobilisation of ultimate capacity.

These formulae/analytical relations generally involve the soil type/geo-technical properties and pipe cover characteristics, as well as pipeline diameter and potential cover height. A subdivision can be made on the basis of soil nature as:

— cohesive (clay)

— granular material (cohesion-less materials are sand, gravel and crushed rock).

For cohesion less material (typically sand), the internal friction angle is a function of the soil relative density, and hence can be subjected to variation from the short to the long term as a function of soil propensity to consolidate. The same applies to cohesive soils, as the shear strength is a function of soil consolidation. It should be considered that in this case trenching produces plastic collapse/fracture in the soil according to the degree of firmness, and the soil then includes separated blocks of clay that might develop some cohesion over time.

Figure 4-1 shows an idealized trench with a pipe covered with natural seabed material and added cover material.

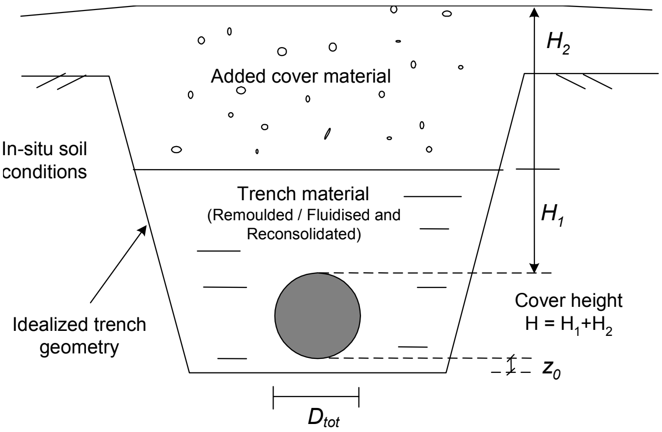

Figure 4-1
Illustration of some key issues and definitions used for Upheaval resistance

More detailed formulations are given in Appendix B.

5. Load effect calculation

5.1 General

In general, any model can be used calculating the load effect as long as it can be documented to give conservative results when compared to more advanced methods. In practice, analytical models may be used for simple configurations for exposed pipelines on even seabed, Scenario I, and buried pipes, Scenario III. For exposed pipelines on uneven seabed, Scenario II, and less simple configurations of Scenario I and III, several analyses with advanced finite element methods are normally required.

For all scenarios, the original configuration contributes strongly to the final stress-stage in the pipeline. It is therefore important to quantify and include it in rational manner, considering the uncertainties of the quantification.

5.2 Load Modelling

5.2.1 General

All loads experienced during construction and operations shall be considered including the reactions caused by these loads. The loads are divided into the following load categories:

— functional loads, both permanent and variable loads
— environmental loads
— interference (trawl) loads
— accidental loads.

For a general definition of the different load categories and the corresponding characteristic loads, see DNV-OS-F101 and for trawl loads, see DNV-RP-F111.

Note:
The sensitivity to the trawl loads is checked and results in the safety factor, γ_C. This is in contradiction to DNV-OS-F101:2000 and implies that trawl loads shall be considered as functional load.

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Load combinations shall be considered for all design phases relevant to the pipeline system. These may typically include:

— start-up
— operation at operational condition
— operation at design condition
— shut-down
— pack-in and shut-in condition.

For load effect modelling of the pipe the pipe shall normally be modelled with un-corroded cross section with the corroded section for the capacity check, see also Sec.3.3.

The thermal expansion is determined by the temperature difference between installation and the maximum expected temperature during operation. Note that the maximum temperature is likely to be governed at a high seawater temperature as this gives low cooling of the internal fluid.

5.2.2 Effective axial force

The effect of internal and external pressures may be taken into account using the concept of an effective axial force:

$$S = N - p_i \cdot A_i + p_e \cdot A_e \qquad (6)$$

The effective (axial) force represents the combined cross section action of; the pipe wall force (axial stress times the steel cross sectional area) and the internal/external pressure. By use of this effective force, the global buckling can be calculated as for members in air. For a more general discussion and application, see ref. /3/ and /4/.

5.2.3 Build up of effective axial force

The choice of pipe-soil axial resistance, f_a, will affect both the load effect calculations as well as the buckling capacity.

Within the anchor zone close to the pipeline end the effective axial force is reduced from maximum the total restrained axial force, S_0, due to end expansion. The reduction of the axial force along the pipeline is governed by the axial friction between the pipeline and soil. Hence, a high resistance will give higher forces close to the end, potential triggering buckles in this area.

For the upheaval buckling analyses the axial friction in the anchor zone shall be increased with γ_{UF}. This will result in the shorter effective anchor length.

The same considerations apply close to buckled sections. A high axial resistance will here cause a faster build up of axial force that may trigger other imperfections closer to the buckled section. Note that this may be highly beneficial for the design since more buckle may be triggered, giving less feed-in into each buckle.

Another related aspect is the post buckling force. The load in the buckle will reduce with increasing feed-in and not drop down to the lower bound value reached directly. This may imply that higher force close to the buckle is achieved prior to reaching this lower bound value, that may trigger the buckle.

5.3 Analytical Methods

5.3.1 Maximum expansion force

Simplified calculations shall always be used to verify the detailed calculations and to avoid gross errors. A pipeline may experience the fully restrained effective force as given in Eq. (7), given that the location is more than one anchor length from the pipeline end and no buckling has occurred (see also Sec. 2.1 and Figure 2-2).

$$S_0 = H - \Delta p_i \cdot A_i (1 - 2 \cdot \nu) - A_s \cdot E \cdot \alpha \cdot \Delta T \tag{7}$$

DNV-OS-F101:2007 Eq. (4.11)

Where

H Residual axial force from laying
Δp_i Difference in internal pressure compared to as laid!
ΔT Difference in temperature related to the temperature during installation.

NB! The temperature change in the pipeline can be related to a change in the surrounding temperature as well as in the internal temperature.

Note:

Δp_i is the difference in internal pressure between the analysed condition and when laid down with the lay tension H on the seabed. Since the internal pressure during installation normally is zero, this will often be identical to the internal pressure for the analysed condition.

This equation represents an analytically correct prediction for linear elastic material behaviour and will constitute and upper bound restrained force. For high utilisation, plasticity may be experienced and the above equation will then over predict the restrained force.

---e-n-d---of---N-o-t-e---

5.3.2 Global lateral buckling, Scenario I

Global lateral buckling of an exposed pipeline resting on even seabed may for some cases be analysed using analytical methods.

Different analytical models are available in the literature, see e.g. Hobbs, see ref. /1/, Taylor and Ben Gan, ref. /2/ and Spinazze et al, ref. /6/. However, analytical methods have several limitations due to the assumptions on which they are based:

— linear elastic material behaviour
— simplified axial and lateral pipe-soil interaction described by a Columb-friction
— small rotation theory
— imposed shape of initial and post buckling configuration according to assumed buckling mode. For small initial imperfection, mobilisation load is related to an assumed modal shape that may differ from the real pipe as-laid configuration.

If one or more of the limitations for the analytical methods are not fulfilled, more sophisticated analysis is required.

5.3.3 Upheaval buckling, Scenario III

For a covered pipeline, the lateral soil resistance is normally much higher than the vertical. Hence, the pipeline will tend to move vertically and try to break out of the soil cover (upheaval buckling). The approach will then be to design a sufficient cover height to prevent upheaval buckling and keep the pipe in its original position.

As for lateral buckling there is also analytical methods available in the literature for upheaval buckling calculation, see e.g. Palmer et al., see ref. /7/. The following linear relationship exists for a prop shape model:

$$S_{eff} = \left(R_{max} + w_p + k_2 \cdot w_o\right)\sqrt{\frac{EI}{k_1^2 \cdot \delta \cdot w_o}} \tag{8}$$

where R_{max} is the total soil resistance. w_o is the submerged weight of pipe during installation and w_p is the submerged weight during operation. δ is the (prop) imperfection, EI is the bending stiffness and $k_1 = 2$ and $k_2 = 11$ are constants determined from FE results for prop shape scenarios.

The above equation along with other analytical methods has their limitations, and should preferably be used only at the conceptual design phase. Typical limitations are:

— only linear elastic material behaviour
— difficult to describe an arbitrary imperfection shape
— soil upheaval resistance is assumed along the entire imperfection wavelength. This is not the case in sag bend regions, where the pipeline will have a tendency to move downwards, resulting in no soil contribution to uplift resistance.
— does not take the vertical soil resistance force-displacement curve into account
— cannot account for cyclic loading and possible creep.

5.4 Detailed FE Analyses

5.4.1 General

Normally, pipe response should be analysed using non-linear finite element methods. The FE analysis should describe the physical phenomena/behaviour adequately.

In general, the FE analysis shall be able to take into account such as:

1) *FE program specifics*

— **Non-linear material (steel) behaviour**
Shall take into account the non-linear and bi-dimensional (in the longitudinal and hoop-direction) state of stress by an appropriate yield surface and hardening rule.
The stress-strain curve based on yield stress and ultimate strength shall be based on the specified minimum values, f_y and f_u, considered being engineering stress values, except for when the mean value is explicitly required by the procedure. The choice of stress strain curve shall be consistent with the FE-program applied.

— **Large rotation theory**
Relevant for pipe rotation larger than about 0.1 radian.

— **Element size and type**
The pipe element type shall enable uniform hoop stress and effect of pressure. The length should typically be in the order of one diameter where the buckle is expected to occur and may be longer in straight portions.

2) *Modelling*

— **Pipe-soil interaction**
Pipe-soil interaction is generally modelled using a series of independent non-linear spring like elements attached to the pipeline, or to be modelled as a contact problem. These "springs" / contact surfaces are characterised by a non-linear force-displacement relationship and represent an integration of the normal and tangential forces acting on the pipe surface when it interfaces with the surrounding soil. Different pipe-soil properties in axial and lateral direction shall be accounted for.
The effect of peak resistance in the pipe-soil resistance shall be carefully evaluated. Omitting this peak resistance in the axial direction may trigger too few buckles and give a too long anchor length.
See also 5.2.3 and 5.4.3.

— **Initial pipeline configuration**
The development of the buckling mode (lateral buckling, up-heaval or a combination of the two) is affected by the pipeline as-laid configuration. The pipeline

should be stress free in a straight configuration. Hence, actual (measured) or assumed imperfection for triggering relevant buckling modes should be introduced from the initial straight and stress free pipe configuration. More details are given in 5.4.2.

For buried pipelines the fitting to the survey data, representing the as laid configuration shall not be done by "Smoothing" but by the stiffness of the pipe as outlined in 5.4.2.2.

— **Effective Axial Force Build up**

  See 5.2.3.

— **Analyse the relevant load sequence(s), including cyclic loading, if relevant**

  The effects of pipeline loading history (water filling, system pressure test, de-watering, shutdown-restart cycle etc.) should be accounted for in the analysis. The effect of cyclic load should be evaluated in the analysis to account for possible strain accumulation effects or reducing moment.

3) *Other*

— **Failure in FEM Analysis**

  Some upheaval buckling analysis addressed in this report requires finding an applied (T_{Rd}) temperature where the pipeline/soil interaction fails. The pipeline fails for UHB when the axial loading can not be increased or the mobilisation of the soil exceeds the failure displacement δ_F

— **Subsidance Related Horizontal Motion**

  Seabed subsidence due to e.g. reservoir depletion may cause both vertical and horizontal movement in the effective region. For pipelines with high axial restraint, such as those that are buried, the horizontal seabed movement can induce large axial forces leading to buckle of overstressing of the pipe. This shall be considered if relevant.

5.4.2 Modelling of the as laid configuration

5.4.2.1 General

It is recommended to verify the design by analysing the pipeline based on as-laid data for scenario I and II. It is required to base the final design of scenario III on as-laid data.

Modelling of the as laid configuration includes modelling of imperfections. Different types of imperfections are important for different scenarios. All these shall represent a stress-free condition in its straight configuration.

For Scenario I, Exposed pipe on even seabed, imperfections in the horizontal plane will be the governing imperfections. These imperfections may be both known (considerable/purposed built in initial imperfection/curves) or unknown. A larger imperfection than anticipated may be required in order to allow this to buckle for all sensitivity studies in line with the design procedure.

For Scenario II, Exposed pipe on un-even seabed, the governing imperfections are in the vertical plane. These imperfections will normally be modelled both in the design phase and after laying. However, an as-laid survey may reduce the uncertainties in the seabed bathymetry.

For Scenario III, Buried pipeline, the vertical out of straightness is most important. During the design phase, the imperfections may be uncertain, whereas after pipe laying, the out of straightness can be measured and hence, the imperfections will be known.

When the configuration is uncertain, a calibration of the structural model on the basis of as-built survey data is recommended to account for actual pipe-soil configuration (soil nature, pipe-soil penetration, free spanning length and clearance, etc.) and appropriate pipe-soil interaction curves (non linear force–displacement relationship).

5.4.2.2 Scenario III - Buried pipe

A central part of upheaval buckling analyses for the installed and buried pipeline is measurement of the pipeline configuration, the actual cover height and additional information as mean seabed and possible trench geometry.

The survey uncertainty of the configuration is significant, i.e. the vertical position of the pipeline is associated with a measurement error typically in the order of 0.1 – 0.3 m (±2 $\sigma_{configuration}$) with a certain spatial correlation.

An estimation of the survey accuracy should therefore be made for a series of independent measurement, by calculating the standard deviation for the measurements. This is normally not performed, but based upon experience and engineering judgement the accuracy for ROV based surveys assumes.

$\sigma_{configuration}$ = 0.05 – 0.15 m for Top Of Pipe

σ_{cover} = 0.10 – 0.15 m for cover height H

Survey data is normally given as data listings related to a KP.

Average spacing between each measurement should be in the order of a couple of diameters but not larger than 1 meter.

Post processing of configuration survey data to transform the measurement to data listing must be performed, but no smoothing of data should be carried out by the contractor performing the survey. The designer should carry out the smoothing, ensuring a consistent treatment of the raw data.

The procedure for "Smoothing" the data should be:

— Clearly un-physical data shall be removed. That is single points far out from the other data.
— Representative soil stiffness is applied to the remaining survey points.
— A straight stress free pipe in the as laid or trenched conditions (i.e. normally laid empty pipeline on seabed, changed to water filled) is lowered down on the soil springs (as a contact problem) without any axial friction applied. The downward stiffness should be taken as a best estimate. Alternative ways of simulating this is allowed and may depend on the FE-programs ability to model the contact problem.
— In this configuration, the soil springs not in contact with the pipeline are connected in a zero stress state simulating soil surrounding the pipe.

5.4.3 Pipe-soil interference modelling

5.4.3.1 Scenario I - Even seabed

For a pipeline laid on an even seabed, attention should be given to axial and lateral pipe-soil interaction.

The lateral pipe-soil interaction is the key parameter for the lateral buckling as it influences both mobilisation load (break-out resistance) and pipeline post buckling configuration. At mobilisation, when the pipeline starts to deflect laterally and the displacements are small, the lateral soil resistance is governed by the peak value. For increasing lateral displacements, the lateral soil resistance may decrease to a lower residual value. In the tail of the buckle the lateral resistance will always be peak resistance and this will influence the post buckling configuration. This controls the final configuration of the pipeline in the buckled zone characterised by large displacements.

The axial pipe-soil interaction will be the most important factor for the buckle initiations (load effect determination) and the amount of feed-in, i.e. in the post-buckling phase. The peak resistance versus the residual resistance may also be important.

5.4.3.2 Scenario II - Uneven seabed

For a pipeline laid on uneven seabed pipe mobilisation under the effect of operative loads, as well as pipe response in the post-buckling phase is in general related to a complex 3-D pipeline behaviour and pipe-soil interaction. The probability of the initial pipeline configuration to share the available thermal expansion over several buckles or to localise in one single buckle is determined by the initial pipeline configuration and soil pipe interaction. All pipe-soil interaction components (axial, lateral and vertical) must be properly defined in the model.

Particular attention should be paid to the prediction of pipeline as-laid configuration, both in the horizontal and vertical plane, as this determines which buckling modes (horizontal, vertical or a combination of the two) will be first triggered in operative conditions along the pipeline route. A 3-D seabed description is recommended in the analysis. Such detailed modelling must be based on accurate processing of survey data.

5.4.3.3 Scenario III - Buried pipe

Both the uplift resistance and downward resistance are essential for the upheaval buckling analyses. The upward resistance is often preliminary estimated by a linear uplift resistance before the actual resistance is modelled in the final analyses.

The downward stiffness will be altered as part of the design procedure in order to determine if there is a possibility for a downward initial failure. Detailed recommendations are given in Appendix B w.r.t. the pipe-soil modelling.

For the axial pipe-soil interaction, see 5.2.3.

5.5 Miscellaneous

For design dominated by thermal effects that are allowed to be released, the expansion of the pipe is not sensitive to the wall thickness and the reduction in wall thickness mostly governs the weight and thereby the lateral friction which then will be the most important effect.

Cyclic effects shall be considered. The curvature of global buckled pipelines often decrease when the buckling is repeated, hence, the maximum curvature occurs the first time the pipeline buckles (for invariant maximum effective axial force). It is allowed to take advantage of this effect if it can be documented by analyses and supported by procedures for operations.

If berms are built up during lateral movement, this may inhibit stress relaxation and shall be considered.

5.6 Engineering Tools

The load effect analyses will often be complex and time consuming. It is therefore important to limit the size of the element model as appropriate in order to simplify these analyses:

— A global model with course element size is normally required to determine where to split the model into smaller parts. A semi-analytical model may also be valuable at this stage.
— 2D FE model in the horizontal plane for exposed pipelines on even seabed.
— 2D FE-model in the vertical plane (modelling sea bottom unevenness and free spans) considering or not the possible development of thermal expansion in the lateral plane after the change of buckling plane from vertical to horizontal.
— 2½D or 3D FE model to consider pipeline uplift at the crest, change of buckling plane from horizontal to vertical and, finally, buckle development in the horizontal plane.

The pipeline designer will decide which approach to use on the basis of experience, design phase, etc.

6. I - Exposed Pipeline on Even Seabed

6.1 Objective and Applicability

The objective of this section is to provide procedures and criteria for pipelines on even seabed.

Pipeline resting on the seabed and exposed to compressive effective force, may buckle globally. When the initiation of global buckling takes place on the seabed, i.e. in the horizontal plane, it is defined to occur on an "even seabed" and this section is applicable.

6.2 Design Process

The design procedure is split into three main steps:

1) Global buckling assessment.
2) Pipe integrity check.
3) Mitigation measure check.

This is shown in Figure 6-1.

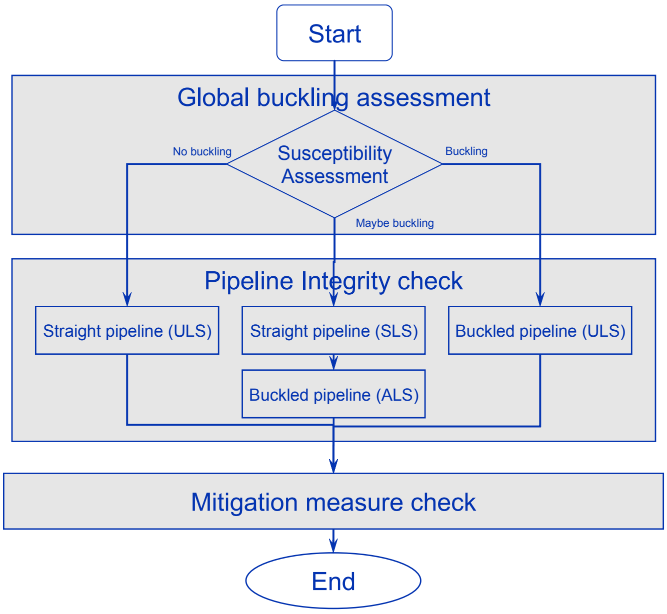

Figure 6-1
Overview of global buckling design flow

Step 1: Global buckling assessment

The objective of this step is to evaluate the susceptibility to global buckling. The axial load global resistance, or capacity, depends on the lateral load e.g. trawl impact, lateral restraint and geometrical imperfection of the global configuration as shown in Figure 6-2. Depending on the susceptibility; *No Buckling, Maybe Buckling* and *Buckling* different criteria for local buckling will be used.

Any purposely made imperfection to trigger global buckling shall trigger global buckling.

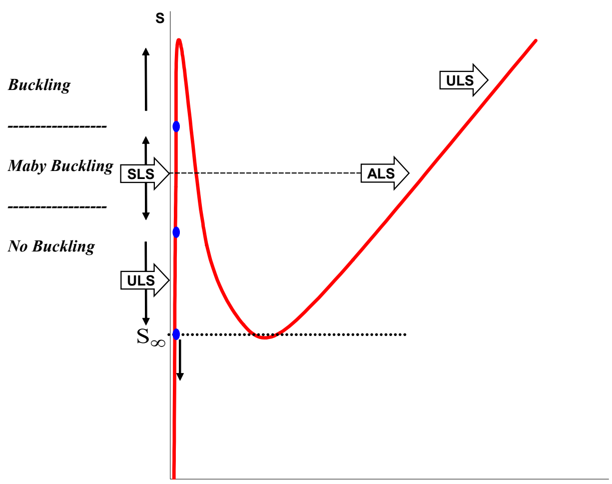

Figure 6-2
Illustration of the criteria for different buckling probabilities

Step 2: Check of the Pipe integrity

The major resulting failure mode for a global buckling is local buckling which constitutes the major part of the pipe integrity check. The extent of the local buckling check depend on the outcome from step 1. For a no buckling pipeline, standard checks only apply. For a buckling pipeline, different non-linear FE-analyses are required.

If the pipe integrity check fails, mitigation measures have to be defined and this step repeated until found acceptable.

Step 3: Check of mitigation measures

The objective of this step is to verify that:

— the mitigation measures introduced in the global buckling assessment have sufficient reliability in order to meet different criteria
— the pipeline behaves as anticipated.

If mitigation measures are included, the analyses used for the load effect modelling of the global buckling assessment shall represent the adopted mitigation measures. The design procedure in Step 2 is then checked with the updated load effects.

To verify that the pipeline behaves as anticipated is a part of ensuring a robust design.

6.3 Step 1: Global Buckling (Pre-buckling) Assessment

6.3.1 Triggering mechanism

For the global buckling assessment, two activation mechanisms shall be considered, see Figure 6-3:

1a External interference from trawl pull-over creating a deviation from rectilinear alignment large enough to activate snap-through and natural development of global lateral buckling.
1b Initial random imperfection (out of straightness) from laying.

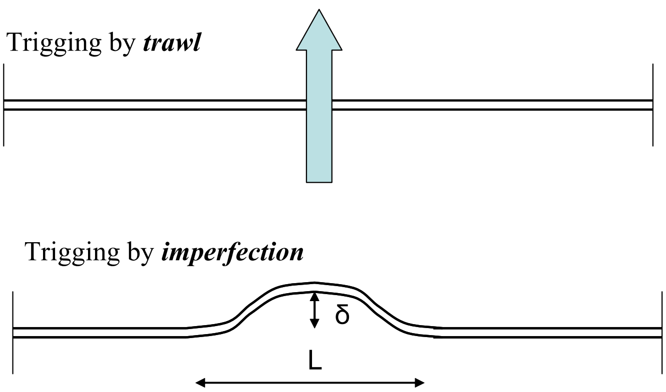

Figure 6-3
Triggering mechanisms of a global buckle

Purpose made imperfection for global buckling shall make the pipeline buckle.

The magnitude of natural out of straightness (i.e. with random location and not purpose-made by laying operation) is linked to the lay technology, relevant residual lay tension and the actual pipe size, i.e., bending stiffness and submerged weight.

A best estimate of maximum imperfection (or curvature) shall be established when design rely on natural imperfections only as part of the design basis.

6.3.2 Step 1a: Global lateral buckling activated by external interference

Trawl impact triggering global buckling shall be evaluated for a set of trawl pullover loads and pipe-soil resistances by FE analyses. The lateral soil friction and trawl pull-over load are defined by the soil-trawl matrix in Figure 6-4. The matrix implies a maximum of 3 FE analyses with different combinations of trawl load and lateral soil resistance forces.

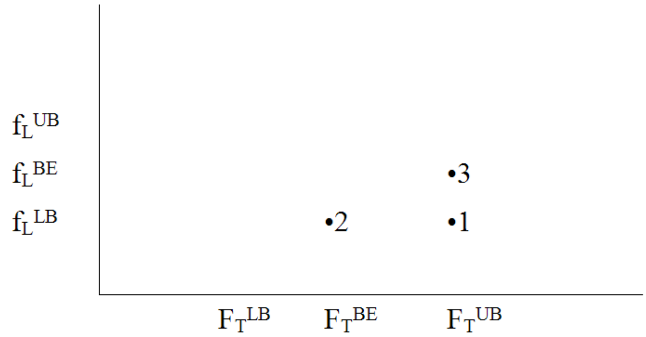

Figure 6-4
Combined lateral soil resistance and trawl load matrix

f_L is the lateral soil resistance force-displacement curve and F_T is the trawl load taken from Sec.3.5.3. Indices UB and LB indicate upper and lower bound value specified typically as a mean value +/- 2 standard deviation (see Sec.3.2), and index BE indicates a "best estimate".

The assessment is based on FE analyses of three scenarios using (F_T^{UB}, f_L^{LB}), denoted •1, (F_T^{BE}, f_L^{LB}), denoted •2 and, (F_T^{UB}, f_L^{BE}) denoted •3, and is performed as follows:

— A *No Buckling* condition is obtained if global buckling not occur for the scenario •1 using (F_T^{UB}, f_L^{LB})
— A *No Buckling* condition is obtained if neither of scenarios •2 or •3 experience global buckling
— A *Maybe Buckling* (SLS/ALS) condition is obtained if either scenario •2 or •3 experience global buckling and a single post-global buckling check is required, see 6.4.3.
— A Buckling (ULS) condition is obtained if both scenario •2 and •3 experience global buckling and a post-global buckling check with a full soil matrix is required, see 6.4.4 and Sec.9.

