MANUAL

UPHEAVAL BUCKLING OF PIPELINES

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DESIGN AND ENGINEERING PRACTICE

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1. INTRODUCTION

1.1 SCOPE

This new DEP gives guidance on the assessment of the risk of upheaval and buckling of offshore pipelines and provides alternatives for corrective action prior to or after pipeline construction to prevent upheaval buckling occurring.

The design equations in this DEP apply to single-pipe pipeline concepts. The basis of these equations may be used to devise design equations for other concepts such as pipe-in-pipe.

Generally applicable definitions and requirements for pipeline engineering can be found in DEP 31.40.00.10-Gen.

The principles described in this DEP may also be used to assist with the design of on-land pipelines against upheaval buckling.

1.2 DISTRIBUTION, INTENDED USE AND REGULATORY CONSIDERATIONS

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This DEP is primarily intended to be used for oil and/or gas transmission pipelines and related facilities.

If national and/or local regulations exist in which some of the requirements may be more stringent than in this DEP, the Contractor shall determine by careful scrutiny which of the requirements are the more stringent and which combination of requirements will be acceptable as regards safety, environmental, economic and legal aspects. In all cases the Contractor shall inform the Principal of any deviation from the requirements of this DEP which is considered to be necessary in order to comply with national and/or local regulations. The Principal may then negotiate with the Authorities concerned with the object of obtaining agreement to follow this DEP as closely as possible.

1.3 DEFINITIONS

1.3.1 General definitions

The **Contractor** is the party which carries out all or part of the design, engineering, procurement, construction, commissioning or management of a project, or operation or maintenance of a facility. The Principal may undertake all or part of the duties of the Contractor.

The **Principal** is the party which initiates the project and ultimately pays for its design and construction. The Principal will generally specify the technical requirements. The Principal may also include an agent or consultant, authorised to act for, and on behalf of, the Principal.

The word **shall** indicates a requirement.

The word **should** indicates a recommendation.

1.4 SYMBOLS

b	breadth of the geotextile (measured transverse to the pipeline)
D	outside diameter
D'	outside diameter + mattress thickness
E	elastic modulus (Young's modulus)
f	uplift resistance coefficient
f'	limiting longitudinal frictional resistance per unit length
F	flexural rigidity

h	height of isolated hill in profile
H	$^1/_2$ (peak-to-trough height of seabed profile) (Figure 2.4); cover height (2.4) (Figure 2.8)
L	$^1/_2$ (wavelength of seabed profile)
L_i	horizontal distance between i-th and i+1-th inflection point
m	mass flow rate
M	bending moment
p	operating pressure
Ne	axial force (longitudinal compressive force)
q	downward force per unit length required to hold pipeline in position
r	uplift resistance per unit length
r'	additional uplift resistance per unit length from geotextile
R	mean radius, $^1/_2$ (outside pipe diameter-wall thickness)
S	shear force
s_L	longitudinal stress
t	wall thickness
T_R	residual tension
U	flow velocity (mean across pipe cross-section)
w	weight per unit length of pipeline (2.3.3.3); weight per unit area of mattress (3.4)
x	horizontal distance along pipeline
y	height of pipeline from datum
z	height of seabed from datum (2.3.3.1); distance over which movement extends (2.2.2)
α	linear thermal expansion coefficient
γ	submerged unit weight of cover
Δ	height of prop imperfection
μ	coefficient of friction between rock and geotextile
μ'	coefficient of friction between geotextile and seabed
ν	Poisson's ratio
ΔT	difference between operating temperature and installation temperature (positive if operating temperature is higher than installation temperature)

1.5 CROSS-REFERENCES

Where cross-references to other parts of this DEP are made, the referenced section number is shown in brackets. Other documents referenced in this DEP are listed in (7) and (8).

2. ASSESSMENT OF RISK OF UPHEAVAL

2.1 INTRODUCTION

Upheaval buckling occurs in buried pipelines that operate at high temperatures and pressures. Figure 1.1(a) shows a dramatic example in a land pipeline bowing upward in a long raised loop, and Figure 1.1(b) a buckled marine pipeline (8.1). The line can be deformed to an unacceptable extent, and/or the buckle may move it into a position in which it is exposed to other kinds of damage. For instance, an underwater line bowed above the seabed becomes exposed to damage by fishing trawls and by current/wave action.

Upheaval buckling has been known for a very long time as a problem of land pipelines, but at first was generally believed not to affect underwater pipelines. In the last ten years it has been recognised as a serious problem. The earliest and best-documented example was the 8-inch Rolf to Gorm pipeline in the Danish sector of the North Sea (8.1), but there have been many other instances in the British, Norwegian and Danish sectors. None of them has yet led to a loss-of-containment incident, but the problem has been recognised as potentially serious. Industry sensitivity to the problem has been increased by a continuing trend towards higher operating pressures. Operating temperatures above 100 °C are nowadays commonplace, and temperatures well above 150 °C are being considered. At such temperatures almost every trenched or buried pipeline has an upheaval problem.

Figure 1.2 is an algorithm which indicates a systematic approach to upheaval buckling, and cross-references the relevant sections in this DEP.

This Section describes how to assess whether or not there is a risk of upheaval. The assessment proceeds in four steps.

The first step is the calculation of driving force, and is covered in (2.2). Three aspects are covered: (2.2.1) covers fully-constrained lines, (2.2.2) covers partially constrained lines, and (2.2.3) discusses residual tension.

The second step is the determination of the total downward force required for the pipeline to stay in position without upheaval: this step is covered in (2.3).

The third step is the calculation of the available downward force (sum of pipeline weight and uplift resistance) if the line is buried: this step is covered in (2.4).

The fourth step is the comparison between the required downward force and the available downward force, described in (2.5).

2.2 DRIVING FORCE

2.2.1 Driving force: fully-constrained pipelines

The driving force that creates the upheaval buckling is the longitudinal compressive force in the restrained pipeline and its contents.

One component of this force is due to temperature increase. A second component of the driving force is due to pressure, which generates a compressive force resultant over the whole cross-section, taking the fluid contents and the pipe wall together. The third component is the residual tension left by laying, and subtracts from the other two.

The longitudinal compressive force Ne in the wall and contents together induced by the operating temperature and pressure and the residual tension is

$$Ne = (1-2\upsilon)\pi R^2 p + 2\pi R t E \alpha \Delta T - T_R \qquad \text{(equation 2.2.1)}$$

If the operating pressure p and temperature increase ΔT are each positive or zero, as is usually the case, Ne is positive.

The notation is listed below:

pipeline dimensions;

- t wall thickness
- R mean radius, $^1/_2$ (D -t)

material properties;

- E Young's modulus (elastic modulus)
- ν Poisson's ratio
- α linear thermal expansion coefficient

operating conditions;

- p operating pressure
- ΔT the difference between the operating temperature and the installation temperature (taken as positive if the operating temperature is higher than the installation temperature)

construction parameters;

- T_R residual tension see (2.2.3).

It is important to avoid over-conservative choices of operating parameters. The calculation of the driving force at a particular point on the pipeline should take account of the predicted temperature and pressure profiles **along** the line, and not use the maximum values at pipeline inlet. Production profiles and reservoir analysis may indicate that the maximum operating temperature may not occur at the same time as the maximum operating pressure: the design longitudinal force ought then to be based on the worst combination that can occur, rather than on the two individual maxima.

The magnitudes involved can be seen by a sample calculation, for the following parameters:

Outside Diameter	273.05	mm (nominal 10-inch)
t	11.3	mm
E	210000	N/mm^2
ν	0.3	
α	1.17×10^{-5}	/°C
p	30	N/mm^2 (300 bar)
ΔT	100	°C
T_R	500	kN

and then

$(1-2\nu)\pi R^2 p$	0.65	MN
$2\pi RtE\alpha\Delta T$	2.28	MN
T_R	0.50	MN
Ne	2.43	MN

Equation 2.2.1 and this specimen calculation both assume that the pipeline remains elastic. If in fact the pipeline is no longer elastic but has reached yield under the design operating conditions, as it may do if strain-based design has been adopted, then the calculation of longitudinal force must take account of yield.

2.2.2 Quantification of driving force: partial longitudinal constraint

Pipelines are not always fully constrained. At an elbow connecting a pipeline to a platform riser, for instance, the pipeline is free to expand towards the platform, because the riser is flexible by comparison with the pipeline. Similarly, an expansion loop is incorporated in order to allow a pipeline to expand, and this expansion reduces the axial force P.

