OTC 6335

Design of Submarine Pipelines Against Upheaval Buckling

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This paper was presented at the 22nd Annual OTC in Houston, Texas, May 7-10, 1990.

ABSTRACT

This paper describes part of a comprehensive joint-industry project on upheaval buckling. It develops a semi-empirical simplified design method and detailed design methods based on a new numerical analysis, and illustrates their application by examples. It assesses alternative design strategies, and the implications of strain-based design.

INTRODUCTION

When a pipeline is operated at a temperature and pressure higher than ambient, it will try to expand. If the line is not free to expand, the pipe will develop an axial compressive force. If the force exerted by the pipe on the soil exceeds the vertical restraint against uplift movement created by the pipe's submerged weight, its bending stiffness, and the resistance of the soil cover, the pipe will tend to move upward, and considerable vertical displacements may occur. The pipeline response might then be unacceptable because of excessive vertical displacement or excessive plastic yield deformation. Upheaval buckling is hence a failure mode that has to be taken into account in the design of trenched and buried pipelines.

The possibility of upheaval buckling is of increasing concern to the operators of flowlines in the North Sea and elsewhere. Their concern has been heightened by upheaval buckling incidents in the Danish and Norwegian sectors, and by the cost of protective measures such as rock dumping. Shell International Petroleum Maatschappij (SIPM) initiated a comprehensive upheaval buckling research program in 1987. In its final form, the research program included six phases:

1 review of available models;
2 comparative analysis;
3 cover response, testing and modelling;
4 numerical upheaval buckling model;
5 analysis of alternative concepts;
6 development of an upheaval buckling guideline.

The present paper is one of a series describing the results of the programme. It concentrates on phases 5 and 6, which were subcontracted by SIPM to a joint venture of three UK engineering companies, Andrew Palmer and Associates, Advanced Mechanics and Engineering, and Lloyds Register.

The paper is primarily concerned with the design aspects of upward buckling of buried pipelines. Unburied pipelines can buckle in a related but different mode, in which they snake sideways across the seabed. This mode too is important in practice, and can play a major part in relieving the effects of axial force, but it is outside the scope of the present paper.

The paper first briefly describes the phenomenon, and puts forward a simplified method for preliminary design. It goes on to describe a conceptual design method based on the application of the UPBUCK

References and figures at end of paper

program, and applies the methods to a design example. The paper then considers alternative strategies to resolve upheaval problems, and examines in more detail one of these strategies, a reduction in wall thickness made possible by the replacement of stress criteria on longitudinal stress with strain-based design criteria.

UPHEAVAL BUCKLING

The source of upheaval buckling is an interaction between the axial compressive force and overbend "hill" imperfections in the pipeline profile.

Figure 1 illustrates a sequence of events which initiates buckling in a buried pipeline. The pipeline is laid across an uneven seabed (Figure 1a), and later trenched and buried (1b). The trenching and burial operations modify the profile of the foundation on which the pipe is resting, so that it is not precisely the same as the original profile. Trenching may smooth the profile overbends, but may also introduce additional imperfections, if, for instance, a lump of bottom soil falls under the pipe.

When the pipe goes into operation, its internal pressure and temperature are higher than when it was installed and trenched, and the axial force becomes compressive. The effective axial force in a constrained pipeline has two components, both of which contribute towards buckling (Palmer [1]). The axial force in the wall is the resultant of a compressive constrained thermal expansion component and a tensile Poisson component, while in addition there is a compressive force component in the contained fluid.

On an overbend, the axial compressive force reduces the upward reaction between the foundation and the pipeline (Figure 1c). A further increase of operating temperature and pressure may reduce the reaction to zero. The pipe then lifts on the overbend, moves towards the surface of the cover, and may break out through the surface (Figure 1d).

SIMPLIFIED ANALYTICAL MODEL

The stability of the pipeline in its initial position turns out to depend on the local profile of the pipe in contact with its foundation, and on whether or not enough downward force is present to hold the pipe in position. If the pipe does not move, the governing factors are the constrained axial force and the pipe flexural rigidity, and other parameters have no effect.