The FE analyses are performed with a straight pipe and pressure and temperature corresponding to the trawling frequency defined in Table 3-4. Trawl loads shall be included as per Table 3-3 and as per DNV-RP-F111. Hydrodynamic forces needs not to be included in these calculations as they may reduce the lateral pipe-soil resistance due to lift effects.

Hence, the design flow for triggering by trawl pull-over load will be as shown in Figure 6-5.

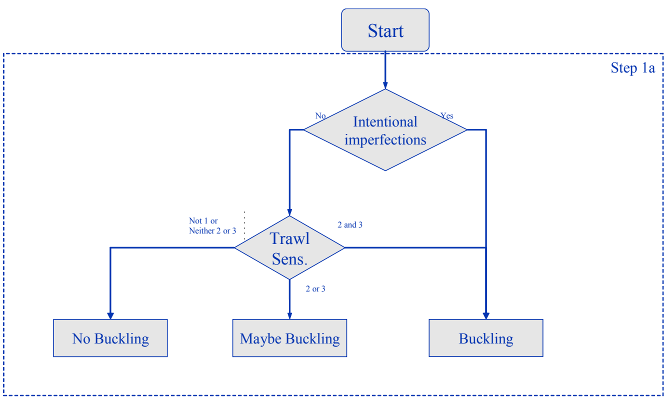

Figure 6-5
Design flow for Global buckling triggered by trawl pull-over

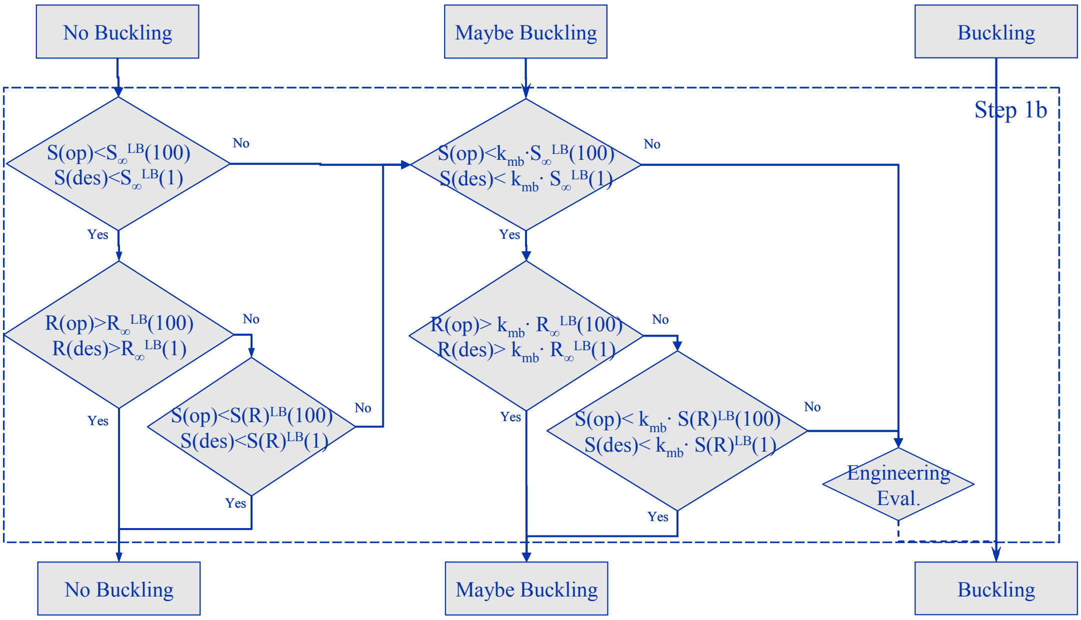

Figure 6-6
Design flow for imperfection triggering global buckling (Step 1b)

6.3.3 Step 1b: Global lateral buckling activated by imperfection

The lateral global buckling capacity is defined as the lower limit for available compression to activate lateral buckling. In the absence of trawl loads it depends on the out of straightness of pipeline alignment and soil lateral resistance.

The following procedure to assess the possibility for global lateral buckling may often result in global lateral buckling. Engineering judgement should then be applied to avoid unnecessary conservatism with respect to pipe-soil interaction capacity. E.g. the initial buckling may be governed by an extreme environmental condition (100-year return period), with a corresponding low lateral resistance (due to lift force and drag force), resulting in favourable buckle configuration. Further increase in pressure and temperature may, however, be based on higher lateral resistance representing a more normal environmental condition.

The design check for global buckling triggering by imperfection is based on Hobb's infinite mode capacity. This capacity depends on the lateral pipe-soil resistance which, in turn, will depend on the pipeline weight. The estimated weight shall include the lift effect from current and waves. Two combinations of loads shall be included in this consideration, see also Table 3-4;

1) Lower bound lateral resistance and extreme bottom flow with associated functional loads.
2) Lower bound lateral resistance, extreme functional loads with associated bottom flow.

Bottom current shall include both the contribution from current and waves. Benefit of non-simultaneously acting current from waves is allowed.

The design process for global buckling triggered by imperfection is shown in Figure 6-6.

No-Buckling implies that the pipeline will develop marginal deviation from as laid alignment and fulfil:

$$S(op) < S_\infty(100\,yr) \text{ and } S(des) < S_\infty(1\,yr) \tag{9}$$

where

S(op), S(des) is given by Eq. (7) based on operational pressure and temperature or design pressure and temperature, respectively.

$S\infty(100yr)$, $S\infty(1yr)$ is the effective axial force for the infinite buckling mode by Hobbs (see ref. /1/) based on lower bound pipe-soil resistance (f_L^{LB}) corresponding to 100-year or 1-year return period environmental condition

$$S_\infty = 2.29 \cdot \frac{EI}{\bar{L}^2} \tag{10}$$

$$\bar{L} = \left[\frac{(EI)^3}{\left(f_L^{LB}\right)^2 \cdot E \cdot A_s}\right]^{0.125} \tag{11}$$

$$f_L = \min\left[f_L^{LB}; \frac{f_L^{BE}(w - F_L) - F_D}{w}\right] \tag{12}$$

f_L^{LB} is the Lower Bound lateral soil resistance force [N/m],
f_L^{BE} is the Best Estimate lateral soil resistance,
w is the pipe submerged weight,
F_L is the maximum hydrodynamic lift force per unit length
F_D is the maximum hydrodynamic drag force per unit length.

Hence, f_L is taken as the minimum of the lower bound lateral soil resistance or the effective resistance in the presence on hydrodynamic loads from wave and current with a specified return period.

The Hobb's capacity may be related to a corresponding imperfection radius.

$$\bar{R}_\infty = 2.41 \cdot (D - t) \cdot \sqrt{\frac{E \cdot t}{f_L^{LB}}} \tag{13}$$

It shall be verified that this radius is less than the expected minimum radius of the pipeline stretch introduced during laying considering the laying technique applied.

In case a smaller radius can be expected e.g. a bend as part of the routing, a lower bound capacity corresponding to this radius shall be used for axial capacity evaluation. This shall include the effect from environmental loads as above.

$$S_{LB} = f_L^{LB} \cdot R_{LB} \tag{14}$$

where R_{LB} is the expected lower bound radius.

For *Maybe Buckling* the same procedure and criteria applies but with an axial capacity increased by a factor k_{mb}. This shall be based on engineering judgment and is expected to be in the order of 1.5.

If all the above criteria fail the pipeline will normally be considered as a *Buckling* pipeline.

6.4 Step 2: Pipe Integrity Check

6.4.1 General

As stated in the introduction, Global buckling is not a failure mode in itself but may cause other failure modes such as; Local Buckling, Fracture and Fatigue. Hence, after the global buckling check, the pipe shall be checked for different failure modes, referred to as Pipe Integrity Checks, in the identified condition i.e.:

— *no buckling*; ULS check for the “straight” pipeline
— *maybe buckling*; SLS check for the “straight” pipeline and ALS for the Buckled
— *buckling*; ULS check in the buckled (post-buckling) configuration.

An overview of the design tasks for pipe integrity checks are given in Figure 6-7.

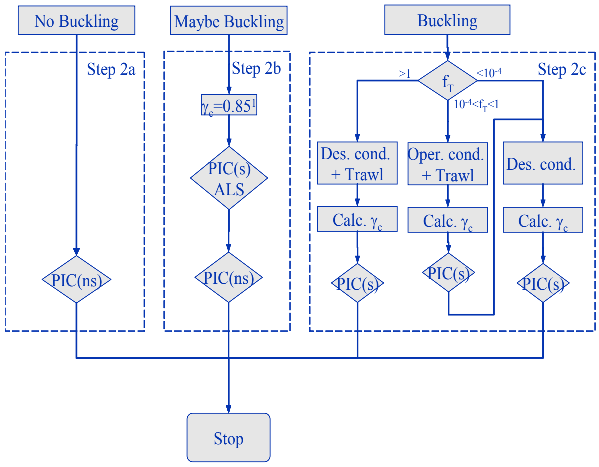

Figure 6-7
Pipe integrity check overview

All integrity checks shall be performed for pressure and temperature values corresponding to the trawl frequency as per Table 3-4

An overview of the required pipe integrity checks is given in Table 6-1 but see also Sec.6.4.5.

Table 6-1 Overview of most relevant pipe integrity checks

	Pressure Contain.	*Local buckling*		*Axial*	*Ratcheting*	*Fatigue*	*Fracture 3)*	*Trawl interf.*	*Free span*
		Load Controlled	Displacement Controlled						
Reference	DNV-OS-F101	DNV-OS-F101	DNV-OS-F101	Eq. (65)	DNV-OS-F101	DNV-OS-F101	DNV-OS-F101	DNV-RP-F111	DNV-RP-F105
No Buckling									
Prior	X	X	-	X	X	X	-	X	X
Post	-	-	-	-	-	-	-	-	-
Maybe Buckling									
Prior	X	X	-	X	X	X	-	X	X
Post	X	$\gamma_c = 0.85$ 1)	-	-	-	-	X	X	X
Buckling									
Prior	X	X	-	X	X	X	-	X	X
Post	X	γ_c 2)	All		X	X	X	X	X

1) To be analyses as an Accidental Limit State, see 6.4.3.
2) Typically [0.80;1.0], see. Eq. 6.4.415). See also 6.4.4.
3) See also comment in Sec.9.

This table does not rule out other checks required by DNV-OS-F101.

6.4.2 No buckling condition – Step 2a

This shall be checked as a normal pipe, including the axial check, as per Table 6-1.

6.4.3 Maybe buckling condition – Step 2b

The condition prior to any buckling shall be checked for axial capacity as per Table 6-1. Note that this will be checked as an ULS check.

The post buckling condition for the Maybe Buckling condition shall be checked as an ALS condition based on best estimate lateral resistance, best estimated axial resistance and best estimate trawl loads, if applicable. A simplified ALS check can be carried out assuming γ_c equal to 0.85 and γ_F = 1.0 in line with DNV-OS-F101, see also 5.4.2.1.

As an alternative, the methodology of buckling condition can be applied.

6.4.4 Buckling condition – Step 2c

In order to document the integrity in the post buckling condition a set of non-linear FE-analyses is required.

The major design check in the post buckling condition in addition to fracture is local buckling for combined loading. By varying the basic parameters, a set of possible responses in terms of moments and strains is determined.

The purpose of the required analyses for the buckling conditions is therefore to determine a specific γ_c in line with the paragraph above giving a γ_c larger than 0.80. A γ_c of 1.0 will correspond to a fully load controlled condition for best estimate values which may not give sufficient safety. In case of a γ_c above 1.0, it is recommended to re-assess the design solution.

The pipeline integrity check for local buckling is satisfied if:

- combined moment check for stress-strain curve based on f_y and f_u, best estimate values of pipe-soil resistance, best estimate (if relevant) trawl loads and determined γ_c
- strain criterion for worst case (of the analysed scenarios in determination of γc)

are fulfilled.

Formally this implies that:

$$M_F\left(f_L^{BE}; f_A^{BE}; f_c; F_c\right) \cdot \gamma_F \cdot \gamma_c \le \frac{M_c}{\gamma_m \cdot \gamma_{SC}} \tag{15}$$

$$\varepsilon_d \cdot SNCF \le \frac{\varepsilon_c}{\gamma_\varepsilon} \quad \text{(For all sensitivity analyses)} \tag{16}$$

The condition factor γ_C is based on the prevailing uncertainty in the response bending moment as per Sec.9 for the parameters given in Table 3-4. These shall be checked for the design criterion in Eq. (15) for the best estimate parameters moment.

The displacement controlled condition, Eq. (16), shall also include relevant SNCF's. The worst case, out of the analysed scenarios, is normally one out of:

- $\varepsilon(f_L^{UB}, f_A^{BE}, f_y, F_T^{BE})$
- $\varepsilon(f_L^{BE}, f_A^{BE}, f_y, F_T^{UB})$.

It is only the scenario with medium range trawling that needs to be analysed for two set of loading conditions. Calculation of γ_c for the extreme functional loads case can be simplified by using the same expression as for trawling, just put the trawling uncertainty; $X_C = 0$.

γ_c tends to be overestimated if calculated based on a not design governing condition. Hence, it should be calculated for the governing condition in design. The simplified calculation of γ_c above is based on trawl combined with operational pressure and temperature. In case functional design loads (no trawl) is a more severe condition, this should be the basis for calculation of γ_c. X_c should then be calculated for trawl condition to modify the γ_c calculated without trawl.

In case the buckling occurs due to un-intentional imperfections for buckling, these shall be increased to allow buckling for upper bound lateral soil model pipe integrity check, causing the pipeline to buckle for all sensitivity analyses.

Additional required checks of the pipe itself are given in Table 6-1.

6.4.5 Pipeline walking

In addition to the above limit states, there is a phenomena called Pipeline walking. A pipeline may move axially for each start-up and shut down for pipelines with steep transient thermal gradient and that:

- do not have a point of full axial restraint (will move towards cold end)
- have a pulling forces in the end (will move towards pulling end, typical SCR's)
- rest on a slope (will move downwards).

Several of these phenomena's may occur simultaneously and have contradicting or adding effects. This shall be considered if relevant, see ref. /8/.

6.5 Step 3: Mitigation Measures

6.5.1 General

A pipeline which is designed to buckle should be based on a "robust design". This implies that sensitivities have to be performed covering e.g.:

- may adjacent imperfections buckle if the buckling capacity of the considered imperfection is slightly higher?
- modelling of lateral pipe-soil capacity; parts of the pipeline that moves little may have a higher lateral capacity than parts with larger movement – how will this affect the response
- is it certain that vertical undulation in the seabed not will trigger a vertical buckle?

A robust design may be achieved by a combination of imperfections and mitigation measures. A general description of mitigation measures is given in Appendix A.

6.5.2 Sharing of expansion into adjacent buckles

An exposed HT/HP pipeline may not only buckle at one location, but at a series of locations. In this Recommended Practice sharing of expansion into adjacent buckles means that the expansion potential is released into imperfections in the pipe configuration at various locations. A mitigation measure to obtain this effect is if the pipeline can be installed with imperfection. If the benefit of sharing is taken into account in the design a safety margin should be documented to avoid excessive feed-in to one of adjacent, pre-defined imperfections.

At start-up the behaviour will be as described in Sec.2.1 and shown in Figure 2-2 and Figure 2-3. In order to activate the second imperfection to buckle (i.e. initiate lateral deflection), a build up of sufficient effective axial force between the adjacent buckles is required. If build up of effective axial force when reaching at the second buckle is less than the axial global buckling capacity, this buckle will not be triggered and localisation of axial feed-in into the first buckle will occur. Potential localisation may be related to:

- in-homogeneous as-laid configuration (as-laid curvature)
- in-homogeneous pipe-soil interaction
- varying (spatial) initial compressive force.

The localisation between two interacting buckles is determined by the axial equilibrium between the difference in force in the buckles and the axial resistance mobilised in the pipe section between the two buckles. Sharing between adjacent buckles can be taken into account if the following equation is fulfilled:

$$S_{r,1} + \Delta S \ge S_{G,2} \tag{17}$$

where

$S_{r,1}$	is the post-buckle effective axial force in the first buckle, calculated assuming lower bound soil characteristics (may conservatively be set to zero)
ΔS	is the axial force build up between the adjacent buckles calculated assuming Lower Bound soil characteristic
$S_{G,2}$	is the axial global buckling capacity force for the second buckle. The capacity must formally be taken as:

$$S_{G,2} = Max\left[S_G\left(R_2^m, f_L^{UB}\right), S_G\left(R_2^{UB}, f_L^m\right)\right] \tag{18}$$

where R_2 is the radius (imperfection) at the 2^{nd} buckle. Hence, the global buckling capacity is conservatively assumed to occur for upper bound lateral resistance at expected imperfection or at a lower bound imperfection at expected axial resistance.

The basic principle is illustrated in Figure 6-8 where it is assumed that the 1st buckle is initiated by some means. The objective is then to evaluate whether the axial force build up combined with the $S_{G,2}$ is sufficient to initiate the 2nd buckle, see also 5.2.3.

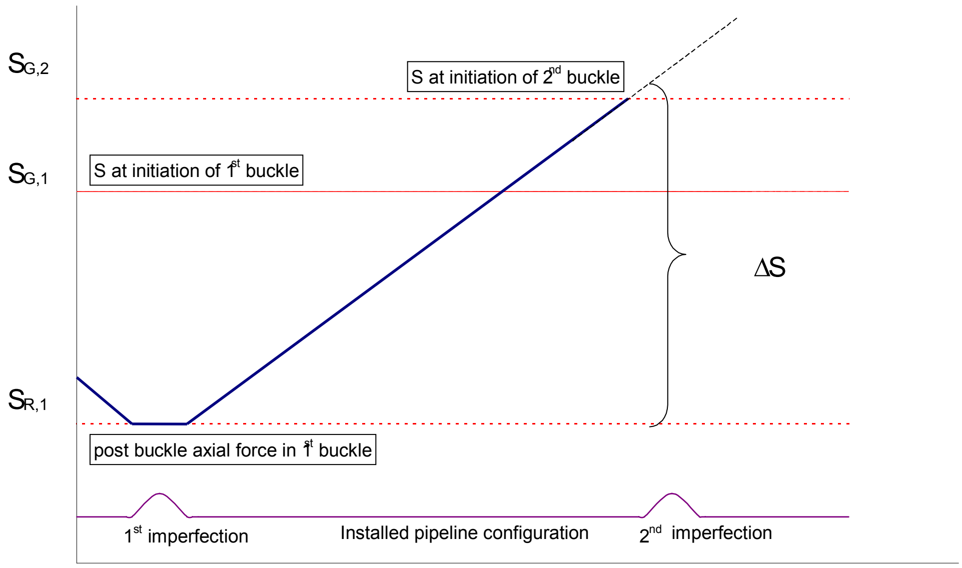

Figure 6-8
Sharing of buckles. Basic Principle

6.5.3 Increasing axial restraint

In case the safety margin given in Eq. (17) is not fulfilled one solution is to increase the axial restraint by e.g. rock dumping.

6.5.4 Monitoring system

If the principle of sharing of expansion into adjacent buckles is to be used a monitoring programme of the pipeline behaviour is recommended. Such programme is intended to verify that the pipeline expand into the buckles originally defined.

7. II - Exposed Pipeline on un-even seabed

7.1 Objective and Applicability

The objective of this section is to provide procedures and criteria for pipelines on un-even seabed.

A pipeline resting on the seabed exposed to compressive effective force may buckle globally. In case this deformation initially takes place in the vertical plane or for combination of un-even seabed and scenario I, e.g. for curves on un-even seabed. it is defined to occur on “un-even seabed” and this section applicable.

7.2 Design Process

A pipeline on an un-even seabed has a more defined imperfection, in the vertical plane, than a pipeline on an even seabed. The global buckling assessment is therefore easier to predict on an un-even seabed than for an even seabed but the FE-analyses will be more complicated.

A pipeline on un-even seabed has also a larger expansion capability than on a flat seabed. Hence, a pipeline that has been designed for even seabed will be acceptable also on un-even seabed without further evaluations from the global and local buckling point of view given that the moment contribution from the vertical plane due to the unevenness is negligible. This also implies that the feed-in length to each buckle shall be equal or less than for the even seabed, see Sec.7.4 Level 1. Standard design checks with respect to spans and other limit states have, however, to be carried out on the actual topography. The design procedure in this section will allow further optimization with respect to expansion capability.

The design procedure is intended for local buckling check of the expansion buckle by means of the condition factor in line with for even seabed. The condition factor calculated in this section should not be combined with condition factor for uneven seabed for the buckled section. For other parts of the pipeline, normal ULS checks shall be applied (e.g. with $\gamma_c = 1.07$, and trawling interference in compliance with DNV-RP-F111). Hence, this Recommended Practice does not apply to integrity check in free spans.

The design process will include three steps:

1) Global buckling assessment.
2) Pipe integrity check.
3) Mitigation measure check.

Step 1: Global buckling assessment

The objective of this step is to determine what buckling modes will be relevant and how far this will proceed. A pipeline resting on uneven seabed may experience the following three global configuration phases:

Phase 1: The free spans deflect and may get in touch with the sea bottom.

Phase 2: Uplift at crests

— Further expansion may lift off the pipe at a few crests in a limited way: say, less than 50% of the pipe diameter for a lifted length less than 50 diameters, being stable against horizontal perturbations.
— Even further expansion may increase the upward displacements at crests, (free span shoulders, of the most pronounced undulations), bifurcate and turn down to seabed developing lateral buckling in the seabed plane.

Phase 3: Even further expansion will increase the bending of the buckled pipeline.

An important practical task in the design is to limit the complexity of the analyses which can be huge, by use of 2D, 2 ½ D or 3D models when possible. Specific design checks may therefore be required to prove the relevancy of the simplified models.

Step 2: Check of pipe integrity

The major resulting failure mode for global buckling in addition to fracture is the local buckling. The objective of this step is, hence, to document that the local buckling of the pipe in the different phases is maintained. In phase three, a similar approach as for Scenario I is used with:

— a combined moment check for best estimate values based on a γ_c, determined by a specified set of sensitivity analyses
— a strain capacity check, to be applied to all sensitivity analyses.

If this check fails, mitigation measures have to be defined and this step repeated until found acceptable.

Step 3: Check of mitigation measures

The objective of this step is to verify that the mitigation measures introduced in the global buckling assessment in order to meet different criteria have sufficient reliability.

If mitigation measures are included, the analyses used for the load effect modelling of the global buckling assessment shall represent the adopted mitigation measures. The design procedure in Step 2 is then checked with the updated load effects.

7.3 Step 1: Global Buckling (Pre-Buckling) Assessment

Documentation of global buckling assessment can be performed either by simplified 2D analysis or by more complex 2½D or 3D analysis.

In case of using simplified 2D analyses in the vertical plane, the following check shall be performed.

$$S_R\left(p_{li}, T_{1,\max}\right) \le \frac{\pi^2 \cdot EI}{\left(L_{uplift}\left(S_R\right)\right)^2} \quad \text{No lateral buckling} \tag{19}$$

$$S_R\left(p_{li}, T_{1,\max}\right) \ge \frac{4 \cdot \pi^2 \cdot EI}{\left(L_{uplift}\left(S_R\right)\right)^2} \quad \text{Lateral buckling} \tag{20}$$

where

S_R is the effective axial force in the uplifted span section
L_{uplift} is the length of the pipeline length lifted off at the free span crests.

If Eq. (19) (pin-pin Euler buckling length) is fulfilled, the uplifted section remains in the plane and the 2D analyses is sufficient. If Eq. (20) (fixed-fixed Euler buckling length) is fulfilled, the uplifted section will buckle laterally and 2D analyses of even seabed or a more optimised design by 2½D or 3D shall be performed. Eq. (20) is valid only if the section modulus is the same in all directions, i.e. that yielding does not occur prior to lateral buckling.

For an effective axial force in between Eq. (19) and (20) a 2½D or 3D is required to document the global buckling behaviour.

If 2½D or 3D analyses are used, the above checks are not required.

In case the analysed pipeline stretch is mainly free spanning, the classical free-span design shall be performed in accordance to the criteria given in DNV-OS-F101, DNV-RP-F105 and DNV-RP-F111. If the pipe structural integrity is assured, the analysis stops, otherwise traditional intervention works are implemented.

In the other case i.e. pipe uplift at free span shoulders is relevant for the analysed pipeline stretch, the advanced design procedure has to be followed.

7.4 Step 2: Pipe Integrity Checks

7.4.1 General

The way to document the integrity in the post buckling condition is following the same principles as for even seabed.

Table 7-1 has grouped the different limit states corresponding to the different phases:

— *Phase 1 – Traditional design*
This check is relevant for the free spanning section and pipe sections in contact with the sea bottom not subjected to uplift, lateral turn down and subsequent lateral buckling.
— *Phase 2 – Uplifted pipe sections not buckling*
This check is relevant for the uplifted pipe sections in case the change of the buckling plane does not occur or is not allowed. The check for plasticity prior to turning lateral is to validate the Euler buckling formulation only and is not required if a full 2½D or 3D model is used.
— *Phase 3 – Laterally buckled pipe sections*
This check is relevant for the pipe sections in contact with the sea bottom subject to lateral buckling after uplift/lateral turn down.

7.4.2 Phase 3: Lateral buckling

If the buckle turns lateral, a series of analyses have to be performed in order to determine the partial safety factor γ_c reflecting the uncertainty in load effects due to the inherent variability of basis parameters.

The pipeline integrity check for local buckling is then satisfied if, identical with for even seabed Sec.6, with γ_c as outlined in Sec.9:

— combined moment check for stress-strain curve based on f_y and f_u, best estimate values of pipe-soil resistance, best estimate (if relevant) trawl loads and determined γ_c
— strain criterion for worst case (of the analysed scenarios in determination of γc).

are fulfilled.

Formally this implies that:

$$M_F\left(f_L^{BE}; f_A^{BE}; f_c; F_c\right) \cdot \gamma_F \cdot \gamma_c \le \frac{M_c}{\gamma_m \cdot \gamma_{SC}} \tag{15}$$

$$\varepsilon_d \cdot SNCF \le \frac{\varepsilon_c}{\gamma_\varepsilon} \quad \text{(For all sensitivity analyses)} \tag{16}$$

As for the flat sea bed scenario, the condition load factor may be defined separating the different sources of uncertainties. By following the same procedure as for the even seabed scenario, but with more refined 2½D or 3D FE-models, it will lead to a lot of heavy and time consuming analyses. Hence, three different levels are distinguished in the calibration procedure for the condition load factor, γ_C.