Figure 2.1 illustrates one simple case, a vertical riser connected through an elbow to a seabed pipeline. At the elbow, the total longitudinal force is the shear force S in the vertical arm between the elbow and the lowest clamp. As far as the pipeline is concerned, S is usually negligible in comparison with the axial force defined in equation 2.2.1.

The expansion movement of the pipeline towards the platform is resisted by seabed friction. If the frictional force between the pipeline and the seabed is f' per unit length, the axial force at a distance x from the platform is S+f'x, up to the point at which the axial force becomes equal to the force in a fully constrained line.

If the operating pressure and temperature are uniform along the length of the line, and the residual tension T_R is negligible, the distance z at which the axial force reaches the fully constrained value is

$$z = \frac{1}{f'}(1-2\nu)\pi R^2 p + 2\pi RtE\alpha\Delta T - S) \qquad \text{(equation 2.2.2)}$$

Accordingly, if x < z

$$Ne = f' x + S \qquad \text{(equation 2.2.3)}$$

but if x > z, Ne is given by equation 2.2.1 as before.

This theory can be generalised (Palmer and Ling (8.2)) to the case where the temperature is not uniform but decreases exponentially with distance from the platform, and to the case where the longitudinal resistance f’ is not uniform with distance along the pipeline. It can readily be generalised further to the case where the pressure is non-uniform, but this is in practice a small effect.

If a lateral or upheaval buckle occurs, the pipeline generally comes to rest with the longitudinal force in the buckle less compressive than the force before the buckle occurred. The pipeline moves longitudinally towards the buckle, but these movements too are resisted by frictional resistance from the seabed. The effect on the longitudinal force in the pipeline is illustrated in Figure 2.2. As far as the force in the rest of the pipeline is concerned, the buckle behaves rather like an expansion loop, and reduces the longitudinal force on either side. For this reason, buckles rarely occur close together.

The longitudinal frictional force per unit length f' must be chosen thoughtfully. It is usually calculated by multiplying the submerged weight per unit length by a friction coefficient μ, but the appropriate value is often uncertain. Soil mechanics indicates that μ is to be expected to be about 0.5 on frictional soils such as sand, but lower on stiff clay. Calculations from expansion movements measured in the field suggest higher values, about 0.9, probably because of the effect of small lateral movements.

It is not always conservative to assign a low value to μ. If μ is assumed smaller than the correct value, f' is smaller and z is larger than the correct value, so that a low value of μ will indicate that the partial restraint reduces the longitudinal force below the fully-constrained value over a longer distance. Calculations should therefore be made for a high value and a low value, and the worst case used in design.

2.2.3 Residual tension

The residual effective tension T_R on the seabed can be calculated with a laystress calculation programme.

If the line is installed by pull or tow, the residual tension is the pull force or the tow force.

Relaxation of the installation tensions should be evaluated.

External pressure should not be incorporated in the longitudinal force as the pipeline is not restrained when applying external pressure during installation (8.3).

2.3 STABILISATION AGAINST UPHEAVAL

2.3.1 Upheaval

A longitudinally-compressed pipeline tends to lift upwards on overbends ("hills") in the profile, and to move downwards on sagbends ("valleys"). There is almost always a high degree of resistance to downward movement, which is of little concern. There is much less resistance to upward movement, which may lead to upheaval.

Once the pipeline begins to move upward, the external force required to hold it in position almost always increases, whereas the external resistance to movement almost always decreases. Generally, the pipeline becomes unstable soon after it starts to move, and then 'jumps' into a new position with very much larger deflections. This is the kind of upheaval it is important to avoid.

The strategy adopted in this DEP is to find the external force required to hold the pipeline against upheaval, and to check whether that force can be supplied by weight or by forces exerted by the surroundings.

2.3.2 Is the pipeline profile fixed or can it change ?

Two cases have to be distinguished when assessing the profile of a pipeline.

In the first case the pipeline profile is fixed, because the pipe is continuously (or almost continuously) in contact with a foundation, such as a trench or the seabed, to hold the pipeline in that position. This case is examined in (2.3.3).

In the second case the pipeline is not continuously supported everywhere and its profile may change during start-up. Also, stabilisation measures such as rock dumping may themselves change the pipeline profile. This case is examined in (2.3.4).

2.3.3 Stabilisation in a fixed profile

2.3.3.1 General profile

The external force required to keep a pipeline in equilibrium depends on the pipe profile. Figure 2.3 shows a pipeline profile and a seabed profile. Horizontal distance is denoted by x, measured from an arbitrary datum. The height of the pipeline is denoted y, measured upwards from an arbitrary datum. The height of the seabed (or, for a trenched pipeline, the trench bottom) is denoted z, measured upwards from the same datum. y and z may not be the same, because the pipeline can lose conformity with the seabed by lifting off it.

From applied mechanics theories, the external force per unit length required to hold the pipe in position is:

(equation 2.3.1)

$$q = -Ne\frac{d^2y}{dx^2} - \frac{d^2M}{dx^2}$$

If the curvature d^2y/dx^2 is less than the yield curvature, the bending moment M is proportional to the curvature d^2y/dx^2. The constant of proportionality is the flexural rigidity F, so that:

(equation 2.3.2)

$$M = F\frac{d^2y}{dx^2}$$

For elastic steel pipelines, F is EI, where I is the second moment of area of the cross-section and can be taken as $\pi R^3 t$, which neglects ovalisation.

Substituting into equation 2.3.1,

$$q = -Ne\frac{d^2y}{dx^2} - F\frac{d^4y}{dx^4}$$ (equation 2.3.3)

Equation 2.3.1 is always applicable. Equation 2.3.3 is applicable if the pipe is known not to have bent so far that it has yielded.

These equations can be used to find the force required to keep the pipeline in position. If the pipe profile is known or can be calculated, the profile can be differentiated to determine the curvature d^2y/dx^2 and its second derivative d^4y/dx^4, and then the equations determine the external force needed to hold the pipeline in position. If the profile does not change when the longitudinal force is increased, and if there is enough external force available to keep the pipe in position, the pipeline does not move and upheaval does not occur.

2.3.3.2 Sinusoidal profile

A continuous supported sinusoidal profile is shown in Figure 2.4. It is composed of regular 'waves' of height 2H and wavelength 2L, described by the equation:

$y = H(1 - \cos\pi x/L)$

so that:

$$\frac{d^2y}{dx^2} = H\left(\frac{\pi}{L}\right)^2 \cos\pi x/L$$ (equation 2.3.4)

$$\frac{d^4y}{dx^4} = H\left(\frac{\pi}{L}\right)^4 \cos\pi x/L$$ (equation 2.3.4)

From equation 2.3.3:

$$q = -\left(NeH\left(\frac{\pi}{L}\right)^2 - FH\left(\frac{\pi}{L}\right)^4\right)\cos\pi x/L$$ (equation 2.3.5)

The largest required downward force q_{max} occurs on the summit of the hills when x = L, and is

$$q_{max} = NeH\left(\frac{\pi}{L}\right)^2 - FH\left(\frac{\pi}{L}\right)^4 \quad \text{if } L > 4.44\sqrt{\frac{F}{P}}$$ (equation 2.3.6)

$$q_{max} = \frac{1}{4}\frac{Ne^2H}{F} \qquad \text{if } L > 4.44\sqrt{\frac{F}{P}}$$ (equation 2.3.6)

2.3.3.3 Profile defined by spot heights

A real seabed profile is less regular than the sinusoid illustrated in Figure 2.4, but the results developed in (2.3.2) can be generalised to idealisations of irregular profiles.

Imagine a general profile characterised by irregular peaks and troughs; Figure 2.5 shows such a profile, represented by a pipeline height y which is a function of a horizontal distance x. The profile has inflection points where the derivative dy/dx has a maximum or a minimum and the second derivative is zero. The inflection points are marked and numbered on the profile: inflection point i is in horizontal position x_i and at height y_i.

Ideally, each peak between consecutive inflection points would be represented by the complete profile, but that information may not be available. The most important information about the sharpness of the peak is covered by two parameters:

H_i is the maximum upward offset between the profile and a line between inflection points i and i+1; and

L_i is $x_{i+1}-x_i$, the horizontal distance between inflection points i and i+1

so that H_i describes the height of the peak and L_i describes its horizontal extent.