An arbitrary pipeline profile can be defined by a height y and a horizontal distance x. The pipeline is idealised as an elastic beam which carries an axial compressive force P and has flexural rigidity EI. It follows from elementary beam-column theory that the downward force w(x) per unit length required to maintain the pipeline in equilibrium in this position is

$$w(x) = -EI\,d^4y/dx^4 - P\,d^2y/dx^2 \quad \ldots.(1)$$

and therefore depends on the profile shape through the fourth and second derivatives.

Consider first a simple sinusoidal profile imperfection of height δ and length L defined by

$$y = \delta \cos^2(\pi x/L) \text{ in } -\tfrac{1}{2}L<x<\tfrac{1}{2}L \quad \ldots(2)$$

The downward force required to maintain this profile is therefore

$$w(x) = (-8\delta EI(\pi/L)^4 + 2\delta P(\pi/L)^2)\cos(2\pi x/L) \quad \ldots(3)$$

and has its largest numerical value at the crest of the imperfection, where

$$w = 2\delta P(\pi/L)^2 - 8\delta EI(\pi/L)^4 \quad \ldots\ldots(4)$$

the downward force per unit length required to stabilise the pipeline at the crest of the profile imperfection.

Equation (4) applies to a particular profile shape, but the specific shape only affects the coefficients and not the general form of the equation. It can be rewritten

$$\Phi_w = 2\pi^2\Phi_L^{-2} - 8\pi^4\Phi_L^{-4} \quad \ldots\ldots\ldots\ldots(5)$$

a relationship between a dimensionless maximum download parameter Φ_w defined by

$$\Phi_w = wEI/\delta P^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

and a dimensionless imperfection length Φ_L defined by

$$\Phi_L = L(P/EI)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(7)$$

This suggests that a useful way of representing the results of upheaval buckling calculations is to plot Φ_w against

Φ_L. The resulting plot is universal, and forms a valuable summary, which can include both the results of numerical calculations and then observations from full-scale field experience and tests.

All points that represent a single imperfection shape will lie on a single curve on a Φ_w against Φ_L plot. Points that represent different imperfection shapes will lie on different curves, and the variability between curves will reflect the effect of imperfection shape (the hardest parameter to determine reliably in practice). The form of equation 5 suggests that the functional relationship between Φ_w and Φ_L is

$$\Phi_w = c\Phi_L^{-4} + d\Phi_L^{-2} \quad \ldots\ldots\ldots\ldots\ldots\ldots(8)$$

where c and d are constants to be determined numerically. This is done by plotting $\Phi_w\Phi_L^2$ against Φ_L^{-2}, in Figure 2, on which the points represent the results of numerical calculations with the UPBUCK program described below. The rightmost group of points represents imperfection profiles whose shape is an upheaval foundation, that is, a foundation in the shape of a pipeline supported by axial force in a post-upheaval mode. The remaining points correspond to "prop" imperfection profiles, whose shape is that taken up by a pipeline laid across a single isolated hill in a horizontal profile. The final relationship turns out to be bilinear, with one pair of values c and d corresponding to small values of Φ_L and a second pair to large values.

An additional condition occurs when the profile includes a very short imperfection, so that the pipeline is only in contact with the crest of the imperfection. The length Φ_L is then immaterial, and this condition forms a cut-off for short imperfections.

The results are combined in Figure 3, a universal curve which can be used directly for design. The three conditions that together give the design download required for stability are:

$$\Phi_L<4.49,\ \Phi_w = 0.0646 \quad \ldots\ldots\ldots\ldots\ldots(9)$$
$$4.49<\Phi_L<8.06,\ \Phi_w = 5.68/\Phi_L^2-88.35/\Phi_L^4 \quad \ldots(10)$$
$$\Phi_L>8.06,\ \Phi_w = 9.6/\Phi_L^2 - 343/\Phi_L^4 \quad \ldots(11)$$

In most preliminary design contexts, the designer can determine the maximum height of a profile imperfection, but not its length. An imperfection length can however be estimated from an assumption that the pipeline takes up a form dictated by the interaction of its flexural stiffness and its weight in the installed condition (not in general the same as the weight in the operating condition). Substituting this into the third equation (11), we arrive at a design formula for the required download for stability in the operating condition:

$$w = [1.16 - 4.76(EIw_0/\delta)^{1/2}/P]P(\delta w_0/EI)^{1/2} \quad \ldots(12)$$

where

- w_0 is the installation submerged weight;
- EI is the flexural rigidity;
- δ is the imperfection height;
- P is the effective axial force in operation.