Level 1

The pipe integrity check is based on an analysis of the real pipeline on real seabed for best estimate pipe-soil properties and a condition factor calculated for the pipeline on a flat seabed.

The condition factor and pitch length (i.e. length between each buckle) is calculated for the actual pipeline but based on a flat sea bed scenario. Note that for Level 1, the length between two buckles for the uneven scenario must be equal or shorter than the length used when calculated the condition factor and pitch length for even seabed.

A verification of the applicability of using the condition $\gamma_{C,Flat}$, for the uneven sea bed scenario can be performed by carrying out 2D FE analyses in the vertical plane (using the actual sea bed topography) with Best estimate values of the relevant parameters for both "flat" and uneven scenarios. For the uneven scenario, relevant mitigation measures should be included in the analyses, while for the "flat" scenario, mitigations in all free spans should be in included in the analyses. If the ratio of the bending moment in the uplifted section calculated in the "flat" and uneven sea bottom scenario, $M_{F,Flat}/M_{F,Uneven}$, is comprised between ± 3% - 5%, the free span distribution adjacent to the buckle does not affect significantly the development of bending moment at the buckle and the load condition factor and pitch (L_{pitch}) calculated for the flat seabed scenario may be used. Otherwise, an optimization of the condition factor based on Level 2 or Level 3 is recommended.

Level 2

The pipe integrity check is based on an analysis of the real pipeline on real seabed for best estimate pipe-soil properties and a condition factor calculated for the pipeline on a flat seabed but adjusted for the axial pipe-soil resistance sensitivity (X_A).

Calculation of a new load condition factors for the uneven sea bottom scenarios, $\gamma_{C,\ Uneven,\ Lat}$, are based on correcting the load condition factor, $\gamma_{C,Flat}$, for the flat sea bottom, in order to consider the effect of free span in the near buckling conditions adjacent to the buckle. Note that for Level 2, the distance between two buckles can be adjusted compared to the flat seabed scenario. The calculated condition factor is in principle only valid for the buckle location where it is calculated, i.e. new updated expression for $CoV(X_A)$ for each buckle location has to be calculated.

Two additional 3D FE analyses of the pipeline laid on the uneven sea bottom scenario, with relevant mitigation measures, are required to calculate the effect of the free spans adjacent to the buckle. These analyses are used to calculate a new coefficient of variation for the equivalent axial resistance factor ($CoV(X_A)$), as specified in Sec.9. The condition factor for uneven seabed, $\gamma_{C,\ Uneven,\ Lat}$, is found by replacing the expression of $CoV(X_A)$, in the calculation of, $\gamma_{C,Flat}$.

Level 3

The pipe integrity check is based on both sensitivity analyses (to determine γ_C) and pipe integrity analysis of the real pipeline on the real seabed.

Calculation of a new load condition factor, $\gamma_{C,\ Uneven,\ Lat}$, performing a fully 2½D or 3D FE analysis (considering and not considering the real 3D roughness of the sea bottom). The calculated condition factor is in principle only valid for the buckle location where it is calculated, i.e. new condition factors has to be calculated for each buckle location.

The effect of the soil resistance matrix and free spans are analysed using a 2½D or 3D FE models. Level 3 is recommended in cases where the pipeline configuration in the lateral plane, due to local built-in lateral curves or sea bottom roughness transversal to the pipeline route, has to match with 3D unevenness and pressure loads and thermal expansion are released by a combination of pipeline uplift/turn-down/lateral buckling at the crests of the most pronounced undulations and lateral buckling developing at the built-in curves or along the transversal down slope.

This means a γ_c larger than 0.80 (for γ_c above 1.0 see 6.4.4). The formal design criterion then becomes identical with even seabed, Eq. (15) and Eq. (16), applying the calculated γ_c for the best estimate parameters.

Phase 3 shall be checked for the most likely combination of extreme loads. This means that it shall be performed for the conditions given in Table 3-4 and also implies that two sets of analyses may be required for pipelines exposed to the mid-range frequency of trawling ($1\text{-}10^{-4}$).

Table 7-1 Required and Normally most governing pipe integrity checks

	Pressure Cont.[1)]	*Local buckling*		*Axial[2)]*	*Ratche-ting[2)]*	*Fatigue[1)]*	*Fracture*	*Trawl interf.*	*Free span*
		Load Controlled	Displacement Controlled						
Phase	DNV-OS-F101	DNV-OS-F101	DNV-OS-F101	Eq. (65)	DNV-OS-F101	DNV-OS-F101	DNV-OS-F101	DNV-RP-F111	DNV-RP-F105
1 No Uplift	X	X[1)]	X[1)]-	X	X	X	X	X	X
2 Uplift									
No lateral t.d.	X	X[2)]	-	X	X	X	X	X	X
Prior to lateral t.d.	-	X[3)]	-	-	-	-	-	-	-
3 Lateral buckling	X	γ_c[4)]	All[5)]	X		X	X	X	X

t.d. turn down

1) Depending on scenario.
2) Normal design applies, i.e. with γc = 1.07.
3) The pipe shall not have yielded prior to turning lateral, i.e. $\sigma_{eq} < f_y$. if the Euler buckling check in Eq. (19) or (20) are applied. Else, full 2½D or 3D analyses are required.
4) > 0.80, see Eq. (15) and Sec.9.
5) See. Sec.7.4, Eq. (16).

In addition to the above limit states, Pipeline walking has to be considered, Sec.6.4.5.

7.5 Step 3: Mitigation Measure Checks

The same mitigation measure checks as in 6.5 apply to un-even seabed, see also Appendix A. For check of robustness it is in particular important to check the sensitivity to neighbouring vertical imperfections due to configuration uncertainty.

8. III - Buried pipeline

8.1 Objective and Applicability

The objective of this section is to provide procedures and criteria for upheaval buckling design and required overburden. It applies to pipelines that are buried in order to avoid global buckling.

A buried pipeline exposed to compressive effective axial forces may get unstable and move vertically out of the seabed if the cover has insufficient resistance. An out-of-straightness configuration will result in forces acting on the cover, perpendicular to the pipeline. In case these vertical forces exceed the cover resistance the pipeline will buckle upwards. This may be acceptable if the pipe integrity can be documented in the post-buckled condition. In this recommended Practice, no guidance on pipe integrity check in the post-buckled condition is provided and upheaval buckling is therefore considered as an ULS failure. In the following a design procedure is given ensuring that the pipeline remains in place with for given probability of failure.

In this Recommended Practice, buried pipelines are therefore designed to remain in place.

Since seismic design causes dynamic excitation (lateral and vertical) and since most upheaval buckling mitigations are by gravitational rather than mechanical restraint type, it should be considered whether the design methodology is sufficient or a separate dynamic (or quasi static) analyses is required. There is also the related question whether soil or gravel is susceptible to liquefaction.

8.2 Design Process

The upheaval buckling design is normally performed at two stages:

1) Pre-installed design phase.
2) As-installed design phase.

Each of these phases is based on the same criteria but with assumed or measured configuration. When the configuration has been determined, it is common practice to calculate a tentative overburden before applying complex analyses.

The resulting soil cover shall be based upon maximum cover coming from two criteria:

— specific cover requirement
— minimum cover requirement.

Analyses of the measured pipeline configuration will give necessary uplift resistance. This *specific cover resistance* will then vary along the pipeline depending on the pipeline configuration (curvature), pressure and temperature. For sand and rock, this specific cover resistance will normally be referred to as the *Specific cover height* H_{spec}.

For upheaval buckling, the downward stiffness may also be important. When the uplift resistance reaches a certain magnitude, the downward deformation will increase. This will imply that the curvature of the uplift section will get increased curvature and eventually penetrate the cover and cause an upward failure.

This implies that for higher uplift resistance the result will be more dependent on the downward stiffness while for lower uplift resistance the characteristic failure temperature will be given by the lower bound uplift resistance with the partial safety factor γ_{UR}.

Survey methods commonly used to determine the configuration of pipelines includes certain uncertainties. A *minimum cover resistance* shall therefore be determined by analyses of a supposed out-of-straightness configuration not detected by the configuration survey. This out of straightness configuration shall be taken as a prop shape configuration with the height of the prop-shape (δ) equal to the standard deviation of the configuration survey accuracy. For sand and rock, this minimum cover resistance will normally be referred to as the *Minimum cover height* H_{min}.

In accordance with the above, the design process criteria are organised in the following steps:

1) specific cover design
   — initial configuration
   — soil resistance modelling
   — upheaval buckling design criterion.
2) minimum cover design
3) specification of cover
4) pipe integrity check.

8.3 Step 1: Specific Cover Design

8.3.1 Initial configuration

The initial configuration in the as trenching/as laid condition shall be determined from the survey data as described in 5.4.2.2. The advantage of several surveys are described in Sec.8.5.2.

8.3.2 Soil Resistance modelling

A tentative soil cover may be determined in some way. This soil cover shall then be verified with the procedures as shown in this section.

The soil characteristics of the analysed cover shall be determined and modelled in line with Sec.4. In the FE model, this lower bound response curve shall be reduced with the safety factor γ_{UR} in Eqs. (21) and (22) as illustrated in Figure 8-1.

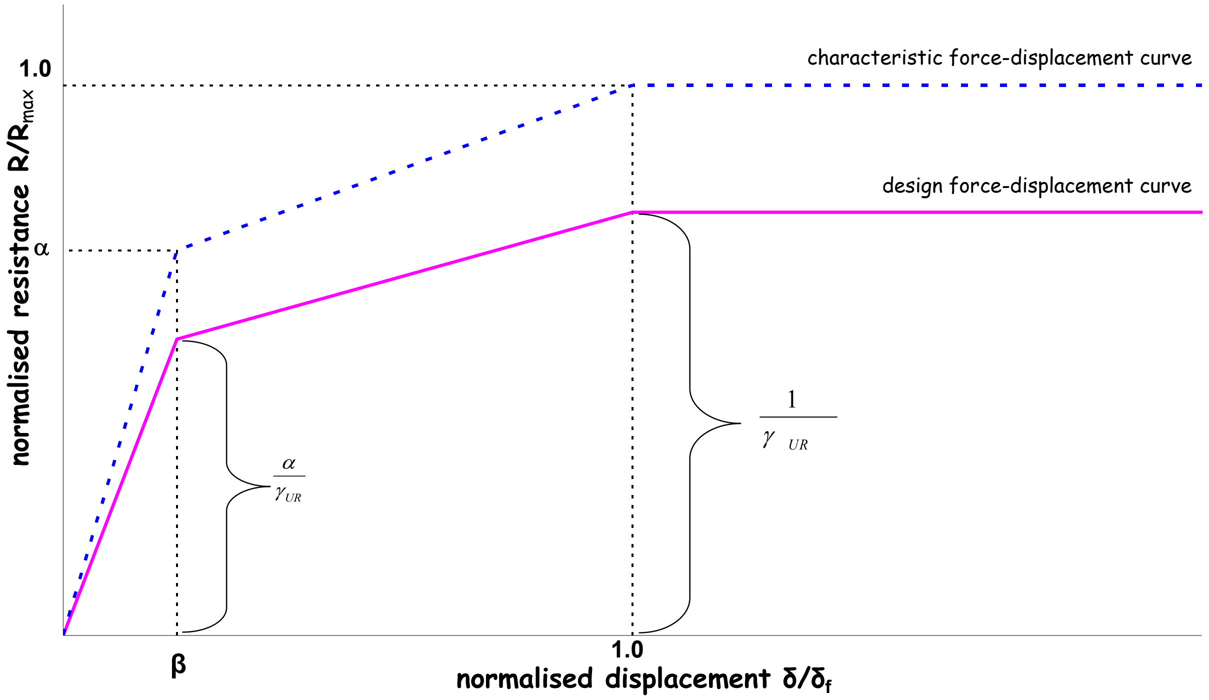

Figure 8-1
Interpretation of resistance factors, γ_{UR} in the pipe soil capacity second analyses of profile

γ_{UR} safety factor related to the vertical upward soil resistance

$$\boldsymbol{\gamma}_{\mathbf{UR}} = 0.85 + 3 \cdot \boldsymbol{\sigma}_{configuration} \left[m^{-1}\right] \quad \text{Non-cohesive soil (“Sand and Rock”)} \tag{21}$$

$$\boldsymbol{\gamma}_{\mathbf{UR}} = 1.1 + 3 \cdot \boldsymbol{\sigma}_{configuration} \left[m^{-1}\right] \quad \text{Cohesive soil (“Clay”)} \tag{22}$$

Where $\sigma_{configuration}$ is the standard deviation for the survey accuracy of the pipeline configuration, given in meters. $\sigma_{configuration}$ shall not be taken less than 0.025m.

The pipe-soil interaction model shall be applied on the configuration as described in Sec.5.4.3.3.

Close to the end, the build up of axial force requires special attention, see 5.2.3.

8.3.3 Design criterion

8.3.3.1 Principle

The design philosophy is outlined as follows, see Figure 8-2.

— The uplift resistance in the load response model is represented by a lower bound characteristic value (From expected (E(R)) to lower bound (R_c))
— This uplift resistance is reduced by the safety factor γ_{UR}. This safety factor is again dependent on the configuration survey accuracy. The resulting uplift resistance curve is shown in Figure 8-1.
— A downward soil stiffness equal to a best estimate is applied (K_{BE}).
— All loads are applied in the model and the temperature is increased until failure occurs at $T(K_{BE})$ in the soil, Eq. (23).
— A corresponding failure temperature is calculated for a lower bound downward stiffness (K_{LB}), resulting in a temperature $T(K_{LB})$, Eq. (24).
— If $T(K_{LB})$ is close to $T(K_{BE})$, this implies that the pipeline will fail upwards. Failure upwards implies that it is limited by the cover uplift resistance and that it is located on the dashed line in Figure 8-2.
— If $T(K_{LB})$ is different from $T(K_{BE})$, this implies that the initial soil failure is downward eventually causing the pipeline upward penetration. Initial failure downwards implies that it is limited by the downward stiffness and that it is located on the solid, more horizontal lines in Figure 8-2.
— Calculate the design resistance equivalent failure temperature, T_{Rd}, Eq. (25). Failure definition is given in 5.4.1.
— Calculate the design load equivalent temperature, T_{Sd}, giving axial effective load factor of γ_{UF}, Eq. (26).
— Verify that the design load equivalent temperature is less than the design resistance equivalent failure temperature, Eq. (27), with the corresponding safety factors, Eq. (28).

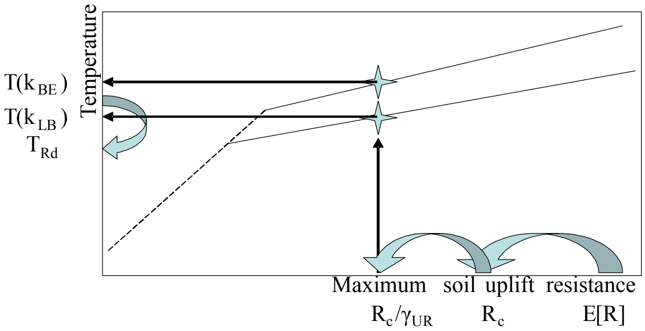

Figure 8-2
Illustration of the design principles in 2nd analyses

$$T(k_{BE}) = T_{failure}(p_{li}, R_c / \gamma_{UR}, k_{BE}) \tag{23}$$

$$T(k_{LB}) = T_{failure}(p_{li}, R_c / \gamma_{UR}, k_{LB}) \tag{24}$$

$$T_{Rd} = \frac{3 \cdot T(k_{LB}) - T(k_{BE})}{2} \tag{25}$$

$$\begin{aligned} T_{Sd} &= T_d \cdot \gamma_{UF} + T_p \cdot (\gamma_{UF} - 1) = \\ &= T_d \cdot \gamma_{UF} + \frac{[\Delta p_i \cdot A_i \cdot (1 - 2 \cdot \nu) - H]}{A_s \cdot E \cdot \alpha} \cdot (\gamma_{UF} - 1) \end{aligned} \tag{26}$$

$$T_{Sd} \leq T_{Rd} \tag{27}$$

where:

$$\boldsymbol{\gamma}_{\mathbf{UF}} = \begin{cases} \mathbf{1.00} & \textbf{for Safety Class Low} \\ \mathbf{1.15} & \textbf{for Safety Class Normal} \\ \mathbf{1.30} & \textbf{for safety Class High} \end{cases} \tag{28}$$

If the analyses fail to fulfil the criterion, the cover must be modified or the configuration changed (re-trenched) and the analyses re-performed.

8.3.3.2 Loads

Upheaval buckling failure given in this report is defined as an ultimate limit state (ULS) and extreme functional loads and associated other shall therefore be used. A conservative temperature profile shall be used.

The safety is ensured by a load effect factor on the effective axial load, γ_{UF}. In order to include the effective axial load contribution from the pressure, this is done by use of the design load equivalent temperature, T_{Sd}.

8.3.3.3 Simplified criterion in case downward stiffness is high

If lower bound downward soil stiffness is significantly higher than the upward initial stiffness the T_{Rd} can be simplified and determined as:

$$T_{Rd} = T(k_{BE}) \tag{29}$$

In case the above does not apply, T_{Rd} can be estimated based upon the following simplified procedure.

8.3.3.4 Applied to a configuration

The determination of T_{Rd} should in theory be applied, and vary, along the whole pipeline. The following simplified approach is recommended to avoid this.

For a range of prop shape configurations the required soil resistance/soil cover height (including safety factors) shall be estimated based upon the best estimate downward stiffness, k_{BE}.i.e. such that:

$$T(k_{BE}) \approx T_{Sd} \tag{30}$$

The height of the imperfections shall represent the out-of-straightness in the pipeline. A minimum of three different prop shape heights must be considered.

Based upon the soil resistance/cover height found above, the failure temperature $T(K_{BE})$ shall be estimated with the lower bound downward soil stiffness k_{LB}. (including safety factors).

For each prop shape, the ratio between the temperatures at failure for best estimate and lower bound downward stiffness shall be calculated:

$$r_i = \frac{T_i(k_{BE})}{T_i(k_{LB})} \tag{31}$$

Where i indicates prop imperfection no i.

If the deviation between temperature at failure for the best estimated and lower bound downward stiffness is within 5%, i.e. $\max(r_i) < 1.05$ mean (r_i) the FE analyses on the actual seabed profile may be analysed using the best estimate downward soil stiffness. If the deviation exceeds 5% modification in the safety factor shall be applied according to Eq. (32), where $\gamma_{UF,OLD}$ is given in Eq. (28).

$$\gamma_{UF} = \gamma_{UF,OLD}\left[\frac{T(k_{BE})}{T_{Rd}} - 0.05\right] \tag{32}$$

T_{Rd} in the formula above is calculated according to Eq. (25) using $T(k_{BE})$ and $T(k_{LB})$ from the case with highest r_i.

8.4 Step 2: Minimum Cover Design

Minimum soil uplift resistance/cover height H_{min} shall be determined for a prop shape configuration. The prop shape is the configuration for a pipe resting on an imperfection.

A prop shape can be determined and analysed using a FE-model of the pipeline lowering the pipeline on one single contact point with a distance δ_f above the "sea bed". The height of the prop shape shall be taken as.

$\delta_f = \sigma_{configuration}$ – not to be taken less than 0.025m (33)

Where $\sigma_{configuration}$ is one standard deviation of the configuration survey accuracy

The minimum cover design follows the same principles as for the specific cover design but with $\sigma_{configuration}$ equal to zero in Eq. (21) and (22).

8.5 Step 3: Specification of Cover

8.5.1 General

The determined cover height, H, (for sand and rock) or soil resistance, R, (for clay), as determined in Sec.8.3 and 8.4 gives the following required cover:

$$H(Kp) \geq Max\left[H_{spec}(Kp), H_{\min}(Kp)\right] \quad \text{Non-cohesive soil ("Sand \& Rock")} \tag{34}$$

$$R(Kp) \geq Max\left[R_{spec}(Kp), R_{\min}(Kp)\right] \quad \text{Cohesive soil ("Clay")} \tag{35}$$

This implies that additional margin may be added to account for uncertainties when documenting this height during construction.

For no-cohesive soils, this will normally imply that the measured height will be

$$H_{Survey} \geq H + 2\sigma_{cover} \tag{36}$$

where σ_{cover} is the standard deviation on the measurement accuracy related to survey measuring the cover height.

This also applies for cohesive soil when this is complemented by additional sand/rock dump.

Further, for cohesive soil, the required time to consolidate the soil resistance shall be determined.

8.5.2 Two or more independent surveys

A survey will be associated with errors and inaccuracies depending on the survey tool, environmental conditions, and obstacles on seabed among others. The present Recommended Practice assumes that a single survey represents the average profile with a upper and lower bound represented by a factor on the survey accuracy. This is a pragmatic approach to avoid artificial modification to the measured profile such a smoothening or definition of local imperfections.

In case of more than one independent surveys are performed an increased confidence in the estimated cover height is obtained. Each survey of the pipeline configuration can be seen upon as an average survey. Required specific cover, $H_1(Kp)$ and $H_2(Kp)$ to $H_n(Kp)$ can be estimated for each of the surveys, but now calculated with reduced safety factors. Each of these cover heights shall independently be verified according to Sec.8.3.

The general soil resistance/cover height based upon n surveys can now be taken as the average of the different surveys,

$$\overline{H}(Kp) = \tfrac{1}{n}\left[H_1(Kp) + H_2(Kp) + \ldots + H_n(Kp)\right] \tag{37}$$

Non-cohesive soil ("Sand & Rock")

$$\overline{R}(Kp) = \tfrac{1}{n}\left[R_1(Kp) + R_2(Kp) + \ldots + R_n(Kp)\right] \tag{38}$$

Cohesive soil ("Clay")

When calculating the average required cover height, engineering judgement has to be taken into account. E. g. maximum required soil resistance may not be at the exact same locations on two surveys.

The safety factors will for a combination of n independent surveys become:

$$\gamma_{UR} = 0.85 + \frac{3 \cdot \sigma_{configuration}}{\sqrt{n}}\left[m^{-1}\right] \quad \text{Non-cohesive soil ("Sand \& Rock")} \tag{39}$$

$$\gamma_{UR} = 1.1 + \frac{3 \cdot \sigma_{configuration}}{\sqrt{n}}\left[m^{-1}\right] \quad \text{Cohesive soil ("Clay")} \tag{40}$$

The same T_{Rd} applies for all n surveys of the pipeline.

The minimum soil resistance/cover height can be calculated for an imperfection of:

$$\delta_f = \frac{\sigma_{Configuration}}{\sqrt{n}} \tag{41}$$

The final cover resistance/cover height for n surveys shall be taken as the maximum of the specific and the minimum cover height.

$$H(Kp) \geq Max\left[\overline{H}(Kp), H_{\min}(Kp)\right] \quad \text{Non-cohesive soil ("Sand \& Rock")} \tag{42}$$

$$R(Kp) \geq Max\left[\overline{R}(Kp), R_{\min}(Kp)\right] \quad \text{Cohesive soil ("Clay")} \tag{43}$$

8.6 Step 4: Pipe Integrity Check

Table 8-1 shows the normally governing pipe integrity checks for a buried pipe.

Table 8-1 Required and Normally most governing pipe integrity check

	Pressure Cont.	*Local buckling*		*Axial*	*Ratcheting*	*Fatigue*	*Trawl interf.*	*Free span*
		Load Controlled	Displacement Controlled					
Reference	DNV-OS-F101	DNV-OS-F101	DNV-OS-F101	Eq. (65)	DNV-OS-F101	DNV-OS-F101	DNV-RP-F111	DNV-RP-F105
Buried pipe	X		X	X	X	X	-	-

In addition to the above limit states, Pipeline walking has to be considered, see Sec.6.4.5.

9. Condition Load Effect Factor For Exposed Pipelines

9.1 Basic Principles

This section includes calculation procedures to calculate the condition safety factor, γ_c for pipelines that buckles. The procedures apply to scenarios for even seabed, un-even seabed, with and without trawl. Depending on the scenario one or more of the parameters may be zero.

The condition factor, γ_c, is based on the prevailing uncertainty in the response bending moment given by:

$$\gamma_c(p,T,F_T) = \max\left[0.80; 0.72\cdot\left(1+2\cdot CoV\left(X_F(p,T,F_T)\right)\right)\right] \tag{44}$$

Note that a γ_c less than unity calculated in this section shall not be applied to the effective axial load in this Recommended Practice.

$CoV(X_F(p,T,F_c))$ is the Coefficient of Variation of the resulting bending moment in the buckle based on characteristic pressure(p), temperature(T) and trawl load (F_T) stated in the criterion. The uncertainty in the bending moment response from the global FE-analyses is assumed to arise from:

— uncertainty in the axial soil resistance, X_A
— uncertainty in the lateral soil resistance, X_L
— uncertainty in the applied stress-strain curve, X_B
— uncertainty in the applied trawl load, X_C (for annual trawl frequency larger than 10^{-4} only).

The uncertainty in the bending moment response may be estimated from:

$$CoV(X_F(p,T,F_T)) = \sqrt{\begin{aligned}&CoV(X_A(p,T,F_T))^2 + CoV(X_L(p,T,F_T))^2 + CoV(X_B(p,T,F_T))^2 + \\ &CoV(X_C(p,T,F_T))^2\end{aligned}} \tag{45}$$

The terms $CoV(X_A)$, $CoV(X_L)$, $CoV(X_B)$ and $CoV(X_C)$ reflects the impact on the resulting bending moment response uncertainty from the uncertainty in the soil parameters, choice of stress-strain curve and uncertainty in the applied trawl pull-over load respectively. The calculation of the condition factor will then also represent the degree of displacement control that the pipeline experience.

In addition, a model uncertainty may be present. $CoV(X_F)$ is not to be taken less than 5%. This is accounted for by the lower bound value 0.80.