The profile between the inflection points can then be idealised as

$$y = y_i + (y_{i+1} - y_i)\frac{x - x_i}{x_{i+1} - x_i} + H_i \sin\left(\frac{\pi(x - x_i)}{x_{i+1} - x_i}\right) \qquad \text{(equation 2.3.7)}$$

which has the following properties:

1) it goes through the inflection points at the correct heights y_i and y_{i+1};
2) it has zero curvature at the inflection points;
3) it has the correct upward offset from the line connecting the inflection points.

A distinction has to be made between long-wavelength and short-wavelength profile overbends. An overbend section of a profile between inflection points at $x_i$and x_{i+1} is

$$\text{'long'} \quad \text{if} \quad L_i > \left(\frac{35FH_i}{w}\right)^{1/4} \qquad \text{(equation 2.3.8)}$$

$$\text{'short'} \quad \text{if} \quad L_i \le \left(\frac{35FH_i}{w}\right)^{1/4} \qquad \text{(equation 2.3.8)}$$

where w is the pipeline weight per unit length (submerged if the pipeline is under water), but does not include any backfill.
A profile can of course include both long and short overbends.

If an overbend is long, the pipeline will be supported by a foundation, and the required downward force to hold the line in position on the hill between inflection points i and i+1 is

$$q = \min\left(NeH_i\left(\frac{\pi}{L_i}\right)^2 - FH_i\left(\frac{\pi}{L_i}\right)^4, \frac{Ne^2H_i}{4F}\right) \qquad \text{(equation 2.3.9)}$$

Short overbends are considered in (2.3.4) below.

Equation 2.3.9 applies only in the elastic range. If the profile contains imperfections large enough for the pipe to yield plastically, a numerical elastic-plastic analysis needs to be carried out.

2.3.4 Stabilisation in a changing profile

A complication is that if the profile has short steep-sided hills, the pipeline may form spans on either side. This situation is illustrated in Figure 2.6. The pipeline has lost contact with the sides of the hill, and in the extreme case it is supported only at a single point. The horizontal extent of the hill then becomes irrelevant, and only the height matters. Pipe profiles of this kind are likely to occur when a line is laid across boulders or reefs, or when lumps of spoil fall into a trench behind trenching equipment. The flexural rigidity F determines the sharpness of the curve the pipeline takes up at the top of the hill.

Equation 2.3.8 defines the condition for an overbend profile to be treated as short. The amount of backfill required depends on whether there is infill material under the pipe. If there is no infill, the spans can shorten during backfill or during subsequent operation, and the overbend will then become sharper, which in turn increases the amount of backfill required.

The most conservative assumption is that there is no infill under the pipe. This is the recommended assumption, unless it can be positively demonstrated that there is infill under the pipe and that it is geotechnically competent to provide resistance to downward movement. In that case, the required downward force is;

$$q = \frac{NeH_i}{4F} \qquad \text{(equation 2.3.10)}$$

which is independent of the overbend length L_i. The bending moment at the crest of the overbend can be estimated from

$$M = 0.88\ NeH_i \qquad \text{(equation 2.3.11)}$$

If it is certain that the pipe is supported, then the required downward force can be calculated from equation 2.3.9, and the bending moment at the overbend crest can be determined from:

$$M = 13.1\ Fh_i/L_i^2 \qquad \text{(equation 2.3.12)}$$

In preliminary design, when seabed data are not available, it is often convenient to assume prop imperfections in the profile, such as those that might be produced by boulders or by soil fall-back during trenching. If the height of such a prop imperfection is Δ, measured from a horizontal level representing the seabed or the trench bottom, the corresponding value of H (see Figure 2.7) is given by:

$$H = (11/27)\Delta \qquad \text{(equation 2.3.13)}$$

and L is calculated from

$$L = \left(\frac{35FH}{w}\right)^{1/4} \qquad \text{(equation 2.3.14)}$$

Equation 2.3.9 to equation 2.3.14 inclusive apply only in the elastic range. If the profile contains sharp steep-sided imperfections large enough for the pipe to yield plastically, a numerical elastic-plastic analysis needs to be carried out.

2.3.5 Additional forces due to flow

An additional upward force on a pipeline in an overbend may be induced by the flow itself. If the mass flow rate (mass/unit time) in the pipeline is m, the mean velocity is U, and the curvature is d^2y/dx^2 (negative in an overbend), the flow exerts a force $mU(-d^2y/dx^2)$ per unit length. This force may be significant in oil pipelines with high flow velocities, and in two-phase lines subject to slugging. The mass flow rate and velocity are then the values in the slug, and not the mean values.

Since the calculation is dynamic, it must use consistent units. If m is measured in kg/s, U in m/s, and d^2y/dx^2 in m^{-1}, the units of $mU(-d^2y/dx^2)$ are N/m. The effect is normally quite small. If, for example, m is 150 kg/s (which corresponds to 100,000 b/d of 810 kg/m^3 oil), U is 1.5 m/s, and the curvature d^2y/dx^2 is 0.02 m^{-1} (which corresponds to a 50 m overbend radius), the force per unit length is (150)(1.5)(0.02) = 4.5 N/m.

2.3.6 Summary: required downward force required for stability

Equation 2.3.8 defines which overbends are long and which are short. Equation 2.3.9

governs long overbends. Equation 2.3.10 through equation 2.3.14 inclusive govern short overbends.

These calculations inescapably require the engineer to assess the profile of the foundation on which the pipe is resting, or, if it is available, the profile of the pipeline itself. There is no way of assessing the risk of upheaval without thinking about how uneven the profile is.

If the sharpest overbend features of the profile are relatively long hills, the engineer should use equation 2.3.9 to determine the necessary downward force. Seabed megaripples and sandwaves give profiles of this type.

Boulders and lumps of soil produced by trenching give profiles with isolated high spots, so that spans are formed and the pipe is not continuously supported. The engineer should then use the approach set out in (2.3.4). He has to consider whether the pipeline will remain in the configuration induced by the profile it is resting on, or whether it can move as a result of stabilisation measures.

2.3.7 Pipe out-of-straightness

Pipeline structural analysis normally assumes that the pipe is straight if it has no externally-applied bending moment. However, some construction techniques such as reeling may leave the pipe out-of-straight, and this can be an additional factor which makes the pipelline more vulnerable to upheaval buckling, if out-of-straightness creates an overbend in the pipe profile.

In equation 2.3.3 and equation 2.3.6, the first Ne term on the right is the actual curvature of the pipe. Out-of-straightness may contribute to that curvature. The second F term is derived from the change of curvature from the initial state in which the pipe is free of bending moment. If out-of-straightness is present, and the initial form of the pipe is known,

$F\dfrac{d^4y}{dx^4}$ in (equation 2.3.3)

should be replaced by $F\dfrac{d^4}{dx^4}(y - y_i)$,

where y_i describes the profile the pipe would have if it were moment-free. In circumstances that arise in practice, the F term is usually small in comparison with the Ne term.

2.3.8 Safety factors

The calculation methods described above do not include any implicit factors of safety. The recommended safety factor to be applied to the total external downward force required for equilibrium is 1.5. If the profile is known to a high degree of accuracy, or can be accurately controlled, as in many onshore pipelines, the factor may be reduced. If the profile is uncertain and cannot be measured accurately, the factor may need to be increased.

2.4 AVAILABLE FORCE TO RESIST UPWARD MOVEMENT

If the pipe is not trenched and not buried, the force per unit length available to resist upward movement is w, the submerged weight per unit length in the operating condition with account being taken of contents weight. An untrenched pipeline usually buckles laterally rather than upwards: lateral buckling is discussed in (3.3). A pipeline which is trenched in a V-shaped trench, but not buried, usually buckles up the side of the trench.

A pipeline buried under soil or rock has a resistance to upward movement provided by both the weight and shear resistance of the cover. This uplift resistance has been investigated in a number of test programmes, some of them at full scale.

The simplest case, and the one that most commonly occurs in practice, is cover by a cohesionless material such as sand, gravel, or rock fragments. The experiments show that the uplift resistance is given by

$$r = \gamma HD\left(1 + f\frac{H}{D}\right) \qquad \text{(equation 2.4.1)}$$

where

r is uplift resistance per unit length of pipeline;
γ is the unit weight of the soil above the pipeline; under water the submerged weight is used;
H is the cover, from the top of the pipe to the soil surface above the pipe centreline;
f is an uplift resistance coefficient determined empirically.