This equation is used to derive the required value for download for preliminary design. Preliminary design calculations apply a spreadsheet, which compares the required download determined from equation (12) with the actual load that can be mobilised, the sum of the pipeline's submerged weight and the uplift resistance of the cover. The research carried out into uplift resistance in the course of this research programme is discussed in detail in a separate paper[2]. In brief, the model recommended for design of buried pipelines is

for cohesionless sand, silt and rock

$$q = HD(1 + fH/D) \quad \ldots\ldots\ldots\ldots\ldots(13)$$

for cohesive clay and silt cover

$$q = cD \min[3,H/D] \quad \ldots\ldots\ldots\ldots\ldots(14)$$

where

- q is the uplift resistance per unit length of pipe;
- H is the cover depth (from the top of the pipe to the surface);
- D is the outside diameter of the pipe;
  is the submerged unit weight of the cover material;
- c is the shear strength; and
- f is an uplift coefficient, determined experimentally and taken as 0.5 for dense materials and 0.1 for loose materials.

The reader is warned that this simplified method is semi-empirical, and that on its own it will not normally be adequate for design. It has been calibrated against UPBUCK, but does not always yield conservative results, especially if there is a possibility of plastic deformation of the pipe wall.

CONCEPTUAL AND DETAILED DESIGN

Conceptual and detailed design are based on the UPBUCK finite-element program described in a separate paper[3]. The program takes account of a number of factors not included in the simplified analytical procedure. These factors have a significant effect on the response of the pipeline after it has begun to move in response to axial force, and thence on its post buckling behaviour and the amplitude of the final displacement. Among the factors are:

1 the finite axial stiffness of the line, which determines how rapidly the axial force diminishes as the line moves upwards;

2 the pipeline's longitudinal resistance to movement through the soil, which determines how far the pipeline can slide towards a developing buckle;

3 the incremental flexural rigidity for large deflections, which determines whether the pipeline's resistance to bending decreases as large displacements occur: the rigidity may be influenced by plastic bending deformation.

4 the finite stiffness of the foundation, which allows the foundation to deflect under forces applied by the pipe, and slightly modifies the deflected form of the pipe.

UPBUCK makes it possible for the user to calculate the response of the pipeline to increasing operating pressure and temperature, and to follow the deformation into the post-buckling range.

DESIGN EXAMPLE

The example is a 10-inch oil pipeline, with a maximum operating pressure of 15 MPa (2175 psi) and a maximum operating temperature of 80 deg C. The chosen steel grade is API 5L;X60, which corresponds to a specified minimum yield stress (SMYS) of 413.7 N/mm². The external anti-corrosion coating is 2 mm of polyethylene. The line is to be designed to DnV 1981 rules.

The line will be trenched, in order to protect it against fishing gear. A review of existing survey data along a nearby pipeline route has established that the seabed is relatively smooth, and that the maximum expected height of overbends will be 0.2 m. However, it is thought that trenching might induce additional imperfections in the bottom profile, and the design imperfection height is therefore taken as 0.3 m. The trench will be backfilled with rock, which has a minimum in-situ submerged unit weight of 8.5 kN/m^3 and an uplift coefficient of 0.5. The assumed in-situ submerged unit weight is a low conservative value, and corresponds to a voids ratio of 0.87 and a rock fragment density of 2650 kg/m^3. The option of trenching and rock-dumping, rather than rock-dumping alone, is selected because analysis has shown that the saving in rock cost more than outweighs the cost of trenching.