9.2 Calculation of Cov(X_a) Axial Soil Resistance

The required set of non-linear FE-analyse used to establish $CoV(X_A)$ in the expression for the condition factor γ_c is indicated in Figure 9-1. This corresponds to the soil matrix in Eq. (46). The triggering mechanism shall be established assuming an imperfection triggering the highest capacity combinations of lateral resistance. The moment responses M_1->M_2 and M_{BE} shall be taken at the final equilibrium state.

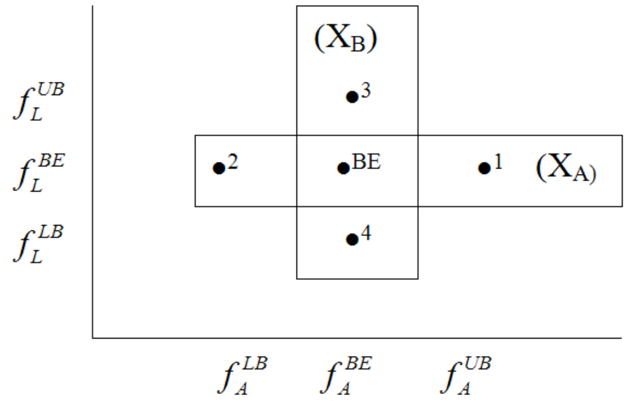

Figure 9-1
Required soil property combinations to be assessed for pipelines likely to buckle globally

The moment responses corresponding to soil matrix becomes:

$$\begin{aligned}&M\left(p;T;f_L^{BE};f_A^{BE};f_y;F\right)\Rightarrow M_{BE}\\ &M\left(p;T;f_L^{BE};f_A^{UB};f_y;F_T\right)\Rightarrow M_1\\ &M\left(p;T;f_L^{BE};f_A^{LB};f_y;F_T\right)\Rightarrow M_2\end{aligned} \tag{46}$$

Where again indices *UB* and *LB* indicate upper and lower bound resistance and f_y indicate analyses with a stress-strain curve defined from specified minimum values f_y and f_u.

The resulting uncertainty contribution from the uncertainty in the soil models become:

$$CoV\left(X_A(p;T;F_T)\right) = \frac{1}{M_{BE}}\frac{|M_2 - M_1|}{n_A} \tag{47}$$

Where n_A accounts for the distance between upper and lower bound values in terms of number of standard deviations, i.e.:

$$n_A = \frac{f_A^{UB} - f_A^{LB}}{f_A^{BE}\cdot CoV(f_A)} \tag{48}$$

If the Upper and Lower bound values are specified as mean value +/- 2 standard deviation (in accordance with normal interpretation), $n_A = 4$ applies.

Note that soil resistance coefficients with upper indices *LB*, *BE*, *UB* represents the complete soil (force-deformation) model rather than a single value. The main objective is to define the deviation of the lower and upper bound models from the best estimate model in terms of standard deviations.

9.3 Calculation of CoV(X_L) Lateral Soil Friction

The required set of non-linear FE-analyse used to establish $CoV(X_L)$ in the expression for the condition factor γ_c is indicated in Figure 9-1. This corresponds to the soil matrix in Eq. (49). The moment responses M3->M4 and M_{BE} shall be taken at the final equilibrium state.

The moment responses corresponding to soil matrix becomes:

$$\begin{aligned} &M\left(p;T;f_L^{UB};f_A^{BE};f_y;F_T\right) \Rightarrow M_3 \\ &M\left(p;T;f_L^{LB};f_A^{BE};f_y;F_T\right) \Rightarrow M_4 \end{aligned} \tag{49}$$

Where again indices *UB* and *LB* indicate upper and lower bound value and f_y indicate analyses with a stress-strain curve defined from specified minimum values f_y and f_u. A total of 2 additional FE-analyses is required.

The resulting uncertainty contribution from the uncertainty in the soil models become:

$$CoV\left(X_L(p;T;F_T)\right) = \frac{1}{M_{BE}} \frac{|M_4 - M_3|}{n_L} \tag{50}$$

Where n_L accounts for the distance between upper and lower bound values in terms of number of standard deviations, i.e.:

$$n_L = \frac{f_L^{UB} - f_L^{LB}}{f_L^{BE} \cdot CoV(f_L)} \tag{51}$$

9.4 Calculation of COV(X_B) Stress-Strain

Uncertainties in the load effect calculations $CoV(X_B)$, which are related to the resulting moment-strain curve from geometry and material properties are assessed from the bases case and one additional response (FE) analyses M5.

$$M\left(p;T;\mu_L^{BE};\mu_A^{BE};\sigma_y;F_T\right) \Rightarrow M_5 \tag{52}$$

Where σ_y indicates analyses with stress-strain curve defined from mean values of yield and ultimate stress. $CoV(X_B)$ is given as follows:

$$CoV\left(X_B(p;T;F_T)\right) = \left|\frac{M_5 - M_{BC}\cdot\lambda}{M_5 + M_{BC}\cdot\lambda}\right| \cdot \frac{2}{n_y} \tag{53}$$

Where n_y is the number of standard deviation between the mean yield strength and minimum specified yield strength, typically 2.

λ is the ratio between the moment capacity using σ_y corresponding to M_5 and moment capacity using f_y corresponding to M_{BE}. This ensures consistency between global response analyses and capacity. λ is given by:

$$\lambda = \sqrt{\frac{4\left(\frac{\sigma_y}{f_y}\right)^2 - 3\left(\frac{q_h}{\alpha_c}\right)^2}{4 - 3\left(\frac{q_h}{\alpha_c}\right)^2}} \quad \text{(Based on DNV-OS-F101:2000)}$$

$$\lambda = \sqrt{\frac{4\left(\frac{\sigma_y}{f_y}\right)^2 - 3\left(\alpha_p \cdot q_h\right)^2}{4 - 3\left(\alpha_p \cdot q_h\right)^2}} \quad \text{(Based on DNV-OS-F101:2007)} \tag{54}$$

The ratio q_h is the normalised pressure utilisation and α_c is the strain hardening factor defined in DNV-OS-F101. q_h and α_c are related to f_y.

$CoV(X_B)$ = 5% may be used as maximum value.

9.5 Calculation of CoV(X_c) Trawl Pull-Over

In case the buckled section is likely to be exposed to trawl pull-over loads, the uncertainty from the trawl must be accounted for by performing 2 additional analyses:

$$\begin{aligned} &M\left(p,T,\mu_L^{BE};\mu_A^{BE};f_y;F_T^{UB}\right) \Rightarrow M_6 \\ &M\left(p,T,\mu_L^{BE};\mu_A^{BE};f_y;F_T^{LB}\right) \Rightarrow M_7 \end{aligned} \tag{55}$$

Where *UB* and *LB* indicate upper and lower bound value for the trawl load applied in the apex of the buckled section represented by M_{BE}.

The moment responses M_6 and M_7 shall be taken as the transient (if largest) maximum values. This is normally a conservative approach.

The resulting uncertainty becomes:

$$CoV\left(X_c(p,T,F_T)\right) = \frac{1}{M_F}\left(\frac{(M_7 - M_6)}{n_F}\right) \tag{56}$$

Where n_F accounts for the distance between upper and lower bound values in terms of number of standard deviations, i.e.

$$n_F = \frac{F_T^{UB} - F_T^{LB}}{F_T^{BE} CoV(F_T)} \tag{57}$$

The trawl load is defined in Sec.3. n_F is typically to be taken between 2 and 4.

Note:

The intention of the trawl sensitivity study is to include this effect on the overall uncertainty on the resulting moment. Since the global buckling moment is mostly displacement controlled, the load controlled trawl moment will not be “added “but to a large extent “replacing” the functional moment form global buckling. If the contribution from the trawl is dominating the uncertainty, special evaluations are required in order to determine a higher γ_c then resulting from the above procedure.

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9.6 Calculation of COV for Parameters with Large Variation and Non-symmetric Upper and Lower Bound

It is assumed that the resulting bending moment response can be described by a linear Taylor expansion as follows:

$$\mathrm{M} = \Sigma a_i x_i + a_0 \qquad \text{where} \quad a_i = \left.\frac{\partial \mathrm{M}}{\partial x_i}\right|_{E[x_i]} \tag{58}$$

Where x_i denote the basic parameters (axial and lateral soil friction, stress-strain curve, trawl load). a_0 is a constant and a_i is a Taylor expansion coefficient around the mean values/base case value for the parameter:

The standard deviation for M, σ_M becomes

$$\sigma_M^{\ 2} = \Sigma\Sigma a_i a_j \sigma_{x,i} \sigma_{x,j} \rho_{ij} \tag{59}$$

Where ρ_{ij} is the coefficient of correlation between parameter i and parameter φ and $\sigma_{x,i}$ is the standard deviation of parameter

x_i. For independent basic parameters (ρ_{ij} = 1) the Coefficient of Variation becomes:

$$\mathrm{CoV}(X_F) = \frac{\sigma_M}{E[M]} = \frac{\sqrt{\Sigma a_i^2 \sigma_{x,i}^2}}{E[M]} \tag{60}$$

The expansion coefficient a_i shall be established from parameters studies around the mean value/base case value. The recommended procedure is to establish a_i based on a 3 point polynomial approximation to $M(x_i)$. Using bending moment point values (M_1, $x_{i,1}$) and (M_2, $x_{i,2}$) symmetric around the mean value (M_{BC}, $x_{i,bc}$) the explicit expressions below appear.

$$\frac{a_i \sigma_{x,i}}{E[M]} = \frac{1}{E[M]} \frac{M_2 - M_1}{n} \quad \text{where} \quad n = \frac{x_{i,2} - x_{i,1}}{\sigma_{x,i}} \tag{61}$$

If the bending moment relationship $M(x_i)$ is known to be linear *$CoV(X_F)$* may be established by only 2 point. In that case

$$\frac{a_i \sigma_{x,i}}{E[M]} = \frac{1}{E[M]} \frac{M_{BC} - M_1}{n} \quad \text{where} \quad n = \frac{x_{i,BC} - x_{i,1}}{\sigma_{x,i}} \tag{62}$$

10. Pipe Integrity checks

10.1 General

This Recommended Practice is based on risk principles and limit state methodology in compliance with DNV-OS-F101.

An overview of the required different design criteria are given in the pipe integrity check section for each scenario.

10.2 Design Criteria Format

The load and resistance factor design format in this RP complies with the format adopted in DNV-OS-F101 with the exception of the characteristic pressure in the local buckling check. The format is, however, given here in the order to ease the use of this RP.

The adopted load and resistance factor format consist of a design load effect term, L_{Sd}, and a design resistance effect term, R_{Rd}. In general, the different limit states have the general format in Eq. (63)

$$f\left(\left(\frac{L_{Sd}}{R_{Rd}}\right)_1, \left(\frac{L_{Sd}}{R_{Rd}}\right)_2, \left(\frac{L_{Sd}}{R_{Rd}}\right)_3, ,\right) \le 1 \tag{63}$$

Both the design load effects as well as the design resistance are built of characteristic values and partial safety factors. These will be described in the following sub-sections followed by the specific limit state criteria.

The design load effect shall be calculated based on a combination of characteristic loads and load effect factors in line with Eq. (64).

Note:

The trawl interference load effect, L_I has been specified separately but with the same load effect factors and condition factors as for functional loads. This is in slight deviation from DNV-OS-F101:2000, where the trawl load effect was defined as an environmental load.

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$$L_{Sd} = L_F \cdot \gamma_F \cdot \gamma_C + L_I \cdot \gamma_F \cdot \gamma_C + L_E \cdot \gamma_E + L_A \cdot \gamma_A \cdot \gamma_C \tag{64}$$

10.3 Pipe integrity Limit State Criteria

10.3.1 Axial loading limit state

The compressive axial limit state may be governing in particular for design for buried pipes. They shall fulfil the following criterion:

$$\varepsilon_d \le \frac{\varepsilon_{cr}}{\gamma_{Axial}} \tag{65}$$

Where:

$$\varepsilon_{cr} \le \frac{4}{3} \cdot \sqrt{\frac{1}{n}} \cdot \frac{t}{D} \tag{66}$$

γ_{Axial} = 3.5
n = steel hardening in Ramberg-Osgood description below

$$\varepsilon = \frac{\sigma}{E}\left(1 + \frac{3}{7}\left(\frac{\sigma}{\sigma_r}\right)^{n-1}\right) \tag{67}$$

Where the capacity is taken from ref. /9/.

10.3.2 Displacement controlled condition combined with internal over pressure

This is in accordance with DNV-OS-F101:2007.

Note that in DNV-OS-F101:2000 the benefit of internal pressure is only allowed if this is the minimum pressure that can be maintained during operation. This is corrected in 2007 version.

11. Documentation for operation

A pipeline that is expected to move has particular challenges with respect to interpretation of survey and inspection results. The documentation shall be as per standard practice but with particular focus on the following:

— alignment sheet showing
— diameter
— thickness
— coating and coating properties
— free-spans
— as function of the KP (kilometerpost)
— submerged weight
— friction models applied including expected penetration
— temperature profile, both design and operating
— pressure and pressure profile
— design solution (how the buckling is expected to be triggered)
— intervention
— as built profiles and cover
— the documentation shall be given in such format that it can be used to
— update inspection planning
— evaluate inspection results.

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APPENDIX A
MITIGATION MEASURES FOR EXPOSED PIPELINES

A.1 General

If, the response from the applied loads exceeds the pipe cross-section capacity, mitigation measures have to be introduced.

Potential localisation is related to inhomogenities in the pipeline as-laid configuration (initial curvature), pipe-soil interacF-tion (friction factors) and temperature profile. Localisation leads to longer feed-in length to the largest buckles, and thereby increased potential for local buckling and fracture.

In this RP, the following mitigation measures have been considered:

— prevent development of buckling
— increase soil restraint
— reduction in driving force
— radical change in pipeline structure
— mitigate the development of additional bending
— intermittent rock dumping
— snaked laying, and
— pre-bend sections.

The selection of mitigation measure depends on the actual configuration achieved by the pipeline on the sea bottom, bending capacity of the pipe section and nature of soil-pipe interaction on the three directions (axial, vertical and lateral).

A.2 Prevent development of buckling

A.2.1 Increase soil restraint

The simplest and most straightforward way of stabilising a pipeline against snaking buckling is to bury it with sufficient cover material. This option is often difficult and expensive, and several alternative design strategies have been examined, ref. Guijt (1990).

Another alternative is to increase the submerged weight of the pipeline. It is not usually practicable to resolve the problem by an increase in submerged weight, because a substantial weight will normally be required.

A.2.2 Reduction of the driving force

The first and most obvious method is to reduce the design operating temperature and pressure. A reduction in operating temperature is generally impracticable, but could be accomplished by adding a heat exchanger to the system. Reducing the wall thickness of the line will have a similar effect. This reduces the temperature component of the effective axial force, which is proportional to the wall thickness (and is usually the largest component), and leaves the pressure component almost unchanged. A reduction in wall thickness can be achieved by increasing the grade of steel.

A second method in the first group is to increase the lay tension. Residual lay tension balances part of the compressive force induced by operation, and therefore reduces the resultant force. A difficulty is that the residual tension cannot be measured directly, but must be calculated from the lay conditions and its continued presence in the line depends on there being no lateral movements by lateral deflection, so entrapping a residual tension in the pipeline.

A third alternative is to preheat the line, and to allow it to move and relax compressive forces induced, ref. Craig (1990).

A.2.3 Radical change in the pipeline structure

One alternative is to replace one or more single lines by a closed bundle supported on spacers in a carrier, ref. Palmer (1974), or equivalently by a pipe-in-pipe system in which the internal flow line is supported in an outer pipe. The internal lines in the bundle might develop axial compressive forces in operation, but those forces can be balanced by tensile forces in the outer carrier, through end bulkheads and possibly intermediate bulkheads. The internal lines may bow laterally between the spacers, but the movements are controlled (higher system stiffness and lower feed-in) and the bending stresses may remain at an acceptable level. This effect can be further enhanced by making the internal bundle helical, ref. Duxbury (1989): this geometry increases the reduction in axial compressive force that accompanies outward movements of the internal lines.

A second radical alternative is to use flexible pipes. Flexible are subject to upheaval buckling, refs. Putot (1989) and (1990), primarily because of the pressure effect. The tendency to buckle in service can be reduced by laying or trenching under internal pressure, or by modifying the internal structure away from a “balanced” design, so as to produce a pipe that tends to contract axially when loaded by internal pressure.

A.3 Mitigate the development of unacceptable bending

A.3.1 Sharing of expansion into adjacent buckles

Stabilisation with intermittent rock dumping can be carried out with the following premises:

— to counteract up-lift at most pronounced of seabed crests undulations
— to increase the resistance to axial feed-in into sections subject to snaking.

In the first case, crushed rock is dumped on critical overbends. See Eq. (A.3). To identify these overbends confidently is a demanding survey task in deep water, or when the critical imperfection amplitude is small.

A method to improve cover efficiency is to place a geotextile or concrete mattress over the pipe before the rock is placed. However, the use of a geotextile in a subsea environment will require a comprehensive investigation of its long term stability against creep and structural deterioration.

In the second case the rock can be dumped in intermittent heaps, with the aim to increase the restraint to axial movements in order to reduce the feed-in into isolated buckles that may be triggered by imperfections or trawl gear interaction.

A.3.2 Snaked laying

Besides the mentioned mitigation measures a new laying procedure has been developed involving pre-snaking the pipe allowing it to expand on the seabed thus mitigating the occurrence of high compressive axial forces and hence the problems associated with buckling.

The possibility to relax the compressive axial force through a regular and controlled initiation of several buckles to prevent the development of excessive bending as a consequence of a localised lateral deflection has been investigated in different contexts. The common factor is that the pipeline should be installed with a number of predefined curves/imperfections (snaked laying) along the route in order to activate several buckles in a controlled way.

The pipeline as-laid configuration may be generated by:

— alternate lateral movements of the barge during laying, or
— pre-bent pipe sections.

Each imperfection can be represented by a short curve between two straight sections. The minimum achievable radius of the curved section depends on the pipe characteristics, residual lay pull and soil lateral capacity stabilising the curve.

The wavelength of the as-laid configuration is determined as a function of the allowable bending moment in the buckled section. The global buckling load is determined by the initial pipe curvature and lateral soil resistance force. The expected laying induced imperfections are such that localisation phenomena should be avoided especially when short pitch is expected. The lower the curve radius, the higher is the confidence that the buckle will initiate at each curve as desired and that the laying tolerance will not give rise to localisation. The objective of mitigation is to avoid localisation of buckles.

In a way the final result will resemble that of regularly spaced horizontal curves, except for the activation mechanism. The mechanism includes a first stage of vertical up-lift over the obstacle, hence turn over to snaking in the seabed plane.

The solution, when feasible, is a promising alternative to traditional mitigation measures (trenching, gravel dumping) where the intention is to activate a regular snaking of the pipeline according to a series of half-waves resulting in an infinite buckling mode in order to reduce the maximum bending moment in the buckled sections. According to this philosophy, each buckle should behave as a wide expansion loop involving a uniform, but limited, lateral deflection. In this way, the initial buckling load is released due to pipeline axial expansion into the loop involving a limited bending effect. However, the real deformation mode of the pipeline can be different from the one previously described as the expansion will tend to localise in one or more generated imperfection. This is a consequence of:

a) Wide arc length of the generated imperfection, consequence of the maximum curvature achievable during laying and required wave lengths to avoid localisation. The arc segment is in general several times larger than the buckling lengths corresponding to a localised buckling mode.
b) Variable profile of the pipe curvature, introduces a non-uniform distribution of the destabilising force along the arc length proportional to the available buckling load and pipe curvature.
c) Inhomogeneties in the as-laid configuration since real pipeline initial configuration may differ from the one assumed in the analytical model.
d) Inhomogenities in soil pipe interaction, especially as regard the soil lateral capacity to counteract the destabilising forces.

These items have to be taken into account in the design.

A.3.3 Single/Multiple buckle

At the design stage, it is difficult to anticipate whether lateral deflection would develop either as isolated buckles, the third or fourth modes described by Hobbs (1984), or as a sequence of interfering buckles each absorbing a distributed expansion (i.e. Hobbs' infinite mode). In particular, localisation of lateral deflection might take place, in the presence of large imperfections. Friedman (1989) and Putot (1989) emphasised this aspect for bar-buckling of hot pipelines and confirmed, both experimentally and practically, the transition from a pseudo-periodic mode of deformation (such as Hobbs' infinite mode) to a localised buckling pattern. On this basis, it is expected that bar buckling may occur localised, probably triggered by a particularly significant geometric imperfection.

However, one way of controlling the development of excessive bending moment is to share the feed-in between two or several buckles artificially generated during pipe lay along the route. The possibility to relax the compressive axial force through a regular and controlled initiation of several buckles at specific locations is a promising alternative to traditional mitigation measures (trenching, gravel dumping, etc.). However, it should be documented that sharing of feed-in will occur and that localization is a remote hazard.

The potential localization between two adjacent imperfections is a function of the mobilization loads of the two involved half waves and of the axial restraint provided by soil-pipe frictional forces between the two imperfections. See Eq. (A.2).

When the applied axial force in the pipeline reach the mobilization load (i.e. the axial load buckling capacity) for the most susceptible of the adjacent imperfections (the one with the maximum curvature and/or weaker lateral restraint and/or highest temperature), this starts to deflect laterally releasing the initial compression. As a result of the release of axial compression force, the two straight pipe sections positioned at the buckle shoulders start to feed-in to the buckled section. The anchoring length La of the sliding section is a function of the initial buckling force and frictional capacity:

$$L_a = \frac{Po - P}{\phi_a \cdot w_p} = \frac{\Delta P}{\phi_a \cdot w_p} \qquad \text{(A.1)}$$

When the pitch between the two imperfections is shorter than the required anchoring length and the other buckle is still stable, the sliding section involves this second buckle. At this point, the buckle with the larger mobilization load will not be triggered as its axial compressive force is limited by the expansion/deflection of the mobilized buckle. See Eq. (A.2).

One way of controlling the feed-in into a buckle is to share the feed-in between two or several buckles. To assume feed-in into one buckle is conservative and is normally the primary choice. If the feed-in is divided between several buckles, it should be documented that sharing of feed-in will occur.

When the applied axial force in the pipeline reach the mobilisation load (i.e. the axial load buckling capacity) for the most susceptible of the adjacent imperfections (the one with the maximum curvature and/or weaker lateral restraint and/or highest temperature), this starts to deflect laterally releasing the initial compression. As a result of the release of axial compression force, the pipe starts to feed-in to the buckled section. At the axial sliding section between the adjacent imperfections/curvatures, the axial force is build up due to the axial soil resistance.

In order to initiate buckling/lateral snaking at the adjacent imperfection(s), the build up of axial compression between the first buckled section and the adjacent imperfection(s) must be larger than the buckling mobilisation load at the adjacent imperfection(s). If build up of axial compression force is insufficient, i.e. due to too short distance between two imperfections and/or too low axial soil resistance force, the next buckle will not be triggered as its axial compressive force is limited by the expansion/deflection of the mobilised buckle. Hence, localisation will occur.

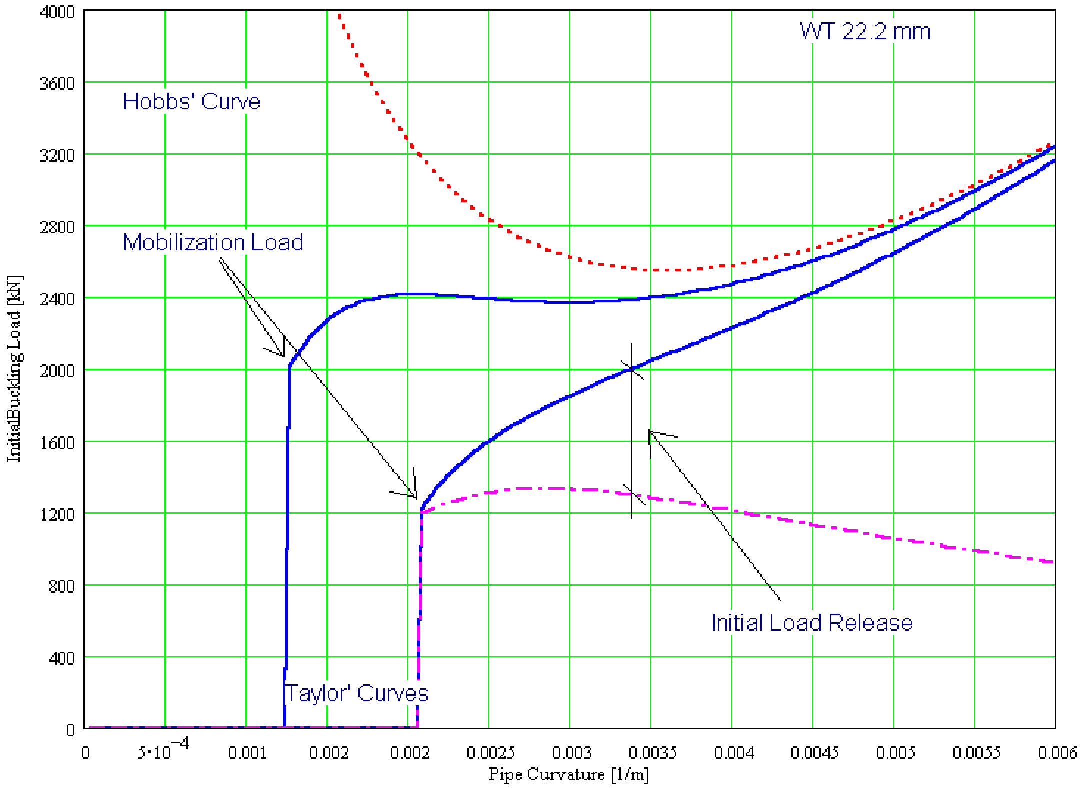

Figure A-1
Buckling load for adjacent buckles

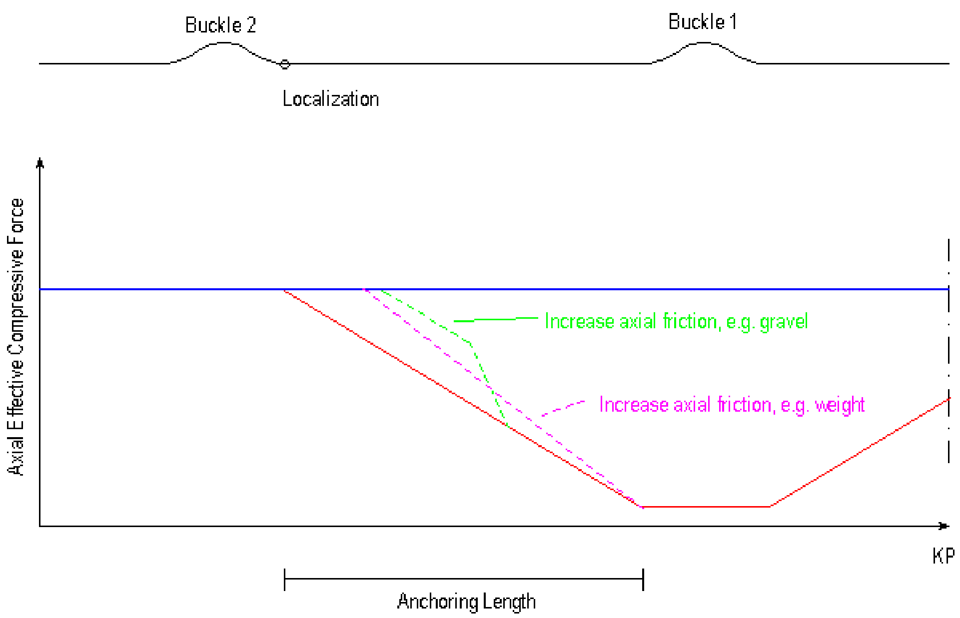

Figure A-2
Anchoring length for adjacent buckles

A.3.4 Prevention of unwanted uplift of pipeline

A pipeline on uneven seabed will tend to raise and lift off a crest in case a given combination of axial force and feed-in is present. This uplift can be a trigging mechanism for global buckling as described in chapter 7, but there may be several reasons why it is not acceptable or unwanted with upward movement of the pipe:

— restrains on the pipeline as end termination or in-line flanges are normally not designed for vertical or lateral movements
— unwanted interaction with other installations close to the pipeline
— uplift from pipe supports.