The uplift resistance coefficient f is a simple representation of a complex geotechnical situation, and must depend on the voids ratio, dilatancy, compaction and initial state of stress of the cover. Extensive measurements (8.4) show that for rock, gravel and dense sand f is 0.5 or more, and this value may be used in design. In loose sand, however, the uplift coefficient is sometimes much lower, and values as low as 0.15 have been observed. The reasons are not fully understood: it is probably due to the tendency for very loose soil structures to collapse as the upward movement of the pipeline forces them to shear, and for positive pore pressures to develop, in turn leading to low effective stresses in the sand above the pipe. It is unfortunate, because loose sand cover is likely to occur if a pipeline trench is naturally backfilled by sand swept into it by seabed sediment transport.

If the soil surface above the pipe is not level, the cover is defined in the way shown in Figure 2.8. Lines inclined outwards at 30° to the vertical are drawn from the 3 and 9 o'clock positions at either end of the horizontal diameter of the pipeline. The lines intersect the surface at points A and B. The cover H is the smaller of:

1. the vertical distance from the top of the pipe to line AB;
2. the vertical distance from the top of the pipe to the soil surface immediately above the top of the pipe.

Another common case is for a pipeline to be covered by a mixture of clay and cohesionless sand or silt. This occurs when the seabed consists of a thin layer of sand overlying clay. If the pipeline is trenched by ploughing, the trench is subsequently backfilled with the excavated material by a backfiller, the backfill in place consists of irregular lumps of cohesive clay in a matrix of loose sand. A limited number of tests have been carried out on mixtures of this type, and show that mixtures of sand and clay lumps should be treated as sand and not as clay. The practical implication is that the backfill does not gain additional strength - at least not in the short term - from the strength of the clay, which is usually higher than the strength of the sand.

Cover by continuous clay is uncommon, because clay does not usually naturally backfill into pipeline trenches, and because backfilling with clay produces a lumpy discontinuous cover. If this case occurs, specialist geotechnical advice should be obtained.

The calculations described in this section do not include any implicit factors of safety.

Factors of safety on required total external downward force q are discussed in (2.3.8), and allow for uncertainty in uplift resistance.

2.5 COMPARISON

The external vertical force per unit length required to hold the pipe in position is q calculated from (2.3), taking account of the safety factor described in (2.3.8).

The vertical force per unit length available to hold the pipe in position is w + r, where w is the submerged weight and r the uplift resistance from (2.4).

If

$$w + r > q \qquad \text{(equation 2.5.1)}$$

the pipeline is stable, and no further action is necessary.

If the pipeline appears to be marginally unstable, it may be appropriate to carry out a more refined analysis by a more sophisticated method, as explained in (5).

If the pipeline is not stable, the situation if upheaval does occur should be assessed, and the pipeline's fitness for purpose should be reviewed. Several countermeasure options are available. They can be used alone or in combination, and range from radical changes in design to minor corrective measures taken after the pipe is in place. An earlier study considered some 40 options, and others have been identified since then, but this DEP confines itself to the options that appear most useful.

The leading options are:

1) to reduce the driving force;

2) to make a radical change in design;

3) to leave the pipeline untrenched and accept that it will buckle laterally;

4) to stabilise the overbends by placing rock or mattresses over them.

These options are the preferred ones, and the engineer should consider all four options, though some of them may be rejected at once. Those four options are examined in (3). Other options that may be useful in unusual cases are examined in (4).

3. PRIMARY OPTIONS TO ELIMINATE RISK OF UPHEAVAL BUCKLING

3.1 REDUCTION IN DRIVING FORCE

3.1.1 Changes in operating parameters

The first option is plainly to reduce the operating pressure p and the temperature increase Δ. The costs of preventing upheaval buckling are high and should be considered when defining the optimum project parameters which include pipeline operating pressures and temperatures. However, it may be occasionally possible to reduce them, for example by omitting external insulation to reduce the temperature, or by taking action to reduce the design pressure.

Design must not be based on overconservative choices of operating parameters.

3.1.2 Reduction in wall thickness

The temperature term in the longitudinal force equation is proportional to the wall thickness t. This indicates that it is advantageous to **reduce** the wall thickness to the minimum possible. This is true even though there is another effect on the flexural stiffness F, which appears later in the analysis and is proportional to t. The adverse effect on F is outweighed by the beneficial effect on the longitudinal force Ne.

Reduction in wall thickness is a major topic in design generally, because it almost always reduces costs. The principal ideas that should be considered are:

1. increase in steel grade
   higher-strength steels up to X80 are now readily available at little or no cost penalty, and can be welded without difficulty. There is active work on stronger steels up to X150 or higher, but they may be costly and there are welding problems;
2. selecting a higher design factor (see DEP 31.40.00.10-Gen. for maximum allowable factors);
3. adopting allowable-strain design.

Codes formerly imposed a limit on equivalent stress, intended to prevent the pipeline from yielding under longitudinal compressive stress induced by temperature increases. This limitation governed the wall thickness for lines that operate at high temperatures, and led to substantial increases in wall thickness. Research in recent years has shown that the traditional requirement on equivalent stress may be replaced by a new and in practice much less restrictive condition on strain (8.5). This option is now accepted by several design codes.

3.1.3 Increased residual tension

Another option is to increase the residual tension T_R. The as-laid tension is the horizontal component of the laybarge or reelship tension applied at the surface. That tension can be increased, but there are practical limitations, among them:

1. possible external coating damage;
2. possible limitations of the mooring or DP (dynamic positioning) system of the vessel;
3. a long distance between the vessel and the touchdown point.

In practice increases in applied tension are usefully significant only for small-diameter lines. For example, the relatively small line used in the numerical example in (2.2) would normally be laid with an applied tension of the order of 500 kN, which is only one-fifth of the longitudinal force induced by the operating conditions. In a large pipeline the longitudinal force induced by the operating conditions might be 10 MN, in comparison with which a residual tension of 0.5 MN is only marginally useful.

3.1.4 Increased flexibility

The longitudinal compressive force can be reduced from its fully constrained value by allowing expansion movements to occur. This can be accomplished by expansion doglegs, or by expansion loops (at the ends, or as midline expansion spools), by laying the pipeline in a snaked or zig-zag configuration, or by laying the pipeline in a curve and allowing it to move outward on the curve, or sometimes by allowing the pipe to buckle laterally. The effect will only occur if the pipeline is able to move. The movements must be calculated to make certain that excessive bending or torsion does not occur in the pipeline, particularly if a nearby section of the pipeline is anchored or constrained against lateral or longitudinal movements.

Lateral buckling is discussed further in (3.3). Zig-zag and snaked configurations are discussed further in (4.3).

3.2 ALTERNATIVE PIPELINE CONCEPTS

A radical response to an upheaval problem is to change the design completely. All radical options have wider implications beyond upheaval, and those implications naturally have to be considered.

One option is to make the pipeline a flexible. Flexibles are subject to upheaval buckling (8.6), but their upheaval is principally determined by the pressure term in the driving force equation and the temperature plays little part. The magnitude of this term is heavily dependent on the details of construction of the flexible, and the propensity to buckle can be reduced by detailed design. The driving force Ne and flexural rigidity F can most reliably be determined by the Manufacturer, and can then be put into the governing equations.

Upheaval can be countered by detailed design and by trenching the line under pressure. The line is laid and then pressurised, so that it buckles laterally, and next trenched while under pressure. Depressurisation leaves a residual tension in the line. Repressurisation to the trenching pressure does not cause any tendency to buckle further.

Another option is to incorporate the line into a bundle. Most bundles are constructed within a carrier pipe, and connected to it at the ends by stiff bulkheads. The internal lines carry pressure, and usually operate at a higher temperature than the sea. The annular space is generally pressurised, but to a much lower pressure than the internal lines, and is at or close to the sea temperature. If the internal lines and the carrier were all free to expand longitudinally, the internals would expand more that the carrier. Since the bulkheads prevent relative movements at the ends, the internal lines are put into compression and the carrier into tension. The bundle as a whole expands longitudinally, but its expansion is resisted by seabed friction.

The resultant force across the bundle as a whole is compressive, but buckling does not generally occur because of the high flexural rigidity provided by the carrier. In addition, bundles in carriers are not generally trenched, so that they would buckle sideways rather than upwards.

More commonly, one or more of the internal lines may be designed to deflect laterally within the carrier. The extent of lateral deflection depends on the distance between spacers and bulkheads. A strain criterion may be applied as lateral buckling is self-limiting.