The wall thickness design arrives at a nominal wall thickness of 11.1 mm. The installation submerged weight is 591.3 N/m (60.2 kg/m, 40.5 lb/ft), and the operating submerged weight is 984.2 N/m.

The first step is preliminary design. It calculates the factor of safety on pipeline weight and cover uplift resistance taken together. This is the ratio between the required value, calculated from equation (12), and the actual value, the sum of the operating submerged weight and the cover uplift resistance taken from equation (13).

Figure 4 plots the calculated factor of safety against the depth of cover. It can be seen that if the cover is 0.6 m or less, the factor of safety is less than 1, that if the cover is 0.7 m the factor of safety is 1.11, and that if the cover is 0.8 m the factor is 1.32. This suggests that the minimum cover should be at least 0.8 m, since a 0.7 m cover leaves only a narrow margin. Accordingly, a minimum cover of 0.8 m is taken as the starting point for UPBUCK calculations.

A separate calculation determines the extreme stresses in operation, at the imperfection and under the axial loads induced by the operating temperature and pressure. Although the hoop stress is relatively low (183 N/mm², 44 per cent of yield), the longitudinal stress is high, and the von Mises equivalent stress is over yield. This condition is allowed by many codes, since the pipeline is continuously supported, but the high level of equivalent stress is an important warning that the pipeline response to axial loading may be influenced by plasticity, and that it will be prudent to carry out detailed UPBUCK calculations using the plasticity option.

Moving now to conceptual design, a series of UPBUCK calculations are carried out. In this phase of the analysis, the pipeline is treated as elastic.

The results of three UPBUCK calculations are summarised in Figure 5, which plots movement against temperature increase for cover depths of 0.7, 0.8 and 0.9 m above the peak of the 0.3 m imperfection. If the cover is 1 m (from the baseline), the pipeline begins to lift (at the top of the imperfection) at a temperature of about 70

deg C, some 10 deg C short of the operating temperature. It becomes unstable and jumps into a buckled configuration at 88.5 deg C. This option leaves too little margin between the operating temperature and the temperature at which large movements might occur, and is therefore rejected.

If the cover is 1.1 m, movements begin at about 80 deg C, and an unstable jump occurs at 95 deg C. At the design maximum operating temperature, the von Mises equivalent stress is 83 per cent of yield, which is within the maximum stress allowed by the code.

If the cover is 1.2 m, movement does not begin until the design temperature is reached. The conclusion of the conceptual design phase is therefore that a cover of 1.1 m (from the baseline) is adequate, and this value is chosen as the starting point for detailed design.

Continuing to detailed design, it has been shown earlier that the pipe is close to plastic yield, and that UPBUCK calculations using the plasticity option are required. The results show that movements initiate at almost exactly the 80 deg C design temperature, but do not exceed 10 mm until the temperature has risen to 88 deg C. The instability temperature is 97.3 deg C. Figure 6 plots the movement of the pipe at the peak of the imperfection against the temperature, and shows that instability will be accompanied by very large movements, in which the pipeline will break above the surface of the cover. However, there is an adequate temperature increase margin between the operating temperature and the instability temperature.

A cover of 1.1 m, measured from the baseline, is therefore selected. It corresponds to 0.8 m cover at the peak of the 0.3 m imperfection. It happens to coincide with the value reached by the simplified method, though this is not always the case.

This cover is only required on the overbends. If the critical overbends can be confidently identified from a survey, rock need only be dumped on those overbends. It will usually be cheaper to devote resources to survey rather than to continuous dumping. This design option was recently applied to the stabilisation of the Tern and Eider pipelines[4]. If on the other hand the survey accuracy is not sufficient to locate the critical overbends, the whole length must be covered.

ALTERNATIVE DESIGN STRATEGIES

The simplest and most straightforward way of stabilising a pipeline against upheaval is to bury it. This option is often difficult and expensive, and prompted a search for alternatives. Phase 5 of the research programme examined some forty alternative design strategies, evaluated each one, and identified the most generally attractive and applicable.