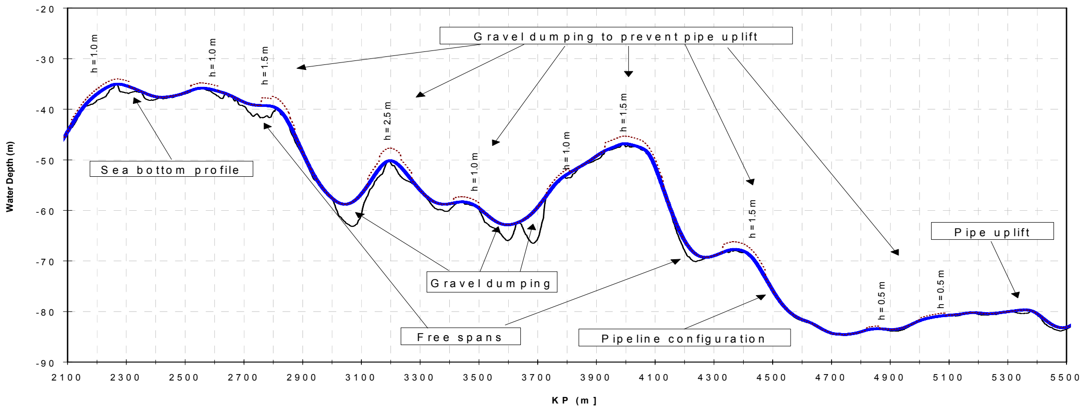

Figure A-3
Uplift restraints at crests

In the FE model used the uplift restrains shall be modelled with the appropriate force/displacement relation. Uplift forces on the restraints F_r shall be estimated for the worst combination of functional loads. The pipeline / soil interaction for lateral and axial resistance can be taken as best estimates.

The restrains preventing the uplift shall be designed according the following principal equation:

$$R_r \geq F_r \cdot \gamma \tag{A.2}$$

R_r uplift resistance
F_r uplift force
γ safety factor.

A rock berm on top of the pipeline is the common mitigation measure to prevent uplift of a pipeline. The resistance in a rock berm can be estimated according to equations in Chapter 5. The safety factor shall be taken as $\gamma = 2.0$.

A.4 References

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APPENDIX B
SOIL RESISTANCE FOR BURIED PIPELINES

B.1 General

B.1.1 Pipe-soil interaction

This appendix gives recommendations regarding the uplift resistance modelling for buried pipeline as well as for the associated uncertainties for buried pipelines. The uncertainty in the resistance is significant and must be considered in a design.

Description of soil models for both cohesive (clay) and cohesionless (sand and rock) soil are given.

The condition of the soil surrounding the pipe is the single most important aspect. This will to some extent be influenced by the trenching method. Secondly, the clearance depth to intact soil conditions below the pipe in the trench will add to the uncertainties in respect to the downward stiffness. A calculation model where this is included is discussed.

Effects from additional gravel are also discussed in the appendix, as commonly being introduced to increase the vertical resistance.

Special design issues like pipeline flotation and soil liquefaction is not addressed in this appendix.

B.1.2 Soil investigation

Unexpected and poorly defined ground conditions are generally the most commonly re-occurring causes of construction project delays and cost escalations. In many cases marine pipeline projects, including buried pipelines, will be particularly exposed to such risks due to their reliance on satisfactory seafloor and subsurface soil properties for trouble free installation and operation.

Reference /7/ give specific recommendations with respect to the a suitable programs for geotechnical investigations for marine pipelines. The information needed from the investigation will largely depend on the occurring soil conditions and specific pipeline challenges as they are revealed during development of the project. It may therefore be preferred to carry out the investigation in progressive stages.

The soil investigation program needs to consider both the intact soil conditions and the soil conditions following a trenching operation. The latter may require the construction of a certain length of dummy trench as part of a soil investigation.

B.2 Definitions

The definition of terms following is intended to serve as a quick reference for the relationships and formulae given in this document. More details about the respective terms are found in the relevant sections of the main text following immediately after this section.

	Local soil failure	Failure mode characterised by soil material displacement around the pipe without noticeable soil heave at the surface
	Global soil failure	Failure mode characterised by soil material displacement upwards leading to noticeable soil heave at the surface. This failure mode is also described as shallow wedge failure.
D	*Pipe Diameter*	Total outer diameter of the pipeline including coating.
R	*Uplift resistance*	The uplift resistance provided by soil (the submerged weight of the pipe is not included).
R_1	*Uplift resistance within layer 1*	The uplift resistance provided by the backfill material surrounding the pipe. (Notation used when required to separate the uplift resistance contributions from the individual layers in a two-layer system.)
R_2	*Uplift resistance within layer 2*	The uplift resistance in a second layer. (Notation used when required to separate the uplift resistance contributions from the individual layers in a two-layer system.)
R_V	*Downward resistance*	The downward resistance.
$R_{V,intact}$	*Downward resistance*	The downward resistance associated with penetration in intact material.
$R_{V,trench}$	*Downward resistance*	The downward resistance associated with penetration within trench material.
z_0	*Depth of disturbed zone*	Depth below pipe to undisturbed (intact) material.
H	*Cover height*	The total height of cover material measured from top of pipe.
H_1	*Height of backfill material (layer one)*	The height of a natural backfill material measured from top of pipe. (Notation only used when required to separate the uplift resistance in a two-layer system.)
H_2	*Height of additional backfill material (layer two)*	The height of additional backfill material measured as total cover height (H) less the initial height of the natural backfill material (H_1).
δ_f	*Failure displacement*	The displacement required to mobilise the uplift resistance
β	*Fraction of failure displacement* d_f*; parameter in load-resistance curve*	$\beta \cdot \delta_f$ is the displacement value of the knee between the first and second linear curve of the trilinear load-displacement curve for uplift resistance.
α	*Fraction of uplift capacity R; parameter in load-resistance curve*	$\alpha \cdot R$ is the mobilized uplift resistance at the knee between the first and second linear curve of the trilinear load-displacement curve for uplift resistance, i.e. $\alpha \cdot R$ is the resistance value which is mobilized at a displacement equal to $\beta \cdot \delta_f$.
s_u	*Intact strength*	The intact (static) undrained shear strength, which is the best measure of the *in situ* undisturbed (intact) soil strength. For pipeline uplift resistance the direct simple shear (DSS) strength or the unconsolidated undrained (UU) triaxial strength is assumed to be the most representative intact strength.
s_{ur}	*Remoulded shear strength*	The undrained shear strength measured by fallcone tests on remoulded clay.
k_{su}	*Shear strength gradient*	The increase in shear strength with depth.
S_t	*Soil sensitivity*	The ratio between s_u and s_{ur}, as determined by fall-cone or UU triaxial tests.

f	*Friction resistance factor*	The friction resistance factor is a factor, which is used to represent the frictional part of the uplift resistance, when the uplift resistance is assumed to consist of a frictional part plus the weight of the vertical soil column directly above the pipe.
f_R	*Post peak friction factor*	For coarse material with sharp edges or dense/densified material, the uplift resistance will drop after the failure displacement δ_f has been reached. This drop takes place without a significant change in remaining cover height and is thus due to a reduced available frictional resistance. The post peak friction factor f_R is smaller than the friction factor f and replaces f in the expression for R when the post peak resistance is to be calculated.
δ_R	*Post failure displacement*	The vertical uplift displacement at which the post peak reduction in the frictional resistance is fully developed, i.e. the drop in frictional resistance factor from f to f_R is completed. This is based on the assumption that there is no reduced cover height as the pipe displaces upwards.
f	*Drained friction angle*	The peak drained friction angle, i.e. the friction angle associated with the maximum resistance.
γ'	*Submerged unit weight*	Submerged unit weight of (intact) soil
γ_1'	*Submerged unit weight (layer one)*	Submerged unit weight of natural backfill material. (Notation used when it is required to separate the uplift resistance contributions between the individual layers in a two-layer system.)
γ_2'	*Submerged unit weight (layer two)*	Submerged unit weight of additional backfill material. (Notation used when it is required to separate the uplift resistance contributions between the individual layers in a two-layer system.)
N_c	*Bearing capacity coefficient*	The bearing capacity coefficient related to a local soil failure.
r	*Roughness factor*	Roughness factor for the pipe surface used for calculating and appropriate bearing capacity factor. For a smooth surface, $r = 0$.
c_v	*Coefficient of consolidation*	Parameter, derived from consolidation tests, and used for calculation of rate of consolidation (i.e. dissipation of excess pore pressure).
K_V	*Vertical downward stiffness*	The vertical downward stiffness associated with pipe penetration.
Q_V	*Bearing capacity of pipe*	The vertical resistance of the pipe associated with increased penetration
Nq	*Bearing capacity factor*	
$N\gamma$	*Bearing capacity factor*	
a	*Attraction*	
p_0'	*In-situ stresses*	Effective vertical in-situ soil stress at depth of centre of pipe

B.3 Cohesionless soils

B.3.1 General

The following describes our suggested approach to how to assess the uplift resistance and the downward stiffness for pipes on cohesionless soils.

B.3.2 Uplift resistance

The soil resistance is represented by a tri-linear model with increasing capacity up to a maximum value. The model is valid for the displacements up to mobilization of full uplift resistance, but is not meant to represent the entire force-displacement relation for a pipe that experiences large vertical upwards displacements and eventually penetrates the soil surface. For dense sand, a post peak behaviour has been included in the model in order to account for effects which are associated with a looser state of the soil at displacements beyond the failure displacement.

The total resistance (shear and weight) can be written in this format:

$$R = \gamma' \cdot H \cdot D + \gamma' \cdot D^2 \left(\frac{1}{2} - \frac{\pi}{8}\right) + K \cdot \tan(\phi) \cdot \gamma' \cdot \left(H + \frac{D}{2}\right)^2 \quad \text{(B.1)}$$

where

H = Cover height (depth to top of trench minus depth to top of pipe)
γ' = Submerged weight of soil
K = Lateral earth pressure coefficient also accounting for increase in vertical stress during uplift
ϕ = Angle of internal friction
D = Pipe diameter

Eq. (B.1) can be rewritten as:

$$R = \gamma' \cdot H \cdot D + \gamma' \cdot D^2 \left(\frac{1}{2} - \frac{\pi}{8}\right) + f \cdot \left(H + \frac{D}{2}\right)^2 \quad \text{(B.2)}$$

where

f = Uplift resistance factor

The key issue is to assess an adequate uplift coefficient f. The uplift coefficient f can be calculated from a drained (peak) friction angle and a lateral earth pressure coefficient. The uplift coefficient f can also be calibrated from model test results. If the drained friction angle is known for the soil which is used in the model tests, then this can be used to calculate the lateral earth pressure coefficient. We have selected to base our model on the drained (peak) friction angle, but it should be emphasised that the friction angle should be established at the relevant stress level. Relative density may be used for correlation to drained friction angle, however one need to consider the following aspects:

a) The drained friction angle of a backfill will be dependent on the relative density of the material. This relation may be found from laboratory tests, but one still has to guess on the actual true density for the trenched material.
b) The relative density of a trenched material may be established through Cone Penetration Test (CPT) measurements, but established relations between cone resistance and density generally have poor accuracy in the low stress region
c) Relative density is a poor measure for gravel and rock (not to be addressed further)

As a consequence, one needs to evaluate carefully any interaction between the uplift factor f and other relevant soil parameters, and a good choice would be to select ranges that the majority of the test results fall within and give several criteria as guidance for how to choose an adequate range in a design situation.

B.3.3 Model for uplift resistance and load-displacement curve

The uplift resistance R_{max} of a pipe in sand consists of two components, viz. a component owing to the weight of the soil above the pipe and a component owing to soil friction. The uplift resistance can alternatively be expressed as:

$$R_{\max} = (1 + f\frac{H_c}{D})(\gamma' \cdot H_c \cdot D) \tag{B.3}$$

where

H_c = depth from the soil surface to the center of the pipe
γ' = submerged unit weight of soil
f = uplift resistance factor
D = pipe diameter.

The factor f is also referred to as the frictional resistance factor, because it refers to the part of the uplift resistance which is due to soil friction.

The uplift resistance factor can be expressed as:

$$f = K \cdot \tan(\varphi) \tag{B.4}$$

where

φ = friction angle
K = coefficient of lateral earth pressure.

For pipes in loose sand, test results indicate that an "at rest" earth pressure model is appropriate for K, hence

$$f = K_0 \tan\varphi = (1 - \sin\varphi)\tan\varphi \tag{B.5}$$

For pipes in medium sand and dense sand, a model for K based on passive earth pressure theory provides the best fit to test data, hence

$$f = K_P \tan\varphi = \frac{\tan\varphi}{(\sqrt{1+\tan^2\varphi} - \tan\varphi\sqrt{1+r})^2} \tag{B.6}$$

in which r is a roughness parameter whose value is negative for the current application and possibly near –1.

For $\tan\varphi$, a normal distribution is assumed together with a COV = 0.10. For this input, the mean value of f is calculated by means of PROBAN for three different sands. Table 4-1 summarizes the basis for the calculations and the resulting mean values of f. Note that the roughness parameter r is chosen to approximately match test results reported by Trautmann et al. (1985). The calculations performed also provide results for the standard deviation of f, however, this standard deviation only reflects the variability in the friction angle φ, and in r where applicable. The true standard deviation is expected to be significantly higher, e.g., owing to model uncertainty which has not been included. The standard deviation will be dealt with below. Finally, the calculations indicate that the distribution of f is skewed significantly to the left.

Table B-1 Results for mean value of uplift resistance factor f

Sand type	*φ (°) for E[tanφ]*	*K model*	*Roughness r*	*E[f]*
Loose	30	K_0	N/A	0.29
Medium	35	K_P	-1.00	0.47
Dense	40	K_P	-0.97	0.62

Schaminée et al. (1990) reference the following lower bound for possible values of f,

$$f_{LB} = 1.4 \cdot (\tan\varphi - 0.5) \tag{B.7}$$

It is assessed that this expression may lead to too large values of the lower bound f_{LB} for φ in excess of approximately 40. A slightly simpler model for f_{LB} is proposed as follows

$$f_{LB} = \begin{cases} 0.1 & \text{for} \quad \varphi \le 30 \\ 0.1 + \dfrac{\varphi - 30}{30} & \text{for} \quad 30 < \varphi \le 45 \\ 0.6 & \text{for} \quad \varphi > 45 \end{cases} \tag{B.8}$$

in which the friction angle φ is to be specified in degrees. Note that with this model and the results in Table B-1, the following simple relationship can be established

$$E[f] = f_{LB} + 0.19 \quad \textit{for}\ 30 \le \varphi \le 45 \tag{B.9}$$

Observed values for f reported by Williams (1998) indicate that the standard deviation of f, independently of soil type, is

$$D[f] \approx 0.11$$

With respect to uplift, lower tail outcomes of f are unfavourable. Hence, it is particularly important that the lower tail of the distribution of f is modelled in a realistic or slightly conservative manner. The distribution of f has been found to be skewed to the left. With this in mind, it is suggested to represent the distribution of f by a uniform distribution with lower bound f_{LB} as given above and with upper bound $f_{UB} = f_{LB} + 0.38$. It appears right away that this distribution model will honour the data by yielding the right mean value. Furthermore, the uniform distribution model leads to the following standard deviation of f,

$$D[f] = \frac{f_{UB} - f_{LB}}{2\sqrt{3}} = \frac{0.38}{3.4} = 0.11 \tag{B.10}$$

which is also consistent with available data. Hence, based on the currently available knowledge about the uplift resistance factor, it seems reasonable to represent it by a uniform distribution model as suggested.

The uplift resistance R_{max} is assumed to be fully mobilized at a vertical uplift displacement δ_f, where δ_f is 0.005-0.010 times the height H. Note that δ_f seems to be independent of the ratio H/D.

The non-linear force-displacement response of a buried pipe is represented by a tri-linear curve as shown in Figure B-1.

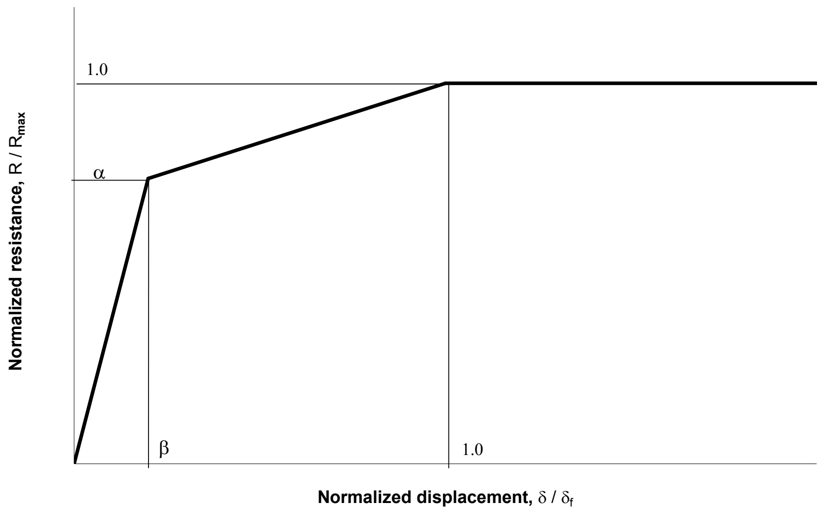


Figure B-1
Tri-linear force-displacement curve model for uplift

Data by Trautmann et al. (1985) supports $\beta = 0.2$ as a reasonable choice for the location of the first break in the force-displacement curve when the tri-linear model is adopted for this curve. With this choice for β, the same data leads to the following mean value and coefficient of variation for the ordinate value a of R/R_{max} at $\beta = 0.2$:

$$E[\alpha] = 0.78$$

$$COV[\alpha] = 0.10$$

B.3.4 Downward resistance

Figure B-2 shows a principal sketch of the downward resistance for a trenched and buried pipe. The pipe is normally lighter in operation than during installation (i.e. $W'_{installation} > W'_{operation}$). As a consequence the downward resistance $R(z)_{installation}$ is less than fully mobilised at time of operation (i.e. the soil is unloaded). Secondly, the resistance at the achieved depth from installation, z, is likely to increase at time of operation due to the denser state of the backfill material and in particular when addition material (e.g. gravel) is used of increase the cover height of the pipe (i.e. $R(z)_{operation} > R(z)_{installation}$). Within a pipe downward reaction $F_{analysis}$ less than $R(z)_{operation}$ and in particular less than $W'_{operation}$ a stiff response is to be expected. However, when vertical pipe downward reaction exceeds the resistance at the depth of installation, the stiffness will decrease significantly and will be described by the increase in bearing resistance with depth. An equivalent linear stiffness K_V will therefore be much dependent on the level of pipe vertical reaction leading to an iterative analysis, and the non-linear resistance vs. pipe displacement is therefore much preferred.

The vertical downward stiffness that the pipe will experience from the support when it penetrates the soil is defined as the contact force transferred between pipe and soil divided by the vertical displacement, i.e. the penetration depth. The contact force, which essentially is a penetration resistance, will be equal to the bearing capacity for the contact area that corresponds to a given penetration, since the penetration of the pipe implies a “continuous” failure situation in the supporting soil.

For a pipe on a cohesionless seabed, the stiffness for the initial penetration will be much governed by the increase in the pipe penetration resistance as the contact area of the pipe increases during the penetration. However, when approximately half the pipe diameter has become embedded in the soil, the increase in bearing capacity for further penetration will be less pronounced and merely governed by the increase in the vertical in-situ stresses with depth.

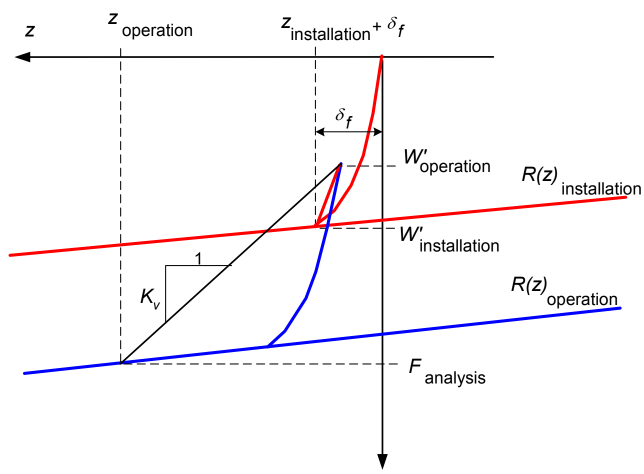


Figure B-2
Principal sketch showing downward resistance and equivalent linear downward stiffness

Generally the penetration resistance for a pipe gradually penetrating “intact soil conditions”, i.e. assuming the backfill soil only contribute to overburden pressure*, can be calculated as:

$$R_V(z) = \frac{1}{2} N_\gamma \cdot \gamma' \cdot B(z)^2 + N_q \cdot (p_0' + a) \cdot B(z) \qquad \text{(B.11)}$$

where

z	=	Pipe penetration in intact (undisturbed) material
$B(z)$	=	Contact width of pipe,
	=	$2 \cdot \sqrt{z \cdot (D - z)}$ for $z \le D/2$ and
p_o'	=	in-situ stresses for pipe embedment,
	=	$\gamma_2' \cdot H_2 + \gamma_1' \cdot (H_1 + D)$ for H_1 and $H_2 > 0$
a	=	attraction of soil (zero for backfill material)
N_q	=	Bearing capacity coefficient for soil below pipe
N_γ	=	Bearing capacity coefficient for soil below pipe

* This simplification neglects the effect of the looser backfill soil in transmitting pressure from the pipe to the underlying harder soil and will therefore tend to underestimate the resistance

However, in case there is a certain clearance distance down to the intact material or it can be assumed that the strength of the back-fill material is equal to that of the intact soil*, i.e. "homogeneous conditions" can be assumed, then the penetration resistance should be calculated assuming a constant bearing area:

* This simplification neglects the presence of the harder soil in prohibiting failure surfaces to develop in accordance with the assumed bearing capacity formula solely within the loose soil and will therefore tend to underestimate the resistance

$$R_V(z) = \frac{1}{2} N_\gamma \cdot \gamma' \cdot D^2 + N_q \cdot (p_0' + a) \cdot D \qquad \text{(B.12)}$$

where

z = Pipe penetration in homogeneous soil
p_o' = in-situ stresses for pipe embedment,
= $\gamma_2' \cdot H_2 + \gamma_1' \cdot (H_1 + D/2 + z)$ for $z > 0$, H_1 and $H_2 > 0$
a = attraction of soil (zero for backfill material)
N_q = Bearing capacity coefficient for soil surrounding pipe
N_γ = Bearing capacity coefficient for soil surrounding pipe

Eqs. (B.11) and (B.12) are valid when the possible cover material with a total height of $H_1 + H_2$ extends laterally beyond the extent of the assumed bearing capacity surface through the original intact soil.

Although we strongly recommend to use the non-linear soil resistance directly in the analysis, we will in the further describe how an vertical downward stiffness may be established. The vertical downward stiffness should generally reflect both the achieved penetration z within the trench during installation as well the deformation required to mobilise the bearing capacity at this penetration depth. A secant stiffness can be defined as follows when the deformation required to mobilise the bearing capacity is disregarded as being small compared to the penetration:

$$K_V = \frac{R_V(z)}{z} \qquad \text{(B.13)}$$

with

$\gamma_1' = \gamma_2' = \gamma'$ and $H = H_1 + H_2$

the following expression gives the secant stiffness associated with gradual pipe penetration into the intact soil at the bottom of the trench.

$$K_V = 2 \cdot N_\gamma \cdot \gamma' \cdot (D - z) + 2 \cdot N_q \cdot [\gamma' \cdot (H + D) + a] \cdot \sqrt{\left(\frac{D}{z} - 1\right)} \qquad \text{(B.14)}$$

valid for

$H \geq 0$

$z \leq D/2$

Should the pipe after installation have a clearance down to intact soil conditions so that looser material is present beneath the pipe, the secant stiffness in this case will be governed by downward displacement within the zone of clearance. This can be described by the following equation:

$$K_V = \frac{D}{z} \cdot \left[0.5 \cdot N_\gamma \cdot \gamma' \cdot D + N_q \cdot \left[\gamma' \cdot \left(H + \frac{D}{2} + z \right) + a \right] \right] \qquad \text{(B.15)}$$

valid for

$H \geq 0$

$z > 0$

The two formulae above neglect the deformation required to mobilise the downward resistance. For small penetrations z this deformation may not be negligible compared to z, and the expressions for the secant stiffness may therefore lead to over-estimation of the secant stiffness for such small penetration values. In cases where the secant stiffness is needed for such small penetrations, a representative secant stiffness should therefore be calculated as

$$K_V = \frac{R_V(z)}{z + \delta f} \qquad \text{(B.16)}$$

in which δz should be taken as minimum 0.05 times the contact width B. In cases where the analysis is sensitive to the secant stiffness, it should be considered to use the full non-linear load-displacement curve, i.e. $Q_V(z)$ vs. $z_{total} = z + \delta f$, rather than just a secant stiffness for some value of z.