3.3 LEAVING THE PIPELINE UNTRENCHED AND ALLOWING LATERAL BUCKLING

Buried and trenched pipelines buckle upwards, because they can more easily move upwards (against their own weight and the uplift resistance of the cover) than sideways or downwards (against the much greater passive resistance of the soil beneath and to the side). An unburied untrenched pipeline, on the other hand, buckles sideways more easily than upwards, because if it moves sideways it only has to overcome the sliding lateral resistance of the soil, which is almost always less than the resistance to upward movement (8.7) and (8.8).

Lateral buckling frequently occurs in untrenched submarine pipelines and often goes unrecognised, because the movements occur over relatively long distances and are not accompanied by localised distress (8.9 and 8.10). Lateral buckling is often harmless, and even beneficial if it relieves longitudinal forces that might otherwise lead to upheaval elsewhere. It is often possible to demonstrate by calculation that though the pipeline is in a technical sense ‘buckled’ it is not at risk from collapse, local buckling of the pipe wall, low-cycle fatigue or rupture.

The interaction between the longitudinal compressive force and local impact from fishing gear may trip a small-diameter pipeline into a buckled configuration, and unacceptably large deformations may occur at the impact point. Lateral buckling is a difficult subject, outside the scope of this DEP, and research is continuing (8.9).

3.4 SELECTIVE ROCK DUMP OR MATTRESS STABILISATION ON IDENTIFIED CRITICAL OVERBENDS

Buckling occurs at high points in the pipeline profile. A straightforward option is therefore to hold down the pipeline at the high points, by placing rock over it. The required total downward force is determined by the equations in (2.3). The submerged weight provides part of it, and the remainder is supplied by the uplift resistance of the cover rock. The relationship given in (2.4) relates the uplift resistance to the depth of cover and the rock properties.

The crux of the problem is the ability to locate and measure the critical overbends confidently. The measurement problem is discussed in (6).

The method discussed in this Section uses much less rock than continuous rock dump, see (4.1), and so the engineer can afford to choose the depth of cover generously.

Rock alone is not very efficient, because it is difficult to place it precisely and because the uplift resistance corresponds to the submerged weight of the volume of rock between two inclined planes which leave the pipe at the 3 and 9 o'clock positions and are inclined outwards at about 30° to the vertical. The rock outside those planes does not directly contribute to the uplift resistance, but is needed to support the rock between the planes. The effectiveness of rock dumping can be much enhanced by laying a structural geotextile across the pipeline before placing the rock. The rock on either side then holds down the geotextile, so that as the pipeline starts to move upward the geotextile develops tension which adds to the uplift resistance. Laboratory scale tests show that even a modest width of geotextile at least doubles the uplift resistance. A simple calculation shows that the additional uplift resistance r' secured by a total breadth b of geotextile is approximately

$$r' = \gamma(\mu + \mu')\ (H + D)\ \left(b - \left(1 + \frac{\pi}{2}\right)D\right) \qquad \text{(equation 3.4.1)}$$

where

- r' is the additional uplift resistance per unit length;
- γ is the submerged unit weight of the rock;
- μ is the coefficient of friction between the rock and the upper surface of the geotextile;
- μ' is the coefficient of friction between the seabed and the lower surface of the geotextile;
- b is the breadth of the geotextile (measured transverse to the pipeline);
- H is the cover (measured from the top of the pipe, as described in (2.4));
- D is the pipe outside diameter,

and the weight of the geotextile itself is neglected. This equation probably underestimates the additional uplift resistance, because incipient movement of the geotextile increases the horizontal compressive stress in the rock above the pipeline and therefore increases its resistance to shear: this effect still has to be fully investigated.

This option has not been applied underwater to suppress upheaval, as far as is known, but geotextiles have been used underwater. The geotextile has to retain a strength of 0.5 r' per unit length over a long period in seawater, without significant extension in creep. There has been much research on the long-term strength of geotextiles, mostly for civil engineering applications with much longer design lives.

Some contractors have found that they can place mattresses over a pipeline as cheaply as dumping rock over it. This can be interpreted as another application of a geotextile, this time heavy rather than light. The full weight of a mattress is only applied to the pipeline after a considerable upward movement. A simple model treats the mattress as infinitely flexible, inextensible and uniform in weight. The uplift resistance when upward movement begins is:

$r = (1/2\pi+1)\, D' w$ (equation 3.4.2)

where

- r is uplift resistance per unit length of pipe;
- w is the weight per unit area of the mattress;
- D' is (outside diameter of the pipe + mattress thickness).

4. FURTHER DESIGN ALTERNATIVES

4.1 CONTINUOUS ROCK DUMP

If the critical overbends cannot be identified confidently, a practicable but expensive option is to rock-dump the whole length. This has been done on a few occasions. The engineer has to decide on the uplift resistance from an assessment of the profile, based on whatever data can be obtained. The cost is highly sensitive to the precise cover requirement.

Cost analyses have shown that if continuous rock dumping is selected it can be more economical to lay the pipe, trench it and then to dump rock over the trenched pipeline, rather than to rock-dump over the pipeline on the natural seabed. This is because the savings from the reduced quantity of rock needed to secure a specified cover over a pipe in a trench more than outweigh the additional cost of trenching.

4.2 ALLOWING THE PIPE TO BUCKLE AND THEN STABILISING UNACCEPTABLE UPHEAVALS

It has been argued that the most economical option is to do nothing to prevent upheaval buckling, to put the line into operation, and then to stabilise any sections that show upheaval. The extent of deformation during upheaval can be calculated using finite-element programs, see (5.2). If the plastic strains that develop in the pipe are acceptable, the pipeline has not suffered any loss of integrity, and it can remain in service. If the upheaval leaves a raised loop of pipeline above the seabed, the loop can be stabilised and protected by careful rock dumping.

No operator is known to have consciously adopted this strategy as a matter of policy, but many operators have stabilised buckles after they have occurred. The environmental, contractual, cost, political and risk implications of deliberately choosing not to take action until a buckle has occurred clearly need to be examined very carefully.

4.3 ZIG-ZAG PIPELAYING

Another option is deliberately to construct the line out of straight, in order to encourage it to flex sideways. This is normally only an option for untrenched lines, because a trenched line cannot move significantly without moving out of the trench.

An ambitious application of this concept was by Shell Oil in the Mobile Bay project, which is in very shallow water. Each 12 m length of pipeline had a central 8° bend, and the lengths were welded together on the laybarge and lowered over a specially-constructed stinger into a wide trench. In operation the pipeline flexed sideways at the vertices of the zig-zag. This option is practicable, but it has several disadvantages:

- the cost of bending each length of pipe;
- the added complexity of line-up, because the pipe cannot be rotated to minimise hi-los due to ovalisation;
- the added difficulty of laying, because of the added width of stinger needed, and the difficulty of applying tension.

These factors make this option unattractive except in shallow water where tension is not required. Alternatively, the zig-zag bends can be introduced by hydraulic jacking on the laybarge.

Another option is to zig-zag or snake the pipe during laying. This happens anyway, because a laybarge does not hold a perfectly straight course, and there are excursions away from a perfect line or curve. Just as vertical profile excursions of the order of 0.1 m have a significant effect on upheaval, so must horizontal excursions of the same order have a significant effect on lateral buckling. They are much too small to be detected by conventional survey methods, but can be picked up by the geometry pig described in (6). The data required for design are easily extracted from geometry pig surveys carried out for other purposes.

4.4 ROUTE SELECTION

A pipeline laid along an uneven profile is much more subject to upheaval than a pipeline laid along a smooth profile. Long gentle variations in height, however large they are, have much less effect than short sudden variations in height. It may be possible to reduce the problem by careful route selection, both on the macroscale and the microscale, and to avoid features such as:

1. pockmarks and ploughmarks;
2. sandwaves;
3. megaripples;
4. boulder fields;
5. coral reefs, pinnacles and hummocks.

4.5 PROFILE SMOOTHING

A pipeline route can be smoothed by "pre-sweeping" dredging. This is sometimes done to reduce spans, but is an expensive option.

An alternative is to smooth the pipeline profile during trenching. Most trenching operations leave the profile of the base of the trench smoother than the original seabed profile, and eliminate short-wavelength irregularities. Given good data about the seabed profile, it is possible to control the trenching depth so as to trench more deeply on the hills and less deeply in the valleys, within the limitations of the trenching equipment.