Two obvious strategies are to reduce the design operating temperature and pressure, and to increase the submerged weight of the pipeline. A reduction in operating temperature is generally impracticable, but could be accomplished by adding a heat exchanger to the system. It is not usually practicable to resolve the problem by an increase in submerged weight, because too much weight is required.

The remaining strategies can be classified into groups. The most promising techniques are listed in table 1.

The first group is based on reducing the driving force, the axial compressive force in the pipeline. The most direct method is to reduce the wall thickness of the line. 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 design factor to the highest allowable level, or by increasing the grade of steel, or by adopting strain-based design criteria, a possibility examined below in section 7.

A second method is to increase lay tension. Residual lay tension balances part of the compressive force induced by operating, and therefore reduces the resultant force. A difficulty is that residual tension cannot be measured directly, but must be calculated from the lay conditions, and that its continued presence in the line depends on there being no lateral movements. A third alternative is to preheat the line, and to allow it to move to relax compressive forces induced by preheat: the application of this technique to the Sun Oil Glamis flowlines is described in a separate paper[5].

A second group of methods depends on making a radical change in the structure of the pipeline. One alternative is to replace one or more single lines by a closed bundle supported on spacers in a carrier (Palmer[6]), or equivalently by a pipe-in-pipe system in which an internal flowline is supported in an outer pipe. The internal lines in the bundle then 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 on the spacers, but the movements are controlled and the bending stresses induced may remain at an acceptable level. This effect can be further enhanced by making the internal bundle helical (Duxbury[7]): this geometry increases the reduction in axial compressive force that accompanies outward movements of the internal lines. A decision to opt for a bundle concept naturally has broader implications that extend far beyond upheaval buckling, which is only one of the factors involved.

A second radical alternative is to replace a rigid steel pipeline with a flexible. Flexibles are subject to upheaval buckling (Bournazel[8], Putot[9,10]), 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 which tends to contract axially when loaded by internal pressure.

A third group of methods is based on stabilising the pipeline with rock, but using the rock in a more efficient way, so to gain an equally effective stabilising effect with less rock. The most effective of these methods is only to place rock on critical overbends (Locke[4]), but this does of course make it essential to be able to identify the overbends confidently, a demanding survey task in deep water, or when the critical imperfection height is small. A second method is to place a geotextile over the pipe before the rock is placed. When the pipeline begins to lift, the weight of the rock on the geotextile on either side of the pipe holds the geotextile down, and generates a tension whose downward component adds to the uplift resistance of the rock above the pipeline. At the same time, the geotextile pulls the rock inward, and increases the horizontal compressive stress in the rock above the pipeline, and that in turn increases the shear resistance across potential shear surfaces above the pipe, further increasing the uplift resistance. Against this, on the other hand, the use of a geotextile in a subsea environment will require a comprehensive investigation of its long-term stability against creep and structural deterioration.

A final alternative is to not to place the rock continuously along the whole length of the pipeline, but to place the rock in intermittent dumps. This possibility is discussed in an accompanying paper[11].

The offshore pipeline industry is only now beginning to explore the many different concepts that can be used to secure effective and economical stabilisation, and much more remains to be learned.

STRAIN-BASED DESIGN CRITERIA

In the past, the conventional approach to wall thickness design was generally governed by two stress requirements. The first requirement is that the hoop (circumferential) stress not exceed a defined fraction of the specified minimum yield stress (SMYS). A second requirement also has to be satisfied, and takes account of longitudinal stress; the maximum value of an equivalent stress must not exceed a specified fraction of the SMYS. The specified fraction for the equivalent stress is generally higher than the specified fraction for the hoop stress. The equivalent stress is defined either as a von Mises equivalent stress (proportional to the square root of the second invariant of the deviatoric stress tensor), or as the difference between the largest and smallest principal stresses.

If this approach is adopted for pipelines that operate at a high temperature, the second requirement often determines the wall thickness. This happens because the longitudinal stress in operation is large and compressive, and because the temperature component of stress is independent of the wall thickness. The need to reduce the equivalent stress below the allowable limit then forces down the allowable hoop stress, and this in turn leads to a high wall thickness.