It follows from the above that the secant stiffness is clearly dependent on the cover height of the pipe, the pipe diameter and the magnitude of the actual pipe penetration.

B.3.5 Modelling recommendations

Within the constraints of a tri-linear soil resistance mobilisation curve (see Figure B-1), we have ended up by suggesting the following range:

Table B-2 Defined ranges for the soil parameters

Characteristics	*Parameter*		*Range*
Loose sand	f	$\in$	[0.1, 0.3]
	δ_f	$\in$	[0.5%, 0.8%]·H
	α	$\in$	[0.75, 0.85]
	β	=	0.2
Medium/Dense	f	$\in$	[0.4, 0.6]
	δ_f	$\in$	[0.5%, 0.8%]·H
	α	$\in$	[0.65, 0.75]
	β	=	0.2
Post peak resistance	f_r	=	$\alpha_f \cdot f$
at displacement	α_f	=	[0.65, 0.75]
	δ_{fr}	=	$3 \cdot d_f$
Rock	f	$\in$	[0.5 , 0.8]
	δ_f	$\in$	[20 , 30]mm*)
	α	$\in$	0.7 · R
	β	=	0.2

* Based on a limited number of tests no clear tendency with respect to cover height was revealed. The range given is representative for gravel / crushed rock material and cover heights vs. diameter ratio less than approximately 4.

with the following limitations on soil cover ratios:

Loose sand: $3.5 \leq \frac{H}{OD} \leq 7.5$

Medium / Dense sand: $2.0 \leq \frac{H}{OD} \leq 8.0$

Rock: $2.0 \leq \frac{H}{OD} \leq 8.0$

Particle sizes:

Rock: $25mm \leq d_{eqv} \leq 75mm$

Based on the expressions established in B.3.4, simplified expressions for the secant vertical stiffness have been derived and expressed as functions of the pipe diameter and the cover-height-to-diameter ratio. The simplified expressions are given in Table B-3. They are only intended to be used to obtain typical values or initial estimates of the vertical stiffness, and they are valid for H/D > -0.5, i.e. more than half the pipe is embedded.

Table B-3 Typical values for vertical stiffness valid for embedded pipes for a limited range of cover heights (H/D > -0.5)		
Soil type	*"Intact conditions"* *Static vertical stiffness K_V (kN/m/m)*	*"Homogeneous conditions"* *Static vertical stiffness K_V (kN/m/m)*
Loose sand	$D\cdot(750+670\cdot\frac{H}{D})$	$D\cdot(960+1500\cdot\frac{H}{D})$
Medium	$D\cdot(1700+1450\cdot\frac{H}{D})$	$D\cdot(2850+3300\cdot\frac{H}{D})$
Dense sand	$D\cdot(5300+4300\cdot\frac{H}{D})$	$D\cdot(11800+9800\cdot\frac{H}{D})$ *)
* Dense sand combined with the "homogeneous condition" model will not be justified for natural backfill material, since the various installation methods are likely to leave the backfill material in a loose to medium dense condition.		

In cases where the analysis is sensitive to the secant stiffness, it should be considered to use the full non-linear load-displacement curve as established Eqs. (B.11) or (B.12), and calculate the stiffness for a relevant penetration.

B.3.6 Probabilistic model for vertical spring stiffness for pipe on sand

The static vertical spring stiffness K_S for a horizontal pipe resting on sand is defined as

$$K_S = \frac{q}{z} \tag{B.17}$$

in which q is the line load transferred between the pipe and the supporting soil, and z is the associated penetration of the pipe, measured from the original soil surface to the bottom of the pipe. The line load q is predicted by bearing capacity theory.

For a pipe of diameter D resting on a sand with friction angle φ and submerged unit weight γ', the following formulae apply:

$$K_S = \gamma' B(N_q + N_\gamma\sqrt{\frac{D}{z}-1}) \tag{B.18}$$

in which

$$N_q = \exp(\pi\tan\varphi)\tan^2(\frac{\pi}{4}+\frac{\varphi}{2}) \tag{B.19}$$

$$N_\gamma = 1.5(N_q-1)\tan\varphi \tag{B.20}$$

$$B = 2\sqrt{(D-z)z} \tag{B.21}$$

These formulae are valid for a pipe resting on the original soil surface. In the case of a covered pipe in a trench, the original soil surface is to be interpreted as the bottom of the trench, i.e. the interface between the trench and intact soil material. However, any effects of external overburden at the level of this interface, such as the vertical pressure from the original soil adjacent to the trench, are disregarded.

For the three sand types "loose", "medium", and "dense", mean values for tanφ and γ' are chosen in the middle of the intervals that characterize the respective soil types, and a coefficient of variation of 10% is chosen for tanφ as well as for γ'. The mean values for tanφ and γ' are given in Table B-4. A possible model uncertainty inn the model for K_S is ignored.

Table B-4 Mean values of soil properties		
Sand type	*E[γ'] (kN/m³)*	*E[tanφ]*
Loose	9.75	tan(29°) = 0.554
Medium	10.75	tan(33°) = 0.649
Dense	11.75	tan(38.5°) = 0.795

In the following, it is assumed that the penetration is $z = D/2$, because this will lead to smaller and thus more conservative stiffness values than other, shallower penetrations. For this penetration assumption, the above formulae imply that the static vertical spring stiffness K_S is proportional to the pipe diameter D.

For the penetration assumption $z = D/2$, the following characteristics for the static vertical spring stiffness K_S are calculated by Mote-Carlo simulation:

— mean value $E[K_S]$
— standard deviation $D[K_S]$
— coefficient of variation COV
— skewness α_3
— kurtosis α_4.

Calculations are performed for either of the three soil types.

Assuming that one has sufficient knowledge to be able to classify a particular soil as either loose, medium or dense, a lower bound $K_{S,LB}$ for the stiffness is calculated based on values for tanφ and γ chosen equal to the lower bounds of the intervals that characterize the respective soil types. The lower bound values for tanφ and γ' are given in Table B-5.

Table B-5 Lower bound values of soil properties		
Sand type	*E[γ'] (kN/m³)*	*E[tanφ]*
Loose	8.5	tan(28°) = 0.532
Medium	9.0	tan(30°) = 0.577
Dense	10.0	tan(36°) = 0.727

The results of the calculations are summarised in Table B-6. In addition to the results given in Table B-5, plots of the distribution of K_S reveal that it may be well represented by a Gumbel distribution with mean value and standard deviation as given in Table B-6.

Table B-6 Static vertical stiffness K_S						
Sand type	*Lower bound $K_{S,LB}$ (kN/m/m)*	*Mean value $E[K_S]$ (kN/m/m)*	*Standard deviation $D[K_S]$ (kN/m/m)*	*Coefficient of variation COV (%)*	*Skewness α_3*	*Kurtosis α_4*
Loose	219D	300D	103D	34	1.0	4.9
Medium	301D	577D	225D	39	1.2	5.4
Dense	780D	1455D	669D	46	1.4	6.5

Based on the findings from the above study, it is suggested to represent the static vertical stiffness K_S by a Gumbel distribution with proposed values for the mean value and the coefficient of variation as given in Table B-7.

Table B-7 Proposed distribution parameters for Gumbel distributed static stiffness

Sand type	*Mean value $E[K_S]$ (kN/m/m)*	*Coefficient of variation COV (%)*
Loose	300D	35
Medium	600D	40
Dense	1500D	45

The results presented in Table B-6 and Table B-7 are based on analyses which assumes independence between $\tan\varphi$ and γ'. Further work to refine these results is recommended to include an investigation of the effects of a possible correlation between $\tan\varphi$ and γ'.

B.4 Cohesive soils

B.4.1 General

The following describes our suggested approach to how to assess the uplift resistance and the downward stiffness for pipes on clay. It appears to be relevant to differentiate slightly between the two main trenching methods, viz. ploughing and jetting. The effect of consolidation is considered to be an important issue, in particular when the trench is jetted.

A brief compilation of the available uplift resistance models for clay is included with presentation of the various involved terms. The uplift resistance models are also presented together in normalised charts for easy interpretation. The expressions presented for the uplift resistance do not include the submerged weight of the pipe.

There are mainly two different failure modes that govern the development of the uplift resistance. These modes consist of a local soil failure where the soil above the pipe is displaced around and beneath the pipe, and a global soil failure where a wedge extending to the soil surface is lifted together with the pipe. The local soil failure will be a simple function of the shear strength at the depth of the pipe, whereas the global soil failure implies a combination of weight and shear resistance. The shear resistance may either be a drained resistance or an undrained resistance. When drained resistance is prevailing, the model is equal to that used for cohesionless cover. When undrained resistance is prevailing, the undrained shear strength of the trench material is used.

B.4.2 Effect of trenching method

B.4.2.1 Jetting

A water/clay suspension is expected to prevail in the trench immediately after trenching. The pipe will be completely surrounded by this material when it is installed in the trench, and the water/clay suspension will gradually settle. Minor penetration of the pipe into the intact material at the bottom of the trench may also be expected. The shear strength of the clay surrounding the pipe will gradually increase from practically zero to that of a normally consolidated clay, depending on the coefficient of consolidation and the thickness of the clay layer. Following the consolidation process the thickness of the clay layer will decrease.

In process of time, the clay in the trench will regain shear strength. The regained shear strength is eventually expected to reach a constant level.

The upward displacements required to reach the maximum uplift resistance are also likely to be affected by the method of trenching. Jetting may introduce water filled voids in the soil in the trench, but generally the soil in the trench will form a homogeneous material.

B.4.2.2 Ploughing

When the pipe is placed in a trench formed by a ploughing device, the water content of the clay will not increase relative to that of the intact material. Thus the remoulded resistance of the clay as established through sensitivity measurements is likely to represent an expected minimum strength.

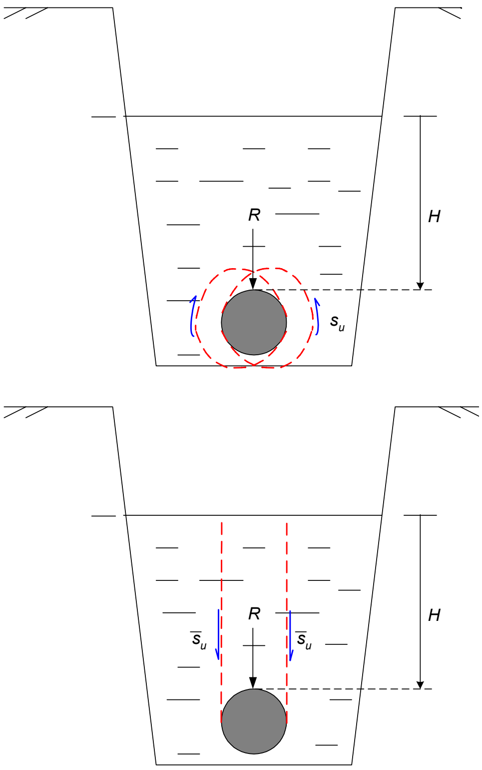
The regained shear strength with time is eventually expected to reach a strength which is proportional to the effective stresses according to theory for normally consolidated clays, or equal to the remoulded shear strength, whichever is the greater.

Ploughing is expected to change the macro structure of the clay by introducing cracks and water filled voids.

B.4.2.3 Conclusion

As a conclusion to the above we suggest to not rely on any upheaval resistance in a jetted cohesive soil shortly/immediately after installation. The upheaval resistance at time of operation need to rely on the consolidation process.

B.4.3 Uplift resistance



Local failure mode
(soil above pipe will displace around and below the pipe when the pipe moves upwards)

Global failure mode
(soil above pipe will displace upwards when the pipe moves upwards)

Figure B-3
Principal sketch of the two failure modes in clay

B.4.3.1 Local soil failure model

The uplift resistance is described by the following equation:

$$R = N_c \cdot \eta \cdot s_u \cdot D \qquad \text{(B.22)}$$

where

N_c = theoretical bearing capacity coefficient
η = empirical factor based on field tests. (range 0.55-0.80).
s_u = undrained shear strength at centre of pipe
D = pipe diameter.

The bearing capacity factor N_c for a deep failure is taken from /2/ being a function of the roughness of the pipe surface. For pipe embedment depths less than approximately 4.5 · D, the pipe is in the 'shallow' failure zone, where the maximum value of N_c, which is achievable when the pipe embedment is larger, cannot be counted on. Various expressions exist for the depth factor d_c, which accounts for the change in N_c within the 'shallow' failure zone. An expression for $N_c = (N_c)_{shallow}$, which accounts for the depth effect within the 'shallow' failure zone, is given in Eq. (B.23)

$$(N_c)_{shallow} = 2 \cdot \pi \cdot \left[1 + 1/3 \cdot \arctan\left(\frac{(H + D/2)}{D}\right) \cdot (1+r)\right] \qquad \frac{(H + D/2)}{D} \leq 4.5 \qquad \text{(B.23)}$$

in which r denotes the roughness factor for the pipe surface. For a smooth surface r = 0, for a rough surface r = 1. The lower value for the bearing capacity coefficient is geometrical deduced. The maximum bearing capacity is taken according from /2/.

The relationship in Eq. (B.23) is shown in Figure B-4.

The theoretical value for the bearing capacity factor will be approximately 9 when accounting for a smooth surface of the pipe, and approximately 12 for a rough surface of the pipe.

Tests performed in remoulded clay at a depth where the maximum bearing capacity factor can be counted on have given considerably lower resistance than the value calculated from this theoretical approach. The difference is believed to be due to rate effects (i.e. the strength of the material may be set too high compared to the rate of loading used in the tests), viscous effects and a progressive failure (believed to be less important in remoulded material). To account for these effects an empirical reduction factor, η, is introduced with a range between 0.55 and 0.8, with a best estimate value of 0.65.

Recent in-house experience from full-scale tests in remoulded material suggests that the maximum resistance offered by a local soil failure in soft clay will require small displacements to become mobilised, i.e. $\delta_f/D = 1 - 3\%$*. Small-scale models tend to give higher values, e.g. 7 – 8% according to Schaminée et al. (1990). We are also aware of small-scale tests where values in excess of 15% are presented for mobilisation of uplift resistance in fluidised clay.

* Displacements following API RP2a for laterally loaded piles in clay will be 1 – 5% for 50% resistance and 10 – 40% for maximum resistance. For e50 in the range of 0.5 – 2%.

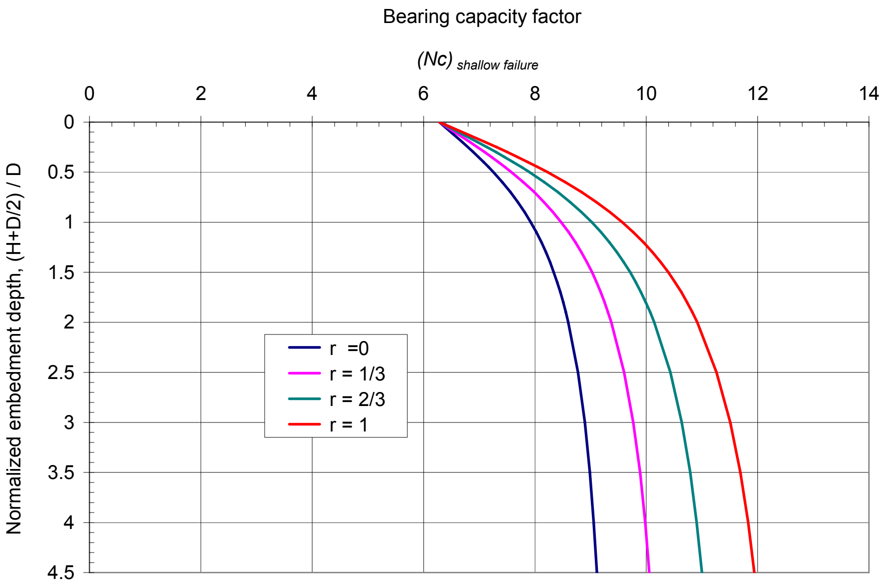


Figure B-4
N_c in shallow failure zone versus normalised depth (H+D/2)/D.

B.4.3.2 Global soil failure model (shallow shear failure)

The uplift resistance, based on global soil failure extending to the soil surface, is described by the following equation:

$$R = \gamma' \cdot H \cdot D + \gamma' \cdot D^2 (\frac{1}{2} - \frac{\pi}{8}) + 2 \cdot \bar{s}_u (H + \frac{D}{2}) \qquad (B.24)$$

where

H = cover height (depth to top of trench minus depth to top of pipe), ref. {hold}

γ' = submerged weight of soil

$\bar{s}_u$ = average undrained shear strength at from centre of pipe to top of trench

D = pipe diameter.

The average shear strength at the failure surface needs to reflect the strength of the trenched material. This will be dependent on the trenching method used.

With a truly undrained behaviour, the pipe cannot displace upwards (leaving a void) without generating a suction beneath (disregarded in Eq. (B.24)). The uplift force is expected to build up rather slowly, such that pipe displacements without suction beneath the pipe is possible, and for which a drained resistance model would actually be more appropriate. Once a 'threshold' resistance is exceeded the rate of displacement is likely to increase.

The tests that are carried out have shown δ_f/H ratios of 7-8% for remoulded clay, 1-6% for intact clay and 20-40% for intact clay lumps.

For small embedments of the pipe, the maximum resistance will be limited by the resistance associated with global soil failure, i.e. the failure surface extends up to the seabed. This resistance may be represented by the vertical slip model, which deviates from the local soil failure model by including a term dependent on the weight of the soil column above the pipe in the expression for the resistance.

B.4.3.3 Drained resistance model

The drained resistance model will be identical to Eq. (B.1) described in B.3.2.

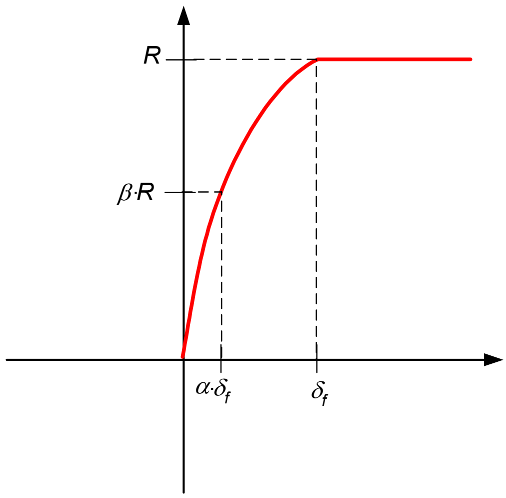


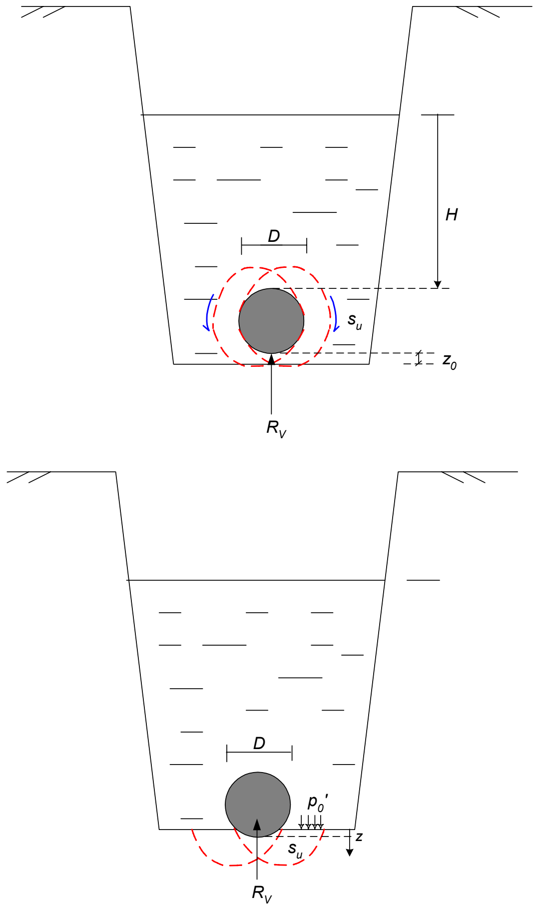
Figure B-5
Principal vertical force – displacement curve for a pipe in a trench with clay

B.4.4 Downward resistance

The principal for the downward resistance in cohesive soil is similar to the one described for cohesionless material (see Figure B-2) and the recommendation of considering the full the non-linear resistance vs. pipe displacement is therefore preferred rather than an equivalent downward stiffness.

The downward resistance in clay is governed by the local soil failure mode and undrained resistance. When calculating the vertical resistance one need to consider that the pipe may not be lying directly on bottom of the ideal trench, but that a certain clearance down to intact soil conditions, z_0. This clearance may be linked to the trenching method, soil conditions and stiffness of pipe. In the following we will take into consideration that there may be such a clearance without going further into the effects that have impact on this clearance.

When the pipe reach the boundary for intact soil conditions, the resistance is likely to increase. This can be calculated assuming penetration in intact clay with an increasing width of the contact area. When half the pipe diameter is penetrated into the intact material the contact width is constant and equal to the pipe diameter. The increase in strength from this point on will be governed by the shear strength gradient, ks_u.



Local soil failure
Resistance governed by the strength of the trench material.

Local soil failure
Resistance governed by the strength of the intact material.

Figure B-6
Principal sketch of the failure mode associated with vertical (downwards) resistance in clay

The vertical resistance within the trench material is described by the following equation:

$$R_{V,trench} = N_c \cdot \eta \cdot s_u \cdot D \qquad \text{(B.25)}$$

where

N_c = theoretical bearing capacity coefficient, see Eq. (B.23).
η = empirical factor. (Field tests on uplift resistance suggest a range from 0.55 to 0.80).
s_u = average re-consolidated shear strength for the entire failure surface, or conservatively, the re-consolidated undrained shear strength at centre of pipe
D = pipe diameter.

The vertical resistance for penetration in intact soil conditions is described by:

$$R_{V,\text{int}act} = (N_c \cdot s_u + p_o') \cdot B(z) \qquad z < D/2 \qquad \text{(B.26)}$$

where

N_c = theoretical bearing capacity coefficient, see Eq. (B.23)
Note! substitute (H+D/2)/2 with z/D to find correct N_c value.
s_u = undrained shear strength at intact soil boundary
$B(z)$ = contact width
=
= $B(z) = 2 \cdot \sqrt{z \cdot (D - z)}$
p_o' = effective overburden pressure
= $\gamma' \cdot (H + z_0 + D)$
γ' = submerged unit weight of trench material.

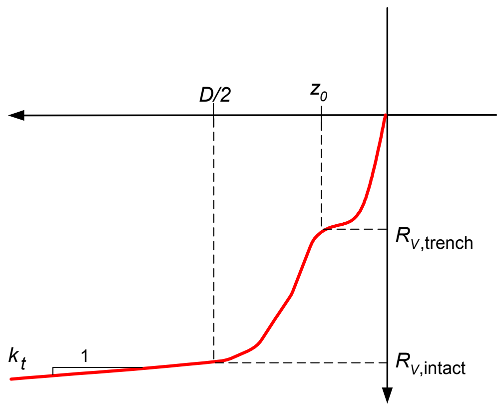


Figure B-7
Principal vertical resistance – downward displacement curve for a pipe in a trench with clay

The resistance within the trench, $R_{V,trench}$, should be calculated using local soil failure mode according to Eq. (B.25). For this failure mode an average shear strength for the entire failure surface should be considered. As a conservative alternative, the shear strength at centre of pipe may be used. Failure displacements, δ_f, for the local soil failure mode should not be taken less than 10% of the pipe diameter or limited to the depth of clearance to intact soil conditions.

The resistance associated with gradual pipe penetration into intact clay, $R_{V,intact}$ should be calculated according to Eq. (B.26). A possible initial penetration may be considered following the required contact area for the submerged weight of the pipe. The failure displacements, δ_f, may therefore be considered to be equal to the difference between initial penetration in intact clay and half the pipe diameter but not less than 10% of the pipe diameter. The penetration required to mobilise 50% of the resistance is typically 15-20% of the failure displacements.

The vertical downward stiffness should generally reflect both the achieved penetration z within the trench during installation as well the deformation required to mobilise the bearing capacity at this penetration depth. A secant stiffness can be defined as follows when the deformation required to mobilise the bearing capacity is disregarded as being small compared to the penetration:

$$K_V = \frac{R_V(z)}{z} \tag{B.27}$$

the following expression (derived at from Eq. (B.26))gives the secant stiffness associated with gradual pipe penetration into the intact soil at the bottom of the trench.

$$K_V = N_c \cdot 2 \cdot \sqrt{\left(\frac{D}{z} - 1\right)} \cdot \left[\left(s_{u,0} + k_{su} \cdot z\right) + 2 \cdot \gamma' \cdot \left(H + z + D\right)\right] \tag{B.28}$$

where

$s_{u,0}$= in-situ shear strength intercept at depth of trench.
k_{su} = shear strength gradient from depth of trench

Should the pipe after installation have a clearance down to intact soil conditions so that looser material is present beneath the pipe, the secant stiffness in this case will be governed by downward displacement within the zone of clearance. This can be described by the following equation (derived at from Eq. (B.25)).

$$K_V = \frac{N_c \cdot \eta \cdot s_{u,0} \cdot D}{z} + N_c \cdot \eta \cdot k_{su} \cdot D \tag{B.29}$$

where

$s_{u,0}$ = re-consolidated shear strength at centre of pipe

k_{su} = re-consolidated shear strength gradient from centre of pipe.