Another alternative is to carry out an as-laid or as-trenched survey, and then to return to smooth the profile by additional trenching at high points, possibly as a combined operation with span correction.

4.6 PREHEATING, LATERAL BUCKLING AND SUBSEQUENT TRENCHING

A further option is to heat the line so that it buckles laterally, and then to trench it in the buckled position. If the heating is to a high enough temperature, the additional longitudinal force during operation is not enough to cause upheaval from the trenched position.

An application of this method to the Glamis pipeline in North block 16/21a is described in a paper by Craig (8.11), which is a useful case study. Action to forestall buckling on the Glamis line was taken because a buckle had been observed on a Balmoral flowline nearby. The heating option was economically attractive because of the availability of 2000 m^3/d of produced water from the Balmoral field, which leaves the separator at 60 °C. Additional heating raised the temperature at the upstream end of the flowline to 70 °C. The line buckled laterally in 14 places, generally over a distance of between 30 m and 40 m, with a maximum lateral movement of 1.9 m. It was then trenched. Further surveys were carried out after the line had been put into operation, and observed movements of 1 diameter or less, except in one place where the pipe had moved 0.3 m up the side of the trench. It was concluded that upheaval movements would certainly have occurred if no action had been taken, and that the countermeasures were successful.

In this instance, the cost of the alternative, of continuous rock dump over the whole line was estimated at GBP 2 million to GBP 2.5 million (1988). The cost of hot water flushing was GBP 0.5 million, which is plainly highly cost-competitive. The choice of method was heavily influenced by the availability of hot water.

4.7 INCREASING THE WEIGHT OF THE PIPELINE

A possible option is to secure resistance to upward movement by the weight of the pipeline itself, and therefore simply to make the pipeline heavier. This is significant in marginal cases, but is not an efficient way of stabilising a whole pipeline.

5. REFINED ANALYSIS

5.1 INTRODUCTION

The simple assessment method described in (2) determines the external force required to hold the pipeline in place in a given profile, and compares it with the available weight and cover uplift resistance.

Two distinct kinds of problems occur. The first occurs when a new pipeline is being designed, and the engineer wishes to know what provisions need be made for upheaval buckling. By definition, the pipeline is not in place, and the engineer has to assess what its in-place profile will be. The second problem occurs when the pipeline is in place and perhaps in operation, the line profile is available, and the engineer wishes to assess the profile to see if upheaval will occur.

The assessment method given in (2) is not a complete analysis of either kind of upheaval problem, because it assumes that the pipeline profile is known and that the pipeline does not move away from its initial position. In reality, if the pipeline begins to move, the terms in the governing equations change (because the local curvature changes), and the uplift resistance alters, because some pipeline movement is needed to mobilise it. Moreover, the pipeline usually becomes plastic after relatively small movements, and then the elastic analysis of (2) becomes only partly valid.

5.2 FINITE-ELEMENT ANALYSIS

A complete analysis uses a finite-element program. It should be carried out for marginal cases where the simple analysis shows the pipe to be slightly unstable, or when the margin of safety has to be quantified. A finite-element program for analysis of upheaval buckling should include the following features:

- the possibility of analysing an arbitrary seabed profile;
- plastic deformation of the pipe;
- allowance for mobilisation distance for uplift resistance.

6. MEASUREMENT OF SEABED AND PIPELINE PROFILES

Upheaval buckling is caused by the interaction between the pipeline longitudinal force and the pipeline profile. The profile is central to the problem, and there is no substitute for factual quantitative data.

The analysis should assemble all available information about the roughness of the seabed. The crucial parameters are the height and breadth of peaks in the profile.

Sources of data include:

1. ROV (Remote Operated Vehicle) surveys of previous pipelines in the same area;
2. accurate bathymetry in the same area;
3. interpreted side-scan sonar.

These data can then be used to estimate a critical imperfection height H and a corresponding distance between inflection points L, for the most severe seabed roughness components that are judged likely to appear.

Profiles can be measured from a ROV, which moves along the pipeline. The ROV system measures the ROV height above the pipeline and the seabed by acoustic profiling or magnetic measurements, or by running with wheels on the pipeline. The ROV measures its depth below the surface by precision high-pressure transducers or by acoustic measurements of the position relative to a support vessel.

Profile measurements for upheaval assessment make demanding requirements on system accuracy. It is pointless to carry them out if the required accuracy cannot be achieved. The requirements are different from most bathymetric surveys. Absolute depth has little or no importance, and neither do depth changes over long distances of the order of 1000 m. Small depth changes over short distances are much more important: in a typical pipeline system, height changes of the order of 0.2 m over horizontal distances of the order of 10 m have to be measured precisely.

A new and probably better way of measuring pipeline profiles is by intelligent geometry pig systems (8.12). The pig carries an extremely sensitive inertial navigation system incorporating gyroscopes and accelerometers, which measure the acceleration of the pig. The acceleration is then integrated twice to determine the profile. Accuracy and repeatability of better than 0.1 m are routinely achieved.

Equation 2.3.3 shows that the critical profile parameters are vertical curvature and its second derivative. If the profile is measured by an ROV as a sequence of heights referred to a datum, the profile has to be differentiated twice to obtain the curvature and twice more to obtain the second derivative of curvature. This can either be done by numerical differentiation or by fitting a mathematically-defined spline curve to the points and then differentiating the curve analytically. High-order differentiation is inherently inaccurate, whichever of the two methods is used, because the derivatives are extremely sensitive to small changes in the height of one point.

Programs are available to process this kind of survey data and identify the locations of critical overbends and the amount of cover required to stabilise them.

Measurement by geometry pig is better in principle. If the pig moves at a constant horizontal speed, the vertical acceleration measured by its accelerometers is directly proportional to the vertical curvature of the profile. It follows that the curvature in the principal term in equation 2.3.3 is measured directly and does not depend on differentiation. The curvature still has to be differentiated twice to secure the less important second term, but the associated errors are smaller.

7. REFERENCES

In this DEP, reference is made to the following publications:

NOTE: Unless specifically designated by date, the latest edition of each publication shall be used, together with any amendments, supplements or revisions thereto.

SHELL STANDARDS

Index to DEP publications and standard specifications	DEP 00.00.05.05-Gen.
Pipeline engineering	DEP 31.40.00.10-Gen.

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NOTE: The following documents are for information only and do not form an integral part of this DEP.

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FIGURES

FIGURE 1.1	EXAMPLES OF PIPELINE UPHEAVEL
FIGURE 1.2	ALGORITHM FOR ASSESSMENT AND ACTION
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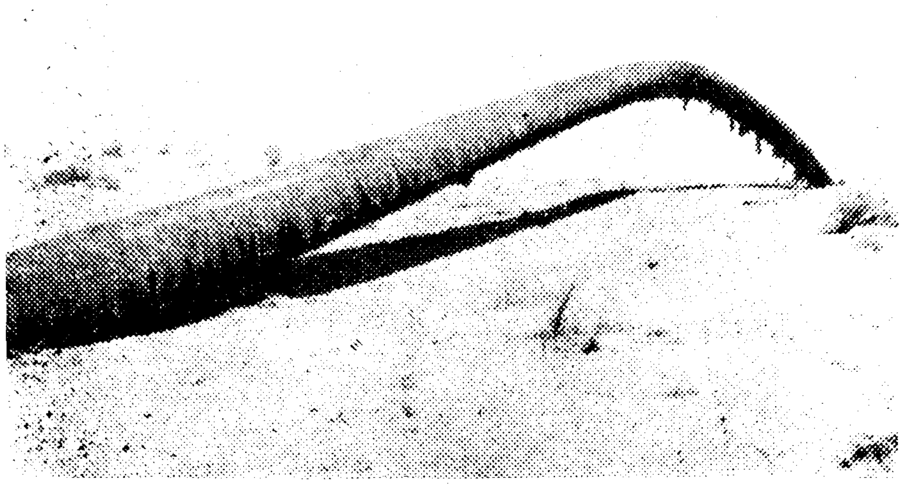
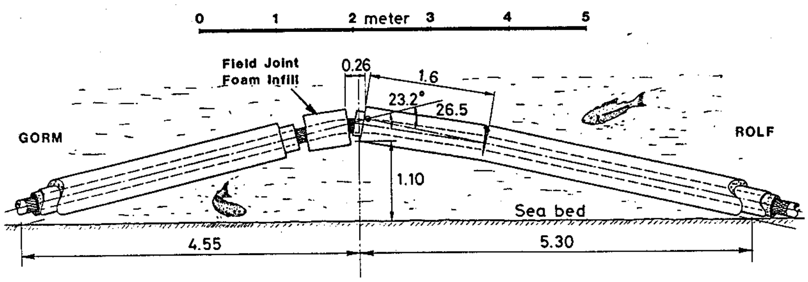
FIGURE 1.1 EXAMPLES OF PIPELINE UPHEAVAL