An alternative approach is better. In the case of a continuously-supported constrained pipeline on or in the seabed, exceedance of an allowable combined stress does not in itself correspond to a limit state which in any way threatens the safety of the line. If the combined stress should reach yield, a limited amount of plastic deformation will occur, but it can be shown that the plastic components of longitudinal and hoop strain are small by comparison with the elastic component of hoop strain (provided that the first requirement still limits the hoop stress, and that the design factor is at customary level). The combined stress requirement can then be dropped, and replaced by a limit on strain. This is a more rational approach, and is allowed by several modern codes. The allowable strain is set at a high level, often 0.01 or 0.02, but this level is almost never reached, except when the pipe is deliberately plastically deformed in a construction operation such as J-tube pull. If the equivalent stress requirement is dropped and replaced by a limit on strain, the hoop stress condition determines the wall thickness. The resulting wall thickness is then often significantly reduced. Figure 7 is an example: it refers to an 8-inch flowline with a design temperature of 125 deg C and a design pressure of 27 MPa (3910 psi), and plots the hoop stress, the longitudinal stress, and the von Mises equivalent stress against

the wall thickness. If the equivalent stress requirement is adopted, and the equivalent stress is limited to 0.96 of SMYS (as in the DnV rules), and the steel has an SMYS of 413.7 N/mm² (corresponding to API X60), the minimum wall thickness is 12.8 mm. If on the other hand the equivalent stress requirement on longitudinal stress is dropped, and the hoop stress is limited to 0.72 of SMYS (as in many codes), the minimum wall thickness can be reduced to 9.6 mm.

This reduction in wall thickness leads to a substantial reduction in axial force in operation, and to a corresponding reduction in minimum cover required for stability.

Adoption of strain-based criteria does however raise a number of wider questions, which tend to reduce the advantages. Figure 8 plots the stress path followed in a sequence of laying, increase in temperature, and increase in operating temperature, in a diagram whose axes are longitudinal stress and hoop stress. The ellipses represent successive yield loci for a von Mises material with an isotropic hardening rule. The heavy ellipse represents the nominal yield locus, corresponding to the 0.5 per cent yield locus, but yield begins before this locus is reached, and is represented schematically by the inner loci.

If a strain criterion is adopted, the stress point S that represents the mean longitudinal and hoop stresses during operation lies beyond the yield locus, even if the hoop stress is less than 0.72 of nominal yield. If the pipe then begins to bend, the points representing the stresses in the pipe in the "inside" and "outside" of the curve follow the paths shown in Figure 9b. On the inside of the curve, the longitudinal stress becomes more compressive, and the stress point moves to the left. This takes it across additional yield loci, and more yield occurs. On the outside, the longitudinal stress becomes more tensile, and the stress point moves inward, so that the incremental response is elastic.

It follows that the incremental bending of a pipeline designed according to strain criteria will always be accompanied by plastic deformation, however small the bending is. The effect is to generate both circumferential and longitudinal plastic strain. The incremental increase in bending moment is much smaller than it would be if the pipe were elastic, because only part of the pipe is responding elastically: the effect is to reduce the incremental flexural rigidity, which is one of the parameters that enable the pipe to resist upheaval buckling. At the same time, plastic deformations relieve some of the axial compressive force in the pipe wall, and therefore reduce the driving force.

Plasticity effects will therefore have a significant influence on upheaval buckling of pipelines designed according to strain criteria. The solution will be sensitive to the details of the stress-strain relation for the steel, particularly to strain-hardening. These effects are taken into account in analysis by the UPBUCK program.

CONCLUSION

This paper summarises design methods based on the results of an extensive program of research into upheaval buckling.

ACKNOWLEDGEMENT

The research described here was carried out as part of a joint industry program led by Shell International Petroleum Maatschappij and supported by the following companies:

Shell Internationale Petroleum Maatschappij
Maersk Olie og Gas A/S
Nederlandse Aardolie Maatschappij BV
Shell UK Exploration and Production
A/S Norske Shell
Britoil Plc (BP Exploration)
UK Department of Energy
Marathon Oil UK Limited
Elf Petroland BV
Occidental Petroleum (Caledonia)
Saga Petroleum A/S
Sun Oil Britain Limited
Total Oil Marine plc

The authors wish to record their gratitude for permission to publish this paper.