The two formulae above neglect the deformation required to mobilise the downward resistance. For small penetrations z this deformation may not be negligible compared to z, and the expressions for the secant stiffness may therefore lead to overestimation of the secant stiffness for such small penetration values. In cases where the secant stiffness is needed for such small penetrations, a representative secant stiffness should therefore be calculated as

$$K_V = \frac{R_V(z)}{z + \delta f} \tag{B.30}$$

in which δz should be taken as minimum 0.10 times the contact width B. In cases where the analysis is sensitive to the secant stiffness, it should be considered to use the full non-linear load-displacement curve, i.e. $R_V(z)$ vs. $z_{total} = z + \delta f$, rather than just a secant stiffness for some value of z.

It follows from the above that the secant stiffness is clearly dependent on both the shear strength and the pipe diameter.

B.4.5 Resistance model in cohesion soil with additional gravel

Additional gravel may be used to increase the uplift resistance. However, the mobilisation of the resistance is largely effected by the presence of clay in the trench. Recent in-house experience suggest that the pipe will only mobilise the resistance associated with a local failure mode as described by Eq. (B.22) as long as the pipe is below the layer boundary between clay and gravel. The added gravel may penetrate into the clay layer and decreasing the height from the pipe to the layer boundary, but experience suggest that this mixed zone of gravel and clay is to be considered as clay rather than gravel.

When more than 50% of the pipe is visible prior to gravel dumping, then the models established for gravel / rock can be used without modifications. When the pipe is covered with clay (either clay to flush with pipe or a certain cover height as can be seen in Figure B-8) then a softer response need to be considered, as well as the consolidated strength of the trench material.

The maximum resistance within the gravel is expressed by a traditional vertical slip model, with a shear resistance as for rock material and the cover height taken equal to the height of gravel.

The movements introduced to mobilise the frictional resistance in the gravel, may largely effect the uplift force. An alternative use of the gravel is therefore to rely on the strength increase in the clay following the weight of the added gravel and the increase in effective stresses as the excess pore water pressure dissipates. This process is called consolidation and is further elaborated in B.4.6.

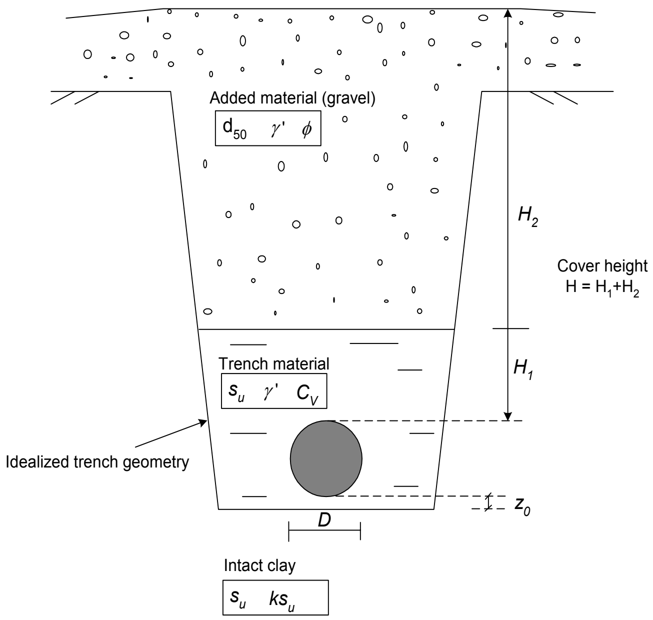


Figure B-8
Principal sketch showing of pipe in trench with natural backfill material and additional cover of gravel.

Figure B-9 shows the principal of the soil resistance vs. pipe displacement for a pipe in a trench with clay and additional cover. The interface between clay and additional cover material is expected to be influenced by the cover material in two ways:

1) The cover material will penetrate into the clay.
2) The clay will be squeezed up in the cover material.

Based on limited number of tests we suggest to start the mobilisation of cover material resistance at a displacement equal to H_1-z_p where z_p may be considered as the penetration of the cover material into the clay and equal to the medium particle size of the cover material d_{50}.

The effect of clay being squeezed up into the cover material will influence the consolidation time as the height up to a draining layer will increase. Considering a volume compatibility for the material being squeezed up into the cover material and the volume of pores in the cover material, one will typically find that the drainage height need to be increased with a factor of 2-3 times the medium grain size d_{50}. This additional height to a “free surface” with respect to drainage neglect the amount of clay that may fall into the trench during the placement of the additional cover.

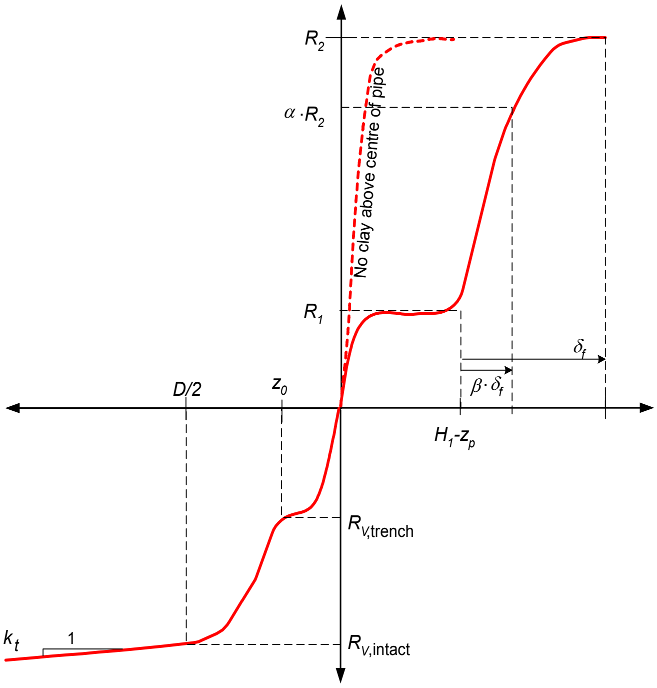


Figure B-9
Principal soil resistance – pipe displacement curve for a pipe in a trench with clay and additional cover material

B.4.6 The time factor

Consolidation is the process where water is dissipated from a soil and pore pressures gradually transferred to increased effective stresses in the soil. This process takes place following the application of a load or, as is the case for a soil in a trench, following the trenching. The effect of consolidation is particularly interesting when the trench is established with a jetting device for which the material inside the trench has close to zero strength just after trenching. In order to establish the applicable uplift resistance, one needs to know the time from trenching to operation of the pipe. The time it takes to complete consolidation is described by the consolidation coefficient C_v with dimension m^2/year. The time to complete consolidation is proportional with the squared length of the drainage path. For a pipe in a trench, the length of the drainage path can be taken as the distance from the centre of the pipe to the top of the trench. For engineering purposes, a ‘compaction front’ model can be established /1/ which can be used to reasonably well assess which effective trench height with final submerged soil unit weight one can count on above the bottom of the trench at any intermediate time after the trenching.

$$H' = \sqrt{K_\gamma \cdot C_v \cdot t} \qquad for \quad H' \leq H_t \tag{B.31}$$

The model is approximate and predicts that consolidation is completed after a time:

$$t_c = \frac{H_t^{\,2}}{K_\gamma \cdot C_V} \tag{B.32}$$

where

H_t = total height of fluidised material in the trench (i.e. drainage height)
H' = effective height of consolidated material in the trench at given time, t
C_v = coefficient of consolidation
t = time
K_γ = factor accounting for actual (triangular) strain distribution, $K_\gamma = 2$ is suggested for consolidation for weight of fluidised material.

The critical issue is to establish the ‘field’ value of the coefficient of consolidation for use in this ‘compaction front’ model. The coefficient of consolidation is generally lower for remoulded clay than for intact clay and will also be lower when water is added to the clay as in case of a jetting process.

It is believed that a value of $C_v = 0.5 – 1.0\ m^2$/year is fairly representative for remoulded soft clay.

A further limitation in the effect from consolidation need to be considered for deep and narrow trenches due to the shear resistance along the trench wall will transfer the consolidation stresses to the intact surrounding clay to each side of the trench rather than the clay surrounding the pipe. A limitation to the consolidation stresses surrounding the pipe can be expressed as a ratio between width and height of the trench and soil parameters for drained resistance (centre of pipe considered as representative depth for consolidated strength of the material surrounding the pipe):

$$H_{concol} = \left[1 - K_0 \cdot \tan(\varphi) \cdot \left(\frac{H_t - D/2}{B}\right)\right] \cdot H' \geq 0 \qquad for\ \ K_0 \cdot \sigma'_v \cdot \tan(\varphi) < \alpha \cdot s_u \tag{B.33}$$

where

K_0 = earth pressure coefficient for soil at rest $K_0 = 1\text{-}sin$(f)
H_t = total height of fluidised material in the trench (i.e. drainage height)
H' = effective height of consolidated material in the trench at given time, t
f’ = friction angle of intact clay along the trench wall
s_u = average shear strength of intact clay along the trench wall
B = width of trench (steep trench wedges considered)
D = pipe diameter
H_{consol} = effective height for consolidation stresses (i.e. $\sigma'_{consol} = \gamma' \cdot H_{consol}$)

The effect of this can be seen in Figure B-10 where the increase effective consolidation height is presented for some selected trench geometry.

When consolidation of the trench material is due to additional cover material (e.g. gravel of rock), then one may assume that the cover material occupies an area much larger than the trench, thus the reduction in effective trench height due to frictional forces towards the wedge of the trench may be neglected. In this case Eq. (B.31) should be used for effective consolidation height directly, however with the factor accounting for strain distribution with depth set equal to $K_\gamma = 1.0$ and the discussion with respect to increased height to free draining layer (see B.4.5).

In order to account for the loss of height of fill material following the dissipation of water from the fluidised material, one need to consider a loss in the order to 10% compared to the measured height immediately after the trenching operation.

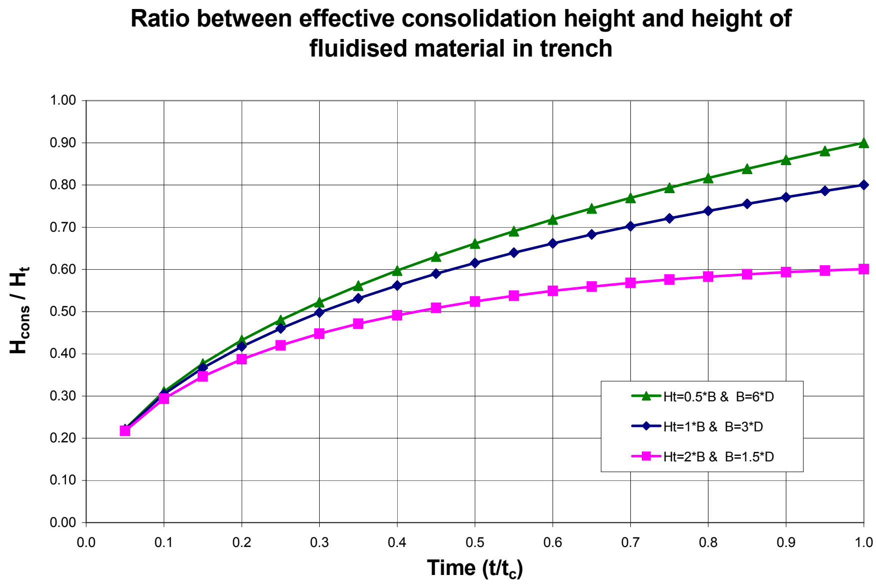


Figure B-10
Case study showing development of effective consolidation height for some trench geometries (B = Width of trench, D = pipe diameter)

B.4.7 Drained vs. Undrained resistance

It may be worth to bear in mind that the vertical slip model for prediction of uplift resistance is not a theoretically "correct" model, but only a practical way of splitting the uplift resistance into a weight contribution and a shear contribution. Even if the model does not describe the physics in a theoretically correct manner, it is possible to calibrate the parameters of the model to test results.

Bruton et al. (1998) describe the uplift resistance model used in the Poseidon Project /1/ and suggest that effective stress analysis should be used for very soft clay. They have interpreted effective friction angles of about 36 degrees for weak clays under low confining pressures, based on a calibration of the vertical slip model. The tests underlying this result are described as being very slow drained direct shear tests for 100mm square boxes. Interestingly enough, the interpretation does not include any cohesion of the material, which probably explains why the effective friction angle fall from 36 to 31 degrees when the vertical stress is doubled.

The nature of the uplift force with respect to duration and rate of increase is not fully known, but it is believed that the increase from normal operation temperature to design temperature may take place within a time period between the time required for a completely drained resistance (days) and the time normally associated with failure in typical laboratory tests (2 hours). Although for the Poseidon project the clay resistance was concluded to be insensitive to the rate of loading, this is not necessarily the general behaviour. It may well be that an undrained creep* model will form just as relevant a theoretical approach to capture the true nature of uplift resistance in soft clay as a drained resistance model.

* Reduction in strength following slow rate of loading.

For less plastic clays for which the remoulded strength of the material may not capture a lower limit of the resistance for low confining stresses, a drained resistance model should be used in order to establish an upper limit of the uplift resistance.

B.4.8 Modelling recommendations

Table B-8 defines the range of soil parameters and the recommended formulae for uplift resistance in a trench with clay. Key parameters defining the mobilisation of the resistance can be seen in Figure B-3.

Table B-8 Defined ranges of soil parameters and recommended formulae for uplift resistance (R_1) in trench with clay

Trenched by jetting	η	$\in$	[0.55 , 0.80]
	N_c	=	According to Eq. (B.23)
	s_u / σ'_v	$\in$	[0.22 , 0.26]
	γ'	$\in$	[4.0, 8.0] kN/m^3
	δ_f	$\in$	[0.03 · D , 0.07 · D] m
	α	=	0.5
	β	=	0.2
	H_{cons}	=	According to Eq. (B.33)
	R_{local}	=	$\eta \cdot N_c \cdot (\frac{s_u}{\sigma_v'} \cdot \gamma' \cdot H_{cons}) \cdot D$
	R_{global}	=	$\gamma' \cdot H_{cons} \cdot (D + \frac{s_u}{\sigma_v'} \cdot H_{cons})$
	R	=	$min(R_{local}, R_{global})$
Trenched by ploughing	η	$\in$	[0.55 , 0.80]
	N_c	=	According to Eq. (B.23)
	$\bar{s}_{ur}$	$\in$	[2, 20] kN/m^2
	γ'	$\in$	[5.5, 8.5] kN/m^3
	δ_f	$\in$	[0.2 · D, 0.4 · D] m
	α	=	0.5
	β	=	0.2
	f	$\in$	[0.25 , 0.40]
	R_{local}		$\eta \cdot N_c \cdot s_{ur} \cdot D$
	R_{global}		$\gamma' \cdot H \cdot D + 2 \cdot \bar{s}_{ur} \cdot (H + \frac{D}{2})$
	$R_{drained}$		$\gamma' \cdot H \cdot D + f \cdot \gamma' \cdot (H + \frac{D}{2})^2$
	R		$min(R_{local}, R_{global}, R_{drained})$

All formulae are valid for 1 < H/D < 8.

Table B-9 defines the range of soil parameters for uplift resistance offered by the additional material in a trench where the primary backfill consist of clay. Key parameters defining the mobilisation of the resistance can be seen in Figure B-9.

Table B-9 Defined ranges of soil parameters for resistance of added material (R_2) in trench with clay

Gravel / Rock	f	$\in$	[0.5 , 0.8]
	δ_f	=	0.4·D m
	α	$\in$	[0.7 - 0.8]
	β	=	0.25
	z_p	$\in$	[0.05 - 0.10] m
Sand or sand mixed with Gravel	f	$\in$	[0.4 , 0.6]
	δ_f	=	0.6·D m
	α	$\in$	[0.7 - 0.8]
	β	=	0.3
	z_p	$\in$	[0.02 - 0.05] m

The same limitations with respect to cover ratios and particle sizes as defined for Table B-2 applies.

Table B-10 defines the range of soil parameters and the recommended formulae for vertical downward resistance in a trench with clay. Key parameters defining the mobilisation of the resistance can be seen in Figure B-9. A simplified expression for the secant vertical stiffness K_t as functions of the shear strength is also given. The simplified expressions consider significant pipe vertical displacement (i.e. beyond soil failure at initial position in trench with more than half the pipe embedded, e.g. *H/D* > -0.5) and is only intended for initial estimates of the vertical stiffness.

For most cases we expect that the analysis is sensitive to the stiffness and so the full non-linear load-displacement curve as established Eqs. (B.28) or (B.29) should be used, also considering the installation sequences.

Table B-10 Defined range of soil parameters and recommended formula for vertical stiffness valid for embedded pipes in clay

Clay "intact conditions"	K_t (kN/m/m)	=	$25 \cdot s_u$ (kN/m/m)
	s_u	$\in$	[20, 60] kN/m^2
	$R_{V,trench}$	=	$N_c \cdot \eta \cdot s_u \cdot D$
	N_c	=	According to Eq. (B.23) Note! substitute (H+D/2)/2 with z/D to find correct N_c value.
	η	$\in$	[0.55 , 0.80]
Clay "homogeneous conditions"	K_t (kN/m/m)	=	$60 \cdot \bar{s}_{ur}$ (kN/m/m)
	$\bar{s}_{ur}$	$\in$	[2, 20] kN/m^2
	$R_{V,\text{int}\,act}$	=	$(N_c \cdot s_u + p_o') \cdot B(z) \qquad z < D/2$
	N_c	=	According to Eq. (B.23)
	z_o	$\in$	[0.1 · D, 0.5 · D] m

B.4.9 Comparison of models

In order to compare the various uplift resistance model the following approach is used:

Normalising each resistance with the term $\gamma' HD$, simplifying the expression:

$$\gamma' \cdot D^2 \left(\frac{1}{2} - \frac{\pi}{8}\right) \Big/ \gamma' H \cdot D \quad \text{to} \quad 0.1 \cdot \left(\frac{D}{H}\right)$$

and substituting the undrained strength of the remoulded clay by a dependency to the effective vertical stresses through the ration $\frac{s_u}{\sigma_v'}$, yields:

Local soil failure model (see Eq. (B.22)):

$$\frac{R}{\gamma' HD} = \frac{s_u}{\sigma_v'} \cdot \eta \cdot N_c + \frac{s_u}{\sigma_v'} \cdot \eta \cdot N_c \left(\frac{D}{2 \cdot H}\right) \tag{B.34}$$

Figure B-11 shows this normalised uplift resistance vs. *H/D* ratio for a N_c value computed from Eq. (B.23) and $\frac{s_u}{\sigma_v'}$ in a range of 0.22 to 0.30.

Global soil failure model (see Eq. (B.24))

$$\frac{R}{\gamma' HD} = 1 + 0.1\left(\frac{D}{H}\right) + \frac{s_u}{\sigma_v'}\left(\frac{H}{D} + 1 + \frac{D}{4 \cdot H}\right) \tag{B.35}$$

Figure B-12 shows this normalised uplift resistance vs. *H/D* ratio for two different $\frac{s_u}{\sigma_v'}$ values, 0.22 and 0.30.

When the remoulded strength of the soil is more appropriate (ploughing), then a global soil failure mode can be rewritten in the form of:

$$\frac{R}{\gamma' HD} = 1 + 0.1\left(\frac{D}{H}\right) + s_{ur} \cdot \left(\frac{1}{\gamma' \cdot H} + \frac{2}{\gamma' \cdot D}\right) \tag{B.36}$$

where

s_{ur} = average remoulded shear strength from pipe to top of cover.

Figure B-13 shows this normalised uplift resistance vs. *H/D* ratio with a remoulded strength of 1 kPa and 2 kPa. A submerged weight of the clay of γ' = 7 kN/m^3 is used in the presentation.

Drained resistance model (see Eq. (B.1))

$$\frac{R}{\gamma' HD} = 1 + 0.1\left(\frac{D}{H}\right) + F\left(\frac{H}{D}\right) \tag{B.37}$$

where

$$F = K \tan(\phi) \cdot \left(H + \frac{D}{2}\right)^2 \Big/ H^2$$

Figure B-14 shows this normalised uplift resistance vs. *H/D* ratio for a range of $K \cdot \tan(\phi)$ between 0.4 and 0.2.

Finally Figures B-15 and B-16 show a comparison of the various models for uplift resistance in soft clay. The three models are presented as function of *H/D* ratio being cover height over diameter ratio.

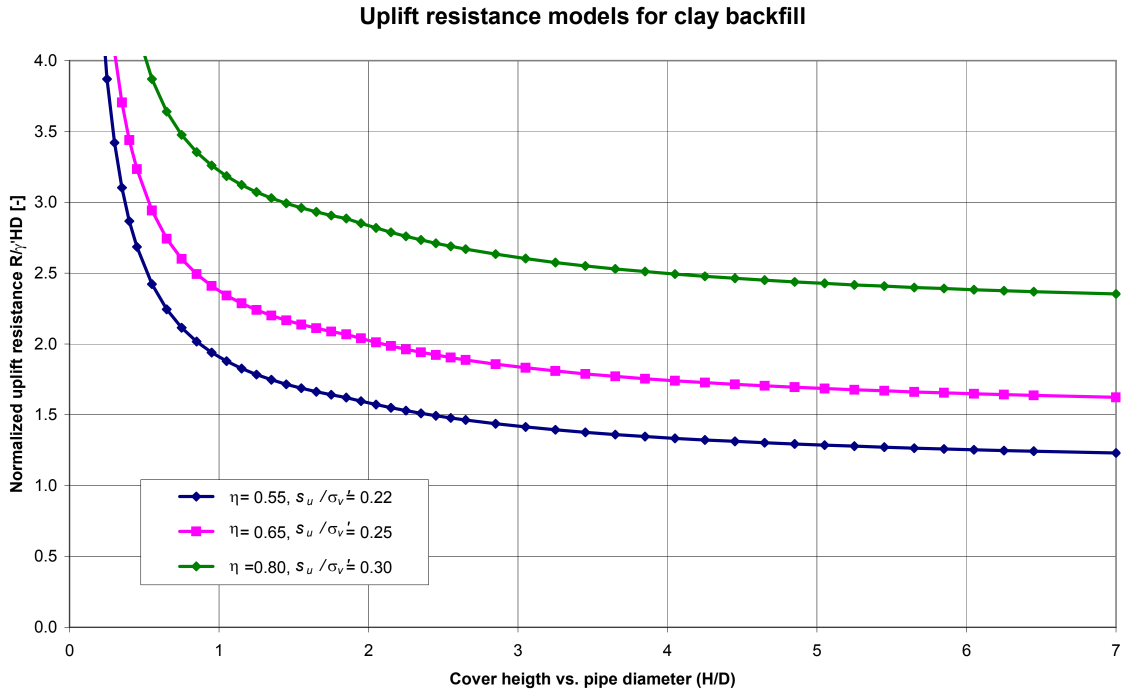


Figure B-11
Normalised uplift resistance - local soil failure $(0.22 < \dfrac{s_u}{\sigma_v'} < 0.30,\ 0.55 < \eta < 0.80)$

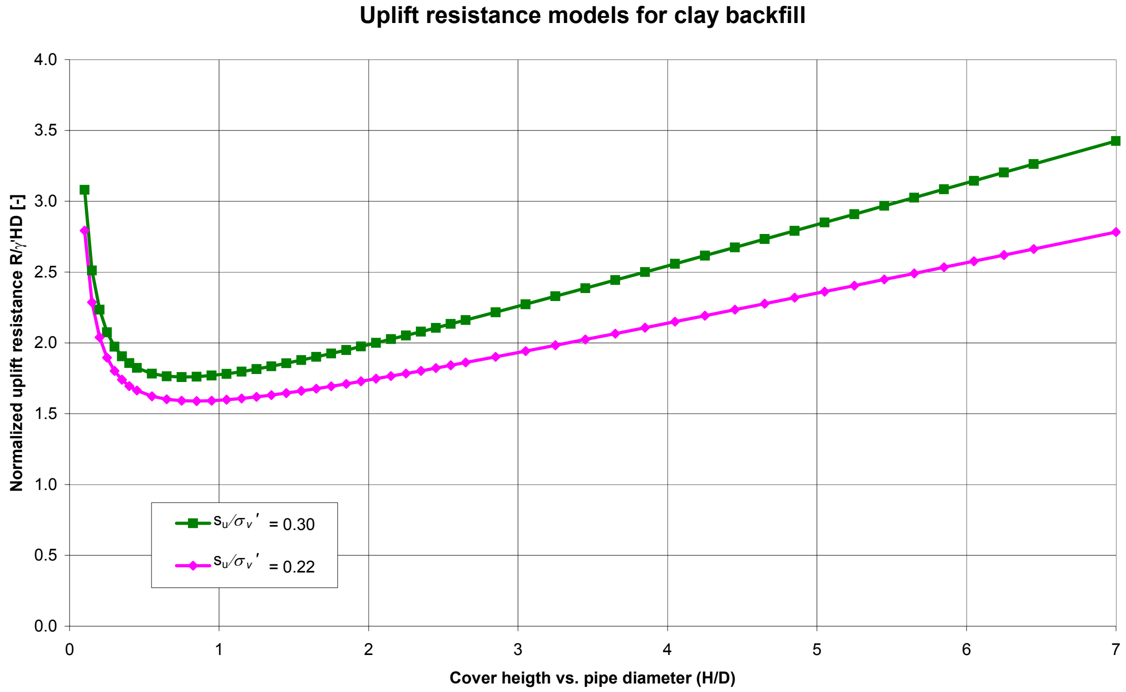


Figure B-12
Normalised uplift resistance - global soil failure $0.22 < s_u/\sigma_{v'} < 0.3$