(a) 40-inch pipeline



(b) 8-inch Rolf to Gorm pipeline from OTC 6488 (1)

FIGURE 1.2 ALGORITHM FOR ASSESSMENT AND ACTION

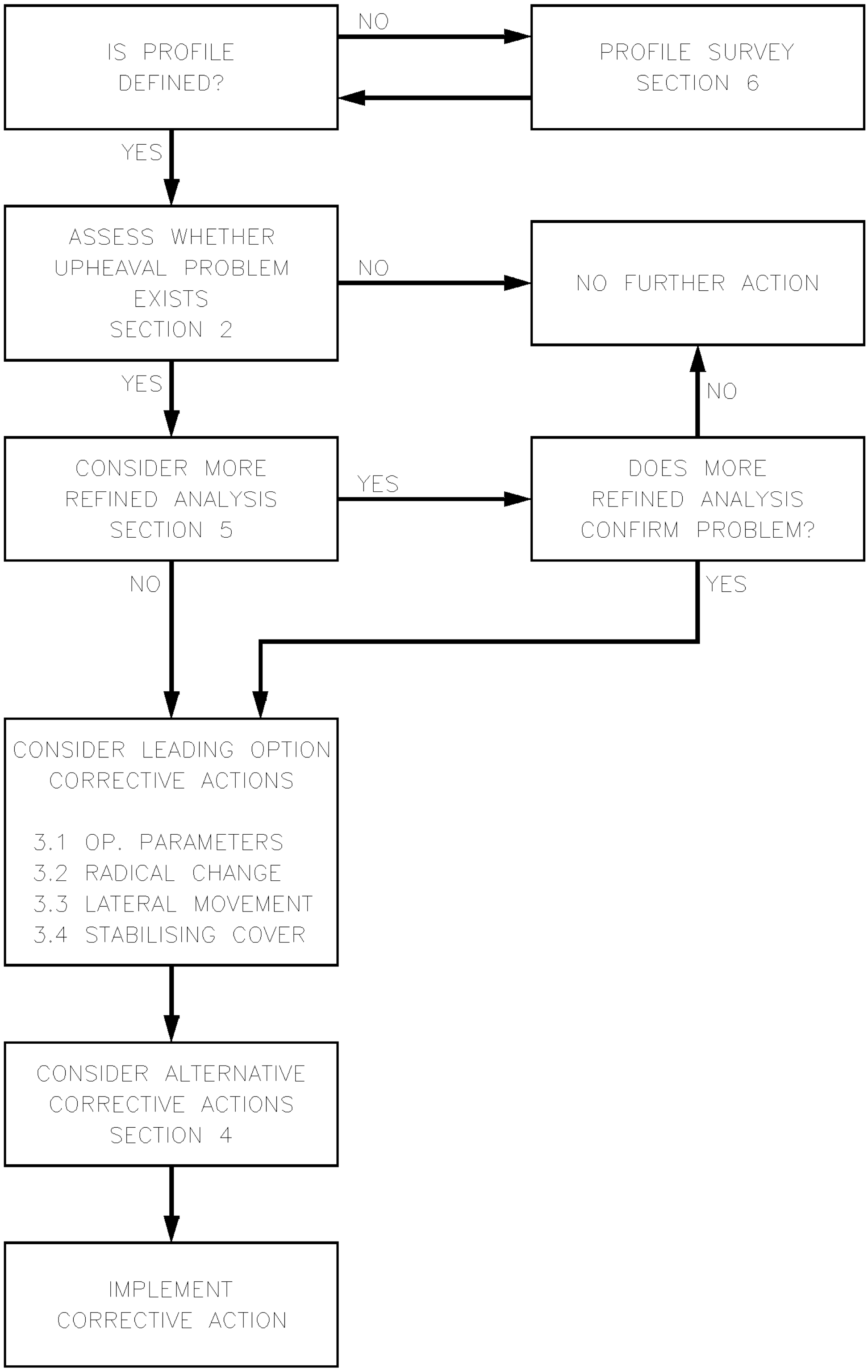
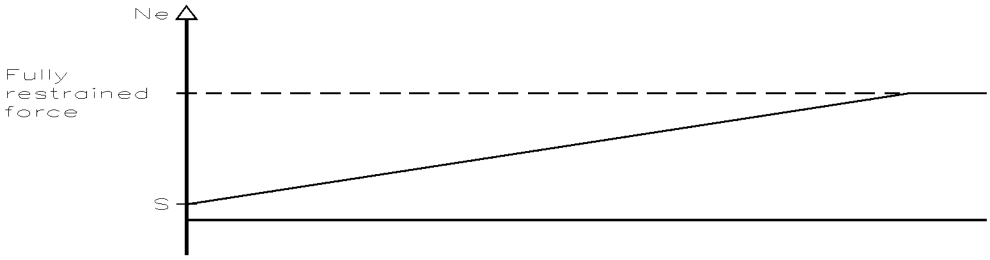