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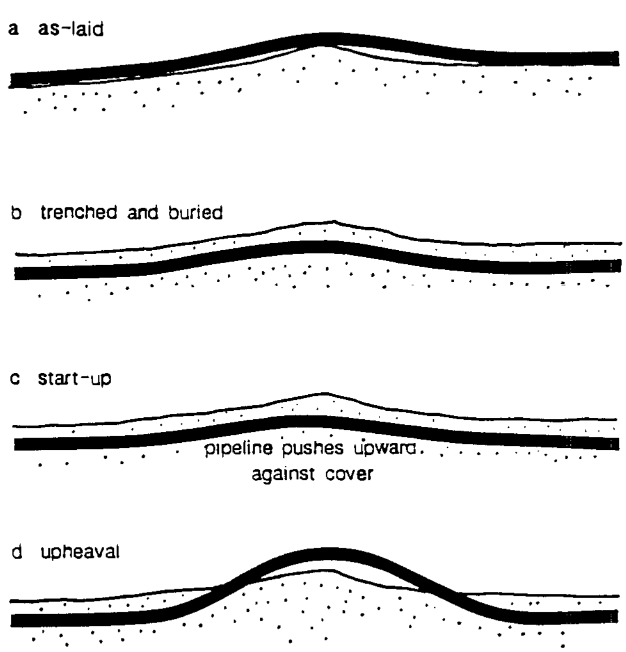