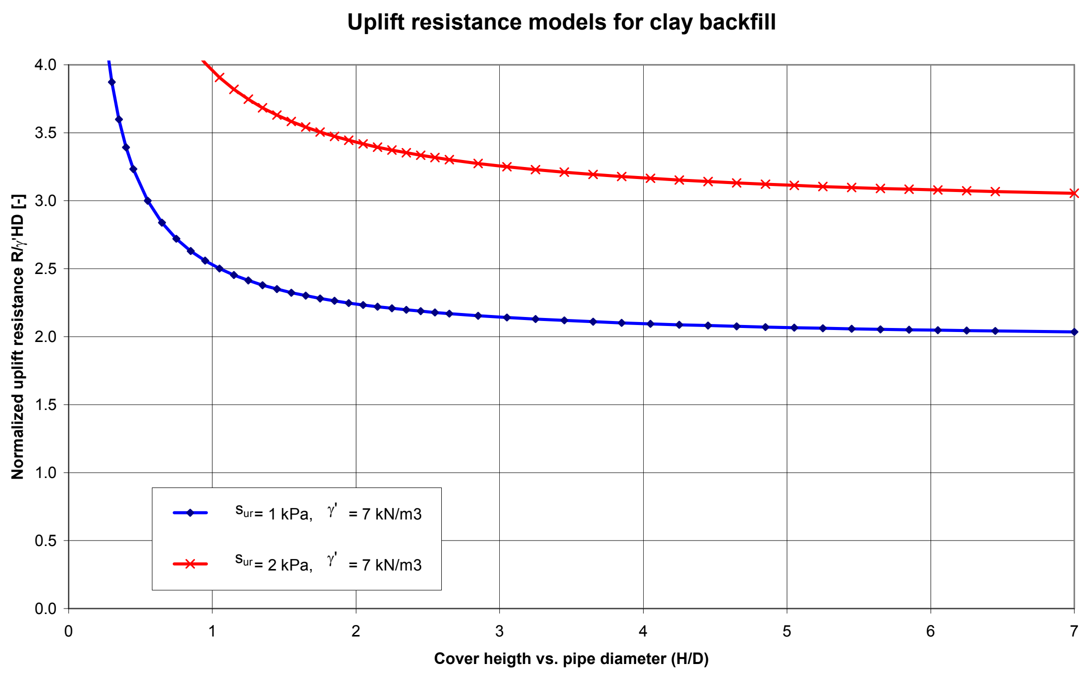


Figure B-13
Normalised uplift resistance - global soil failure - constant remoulded shear strength

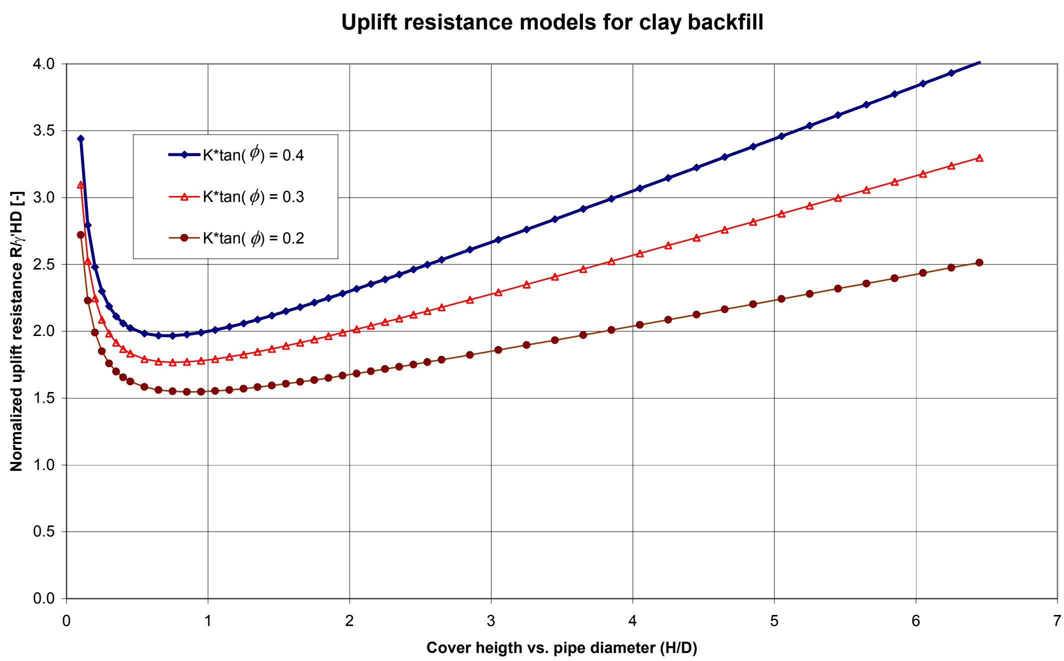


Figure B-14
Normalised uplift resistance – global soil failure - drained resistance

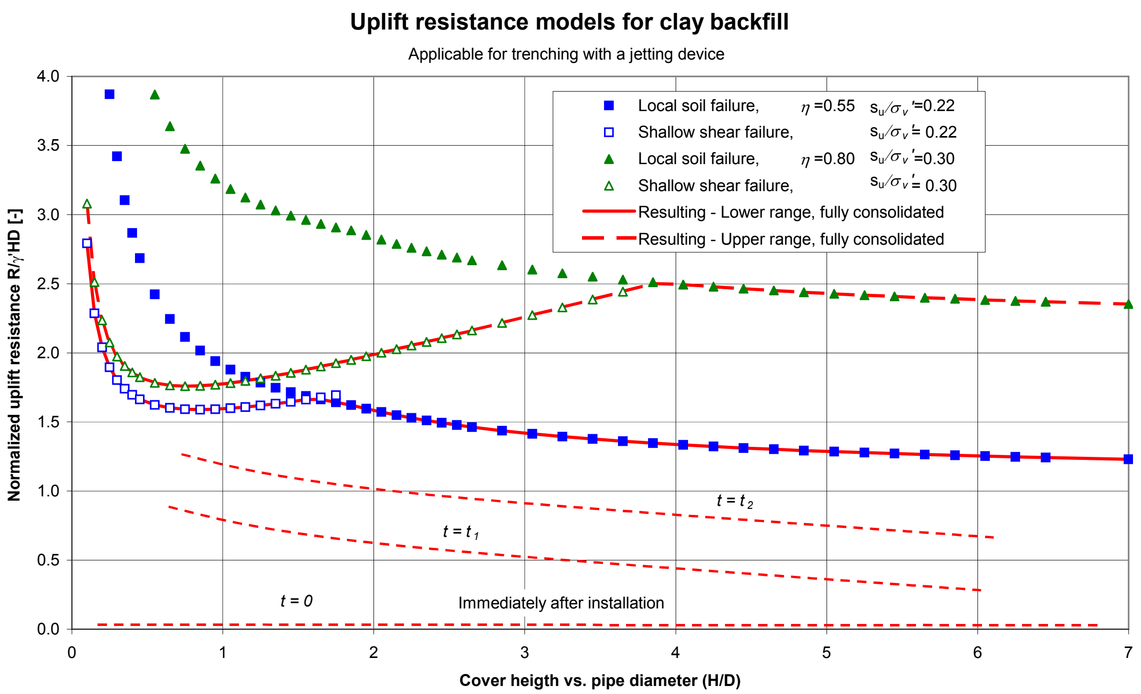


Figure B-15
Normalised uplift resistance model for a pipe trenched with a jetting device

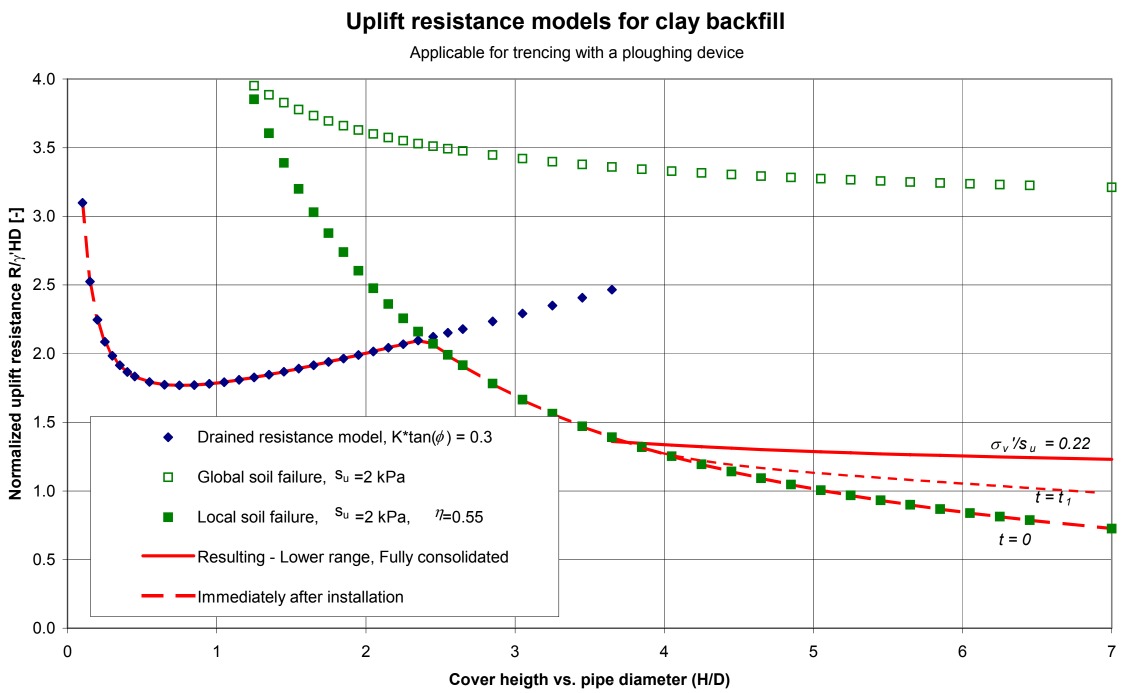


Figure B-16
Suggested normalised uplift resistance model for a pipe trenched with a ploughing device

B.5 References

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/2/ M. F. Randolph and G. T. Houlsby “The limiting pressure on a circular pile loaded laterally in cohesive soil”, Gèotechnique 34, no. 4, pg.613- 623.

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/4/ Schaminée, P.E.L., N.F. Zorn, and G.J.M. Schotman, “Soil response for pipeline upheaval buckling analyses: full-scale laboratory tests and modelling,” Proceedings, 22nd Annual Offshore Technology Conference, Houston, Texas, Paper No. OTC6486, 1990.

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/6/ DNV, “Recommended Practice, DNV-RP-F105, Free Spanning Pipelines,” Edition February 2002.

/7/ OSIF (Offshore Soil Investigation Forum), “Guidance Notes on Geotechnical Investigations for Marine Pipelines”, rev. 03, dated 22nd of February 1999.

APPENDIX C
EXAMPLE OF DESIGN FLOW FOR EXPOSED PIPELINES

C.1 General

C.1.1 Pipeline configuration phases

As stated in the main part of this RP, the global buckling will experience the following three phases:

Phase 1: The free spans deflect and may get in touch with the sea bottom.

Phase 2: Uplift at crests.

a) Further expansion may lift off the pipe at a few crests in a limited way: say, less than 50% of the pipe diameter for a lifted length less than 50 diameters, being stable against horizontal perturbations

b) Even further expansion may increase the upward displacements at crests, (free span shoulders, of the most pronounced undulations), bifurcate and turn down to seabed developing lateral snaking in the seabed plane.

Phase 3: Even further expansion will increase the bending of the snaked pipeline

C.1.2 FE-modelling

An important practical task in the design is to limit the complexity of the analyses which can be huge, by use of 2D, 2 ½ D or 3D models when possible. Specific design checks may therefore be required to prove the relevancy of the simplified models.

C.1.3 Expansion

The pipeline engineer has to perform a design check to verify how far the expansion will develop: the first phase is a standard design condition while the latter two are more critical and need particular attention. Which of the two deflections (vertical or lateral) is more relevant, depends on:

— the local features of the uneven seabed (2 vs. 3-Dimensional roughness)
— service load conditions
— length of the expected uplift sections
— possibly, pipe out-of-straightness from the laying procedure.

In these circumstances, mitigation measures can be required to avoid undesirable/uncontrolled developments of bending moments/deformations.

C.1.4 Expansion control

The development of the deformation pattern (upheaval, lateral turn down and lateral snaking) and associated bending deformations can be controlled by:

— smoothing the sea bottom roughness (trenching technology) and/or covering the pipeline with gravel at the sea bottom crests to cancel lift off where undesirable
— reducing/controlling the axial feed-in by gravel dumping, suitably located and distributed in the proximity of the expected buckles, in order to limit the development of bending allowing the axial expansion to be shared between adjacent buckles.

C.1.5 Mitigation

The shape and extent of mitigation measures should be designed in order to:

— maintain the maximum bending moment/deformation in the buckled regions below allowable limit
— control pipeline uplift in order to avoid pipeline turn down if the change of buckling plane from vertical to horizontal and subsequent development of bending deformation in the horizontal plane is not allowed due to the sea bottom roughness transversal to pipeline route or e.g. third party activities.

C.2 Design Procedure

The procedure is split in three levels associated to different analysis methodologies, design criteria and mitigation approach, see Figure C-1:

1) *Level 0 - Traditional Design*: This analysis level includes Traditional 2 Dimensional (2D) Bottom Roughness Analysis. This level should typically be applied:

   — during the feasibility and basic design phase
   — when minor pipeline uplift occurs at a few locations along the analysed pipeline stretch and lateral turning down is not expected to occur. Traditional intervention works can be used to cancel or control or allow (wait and see) the uplift at the interested pipe sections.

2) *Level I - Advanced Design*: This advanced level should typically be applied:

   — during the basic and detailed design phase
   — when the thermal and pressure loads are released by upward displacements at several but well defined locations (natural crests of the sea bottom roughness or purpose made crest) and, finally, by lateral snaking developing in the seabed plane after turning down.

   Mitigation measures are defined in order to control the axial feed in by limiting the anchoring length. In particular, the interested pipeline stretch is subdivided into a number of flexurally and axially independent sections, using gravel cover lengths, opportunely located along the pipeline stretch, sufficient to fully anchor the pipeline. In this condition, it is anticipated that one buckle is expected to develop in each pipeline section between cover lengths. Both 2D, 2½D and 3D FE models can be used to perform the analyses of the pipeline under operating conditions.

3) *Level II - Advanced Design*: This level should typically be applied:

   — during the detailed design phase
   — in cases where the pipeline configuration in the lateral plane, due to local built-in lateral curves or sea bottom roughness transversal to the pipeline route, has to match with 2½ or 3D unevenness and effective axial compression are released by a combination of pipeline uplift/turn down/lateral snaking at the crests of the most pronounced undulations
   — in cases where the concept of axially independent sections is not pursuable and the axial interaction (sharing of axial expansion) between adjacent buckles must be pursued to control the axial feed in at releasing of pressure loads and thermal expansion based on lower bound axial restraints.

   Mitigation measures are defined on the basis of the axially interacting pipe concept controlling the axial feed-in by sharing the axial expansion between adjacent buckles. Generally, 2D or 3D FE models are applied to define intervention works due to temporary conditions (as-laid, flooded and pressure test), while 2½D or 3D FE models are generally used to perform the analyses of the pipeline under operating conditions.

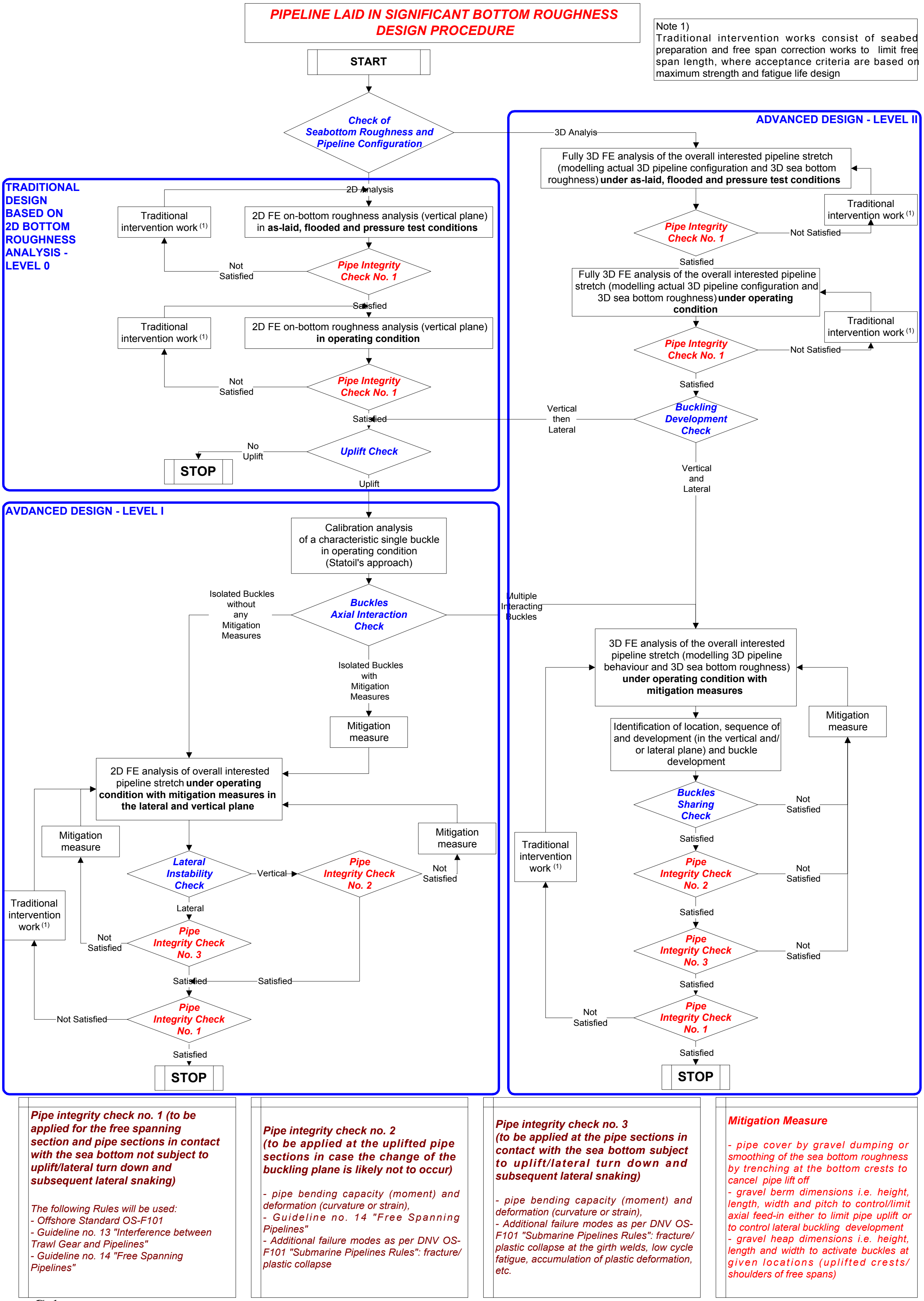

Figure C-1
Design procedure for pipelines on un-even seabed

C.3 Global buckling assessment considerations

In order to document global buckling assessment, ether 2D, 2½D or 3D analyses can be used. Special attention should be taken in cases where there are purpose-built in curves in the horizontal plane or considerable sea bottom unevenness transversal to the pipeline route.

Generally, sensitivity FE analysis in the vertical plane (2D, 2½D or 3D FE analysis) of the interested pipeline stretch under operating condition, should be carried out (with respect to axial and lateral frictional restraint). In case the sea bottom roughness, transversal to the pipeline route is such as to change the configuration of the pipeline for minor transversal shifts, the development of buckle is quite difficult to predict. Should the axial frictional force be high, it will result in a larger number of buckles than in case of a lower frictional force. An increased number of buckles cause less thermal feed in to each of them and hence less bending moments and strains developed than in case of isolated or distant buckles, for a given value of the lateral friction resistance. The principles of sharing in Sec.6.5 shall apply.

Evidence from structural modelling is that the effective axial load applied on the uplifted span section increases as the internal pressure and temperature increase since the uplifted section is able to absorb compression.

APPENDIX D
UNCERTAINTY DISCUSSIONS

D.1 General

In the HotPipe Phase 1, an analysis methodology was proposed to quantify the load condition factor multiplying the applied bending moment as a function of the relevant parameters i.e. lateral soil restraint, axial soil restraint, steel stress-strain curve and pipeline interaction with trawling board, if any. Appendix A of this document reports also an equivalent methodology to include the effect of the presence of free spanning pipeline sections on the maximum bending moment applied on the pipeline in case of lateral expansion and/or vertical expansion.

Assuming that the sea bottom unevenness, transversal to the pipeline route, is not so relevant, the main difference between the flat and uneven sea bottom scenario is that, in the latter case, the feed in region contain spans or other seabed unevenness, not large enough to cause another buckle, but perhaps large enough to result in significant geometric shortening of the pipe under increasing operating loads (the longer the free span and the denser the distribution the lower the axial feed-in at the buckle), then affecting the axial feed in at the buckle and the subsequent development of bending moment at the buckle. Particularly, a pipe stretch adjacent to the buckle feeds into the buckle and, in case of extensive free spanning, the axial feed in is less than in case of flat sea bottom as a function of free span length and extension.

In fact, the axial feed-in at the buckle depends on the following main parameters:

— free spans density i.e. overall length of the pipeline free spanning with respect to overall pipeline length in touch with the sea bottom
— length of free span and location i.e. whether and how the free spans present in the feed-in region are close or far from the Euler bar buckling condition.

The effect of free spans on the maximum load effect, and consequently on the condition load factor, γ_C, is shown in Figure E-1. The free spans present in the pipeline sections adjacent to the buckle give:

a) A reduction in the mean value of developed bending at the buckle, due to the release of the thermal expansion caused by the downward deflection of long and sagged free spans. This causes a reduction of both the characteristic value of developed bending and of the failure probability for given resistance characteristics and design conditions.
b) An increase in the standard deviation of the prediction due to increased role of friction on the effective compression. This causes an increase of the failure probability and then of the design load effect for given resistance characteristics and design conditions.

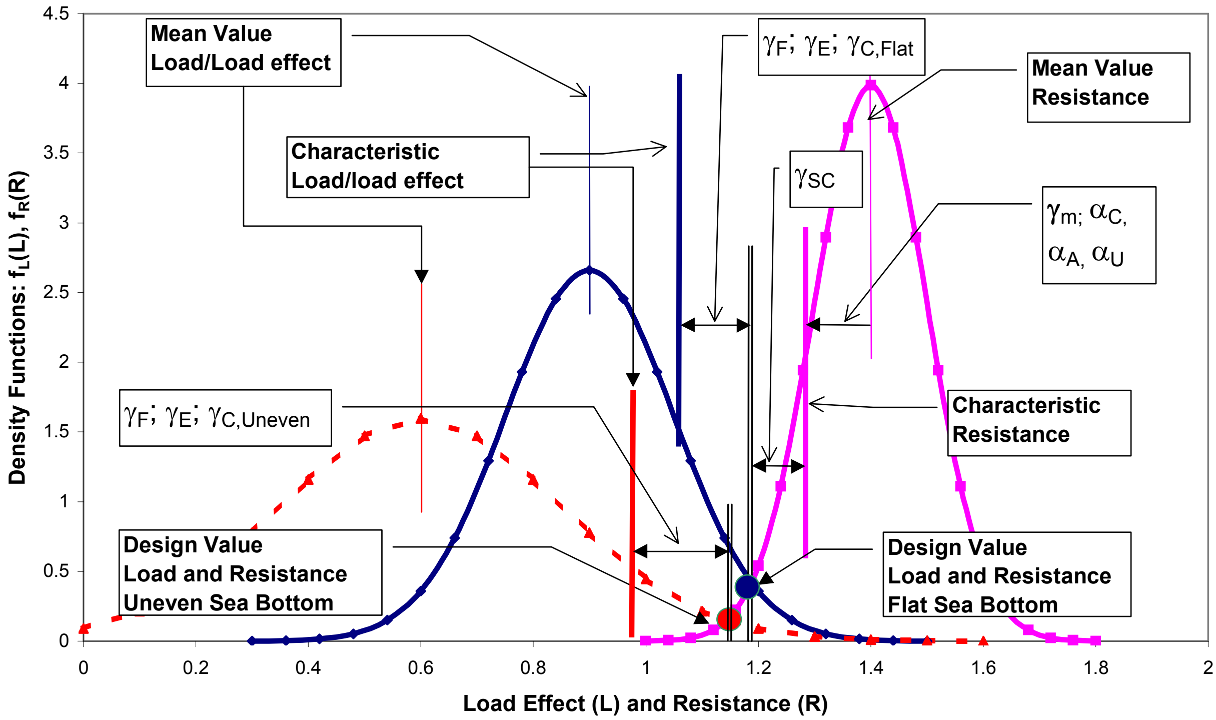

Figure D-1
Schematization of the effect of the free spanning section on the applied maximum bending moment in an adjacent buckle.

In general, assuming a constant value of the failure probability, it is expected that the design point for an uneven sea bottom is lower (or at maximum equal) than the one relevant for flat seabed.

Considering both effects, going from a flat seabed to an uneven one, it is expected that:

— the design load effect of the uneven seabed must decrease (at limit condition where free span are not affecting the axial feed in at the buckle, it will be coincident with the one relevant for flat seabed)

— the condition load factor, γ_C, slightly increases depending on sea bottom roughness

Table 6-2 summarises the main parameters affecting the total bending moment and therefore the load condition factor in case of flat and uneven sea bottom. In particular, the uncertainty of the total bending moment applied in the buckle, depends on the following parameters, see also Appendix B:

a) Uncertainty in the axial friction, μ_A, and the free span distribution on the two pipeline sections adjacent to the buckle (relevant for the development of bending moment in the vertical plane before and after turndown in case turndown is not allowed).

b) Uncertainty in the lateral soil friction, μ_L (relevant for development of bending moment in case of lateral snaking).

c) Uncertainty in the shape and dimensions of the initial imperfection affecting pipe curvature at the sea bottom crests, relevant for the maximum applied bending moment in case that lateral snaking does not occur and the pipeline expands in the vertical plane. This uncertainty is also relevant for the evaluation of the maximum bending moment at turn-down in case of first vertical and then lateral expansion.

d) Uncertainty in the applied stress-strain curve, as considered in HP1GL.

e) Uncertainty in the applied trawl load as considered in HP1GL, if relevant.