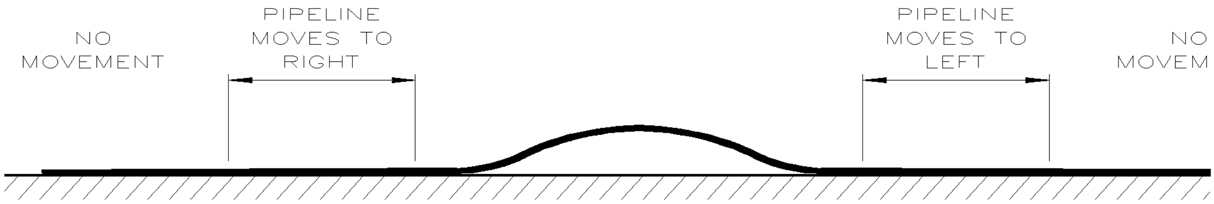
FIGURE 2.1 VERTICAL RISER CONNECTED THROUGH AN ELBOW TO A SEABED PIPELINE

(a) Schematic

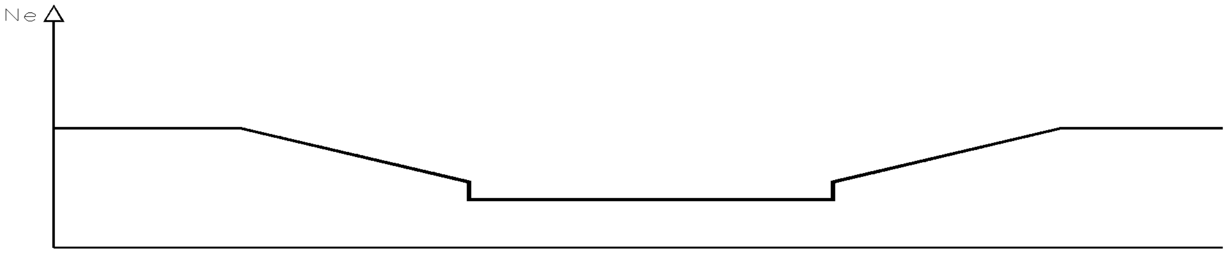


(b) Variation of longitudinal compressive force

FIGURE 2.2 EFFECT ON THE LONGITUDINAL FORCE IN THE PIPELINE



(a) Schematic



(b) Variation of longitudinal compressive force Ne

FIGURE 2.3 PIPELINE UNDER AXIAL LOAD Ne AND VERTICAL LOAD INTENSITY Q

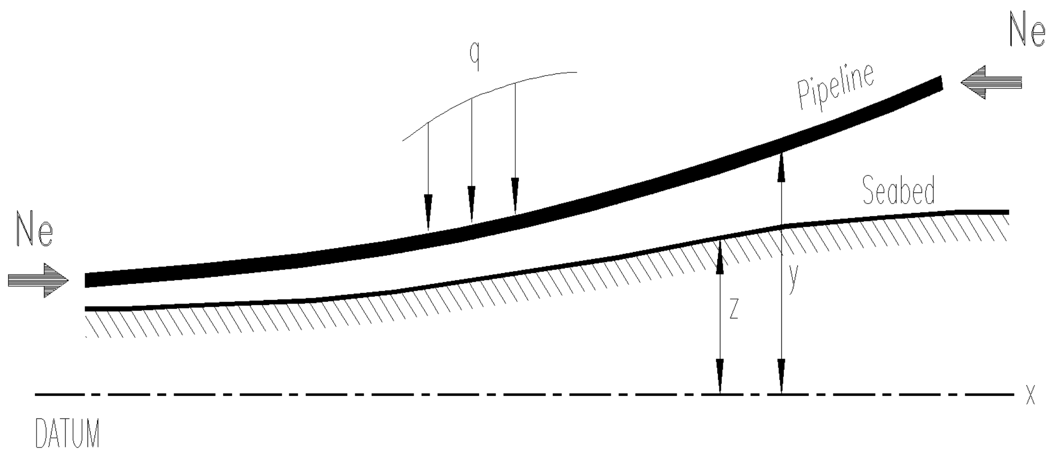


FIGURE 2.4 SINUSOIDAL PROFILE

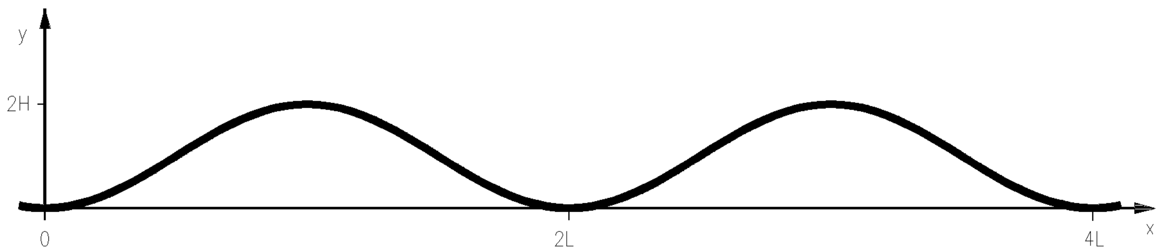


FIGURE 2.5 ARBITRARY PROFILE

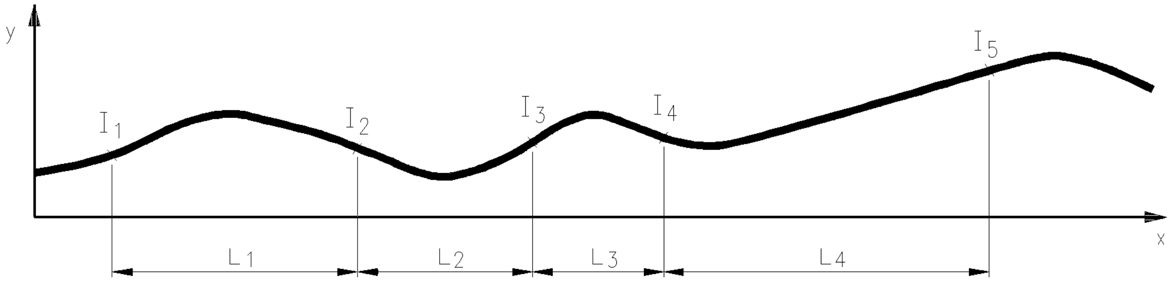


FIGURE 2.6 PROFILE WITH HILL TOO SHORT FOR PIPELINE TO CONFORM

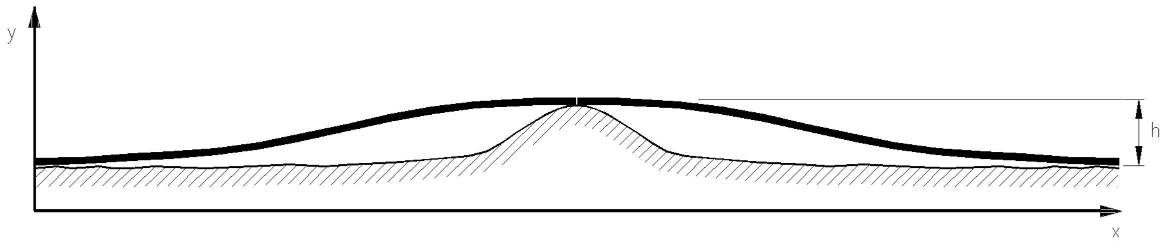


FIGURE 2.7 ASSUMED PROP INPERFECTION

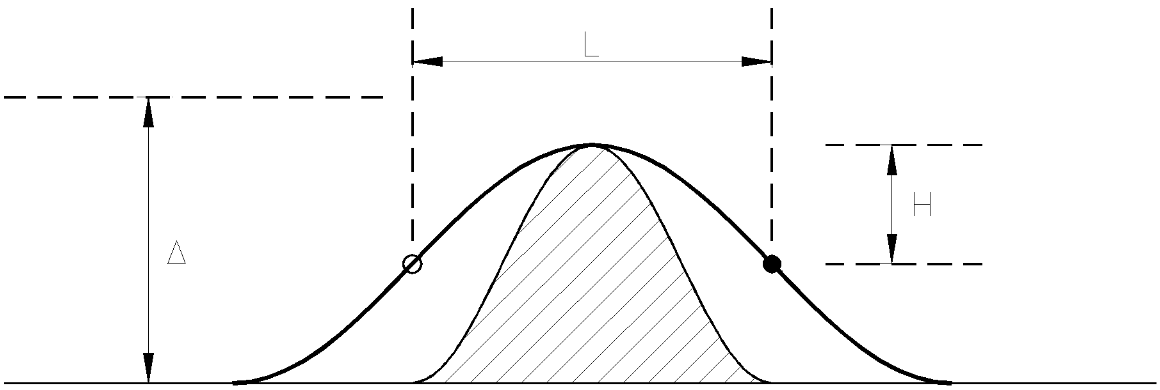
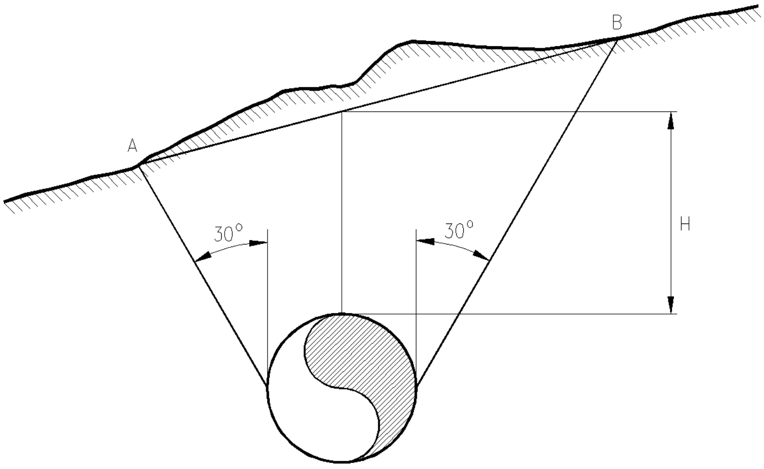
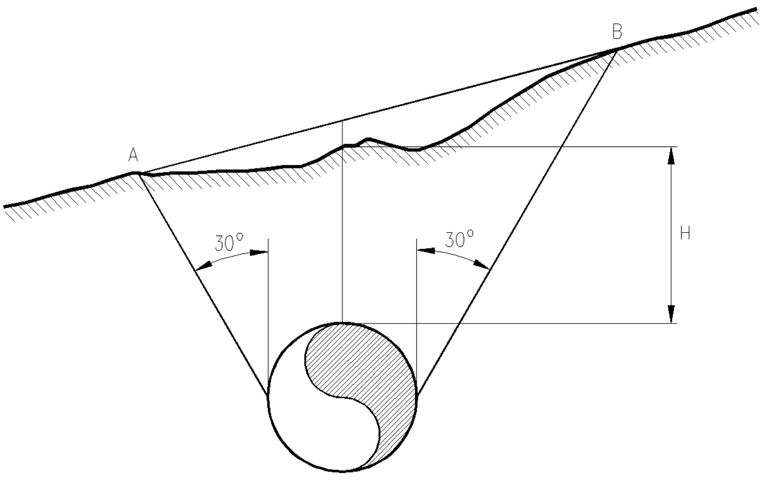


FIGURE 2.8 DEFINITION OF COVER HEIGHT



(a) Cover surface higher than AB



(b) Cover surface lower than AB