Figure 1 Sequence of laying, trenching and upheaval

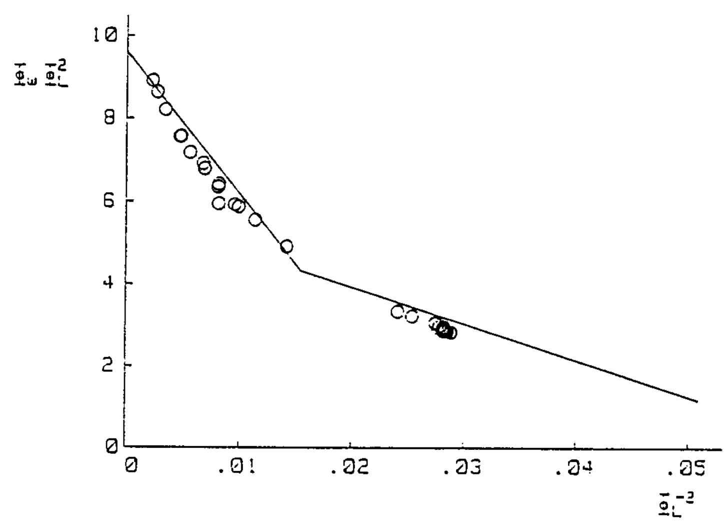


Figure 2 Numerical correlation of UPBUCK results

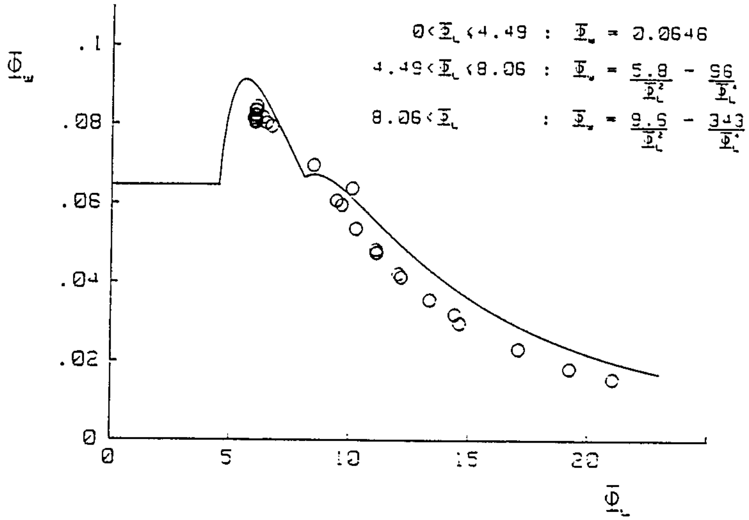


Figure 3 Universal design curve

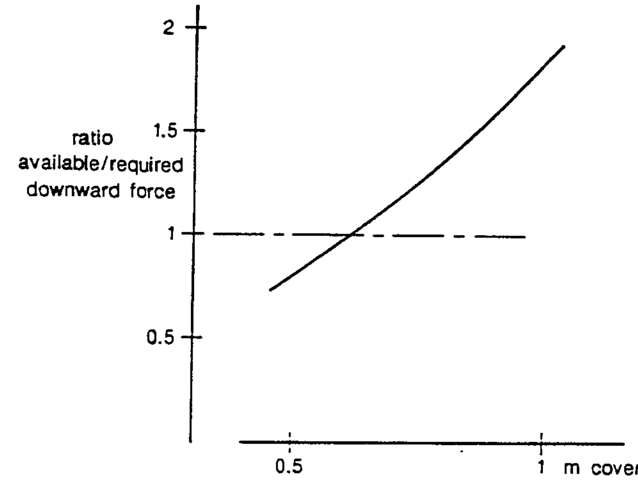


Figure 4 Design example 1: simplified method

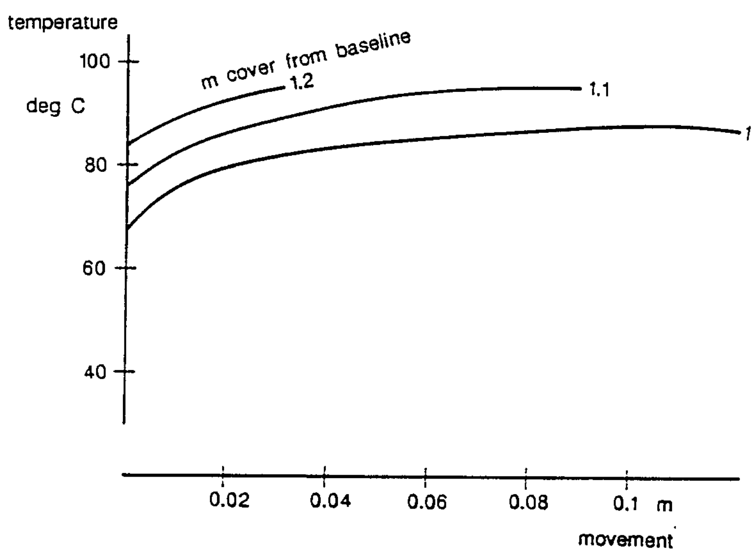


Figure 5 Design example 1 Results of level 1 UPBUCK (elastic)

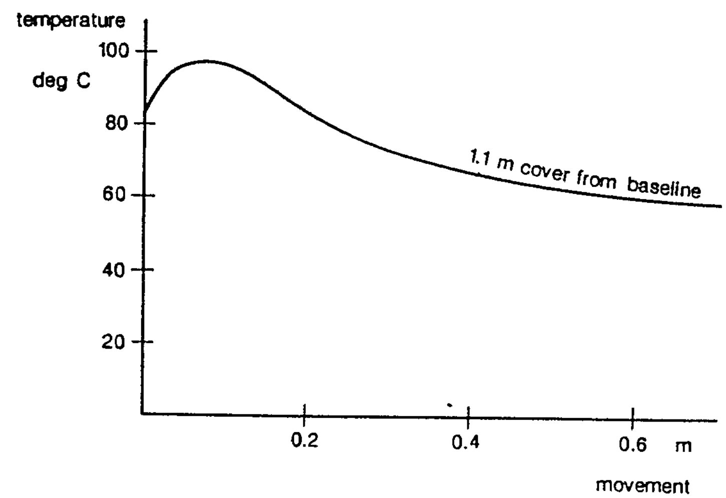


Figure 6 Design example Results of level 2 UPBUCK (elastic-plastic)

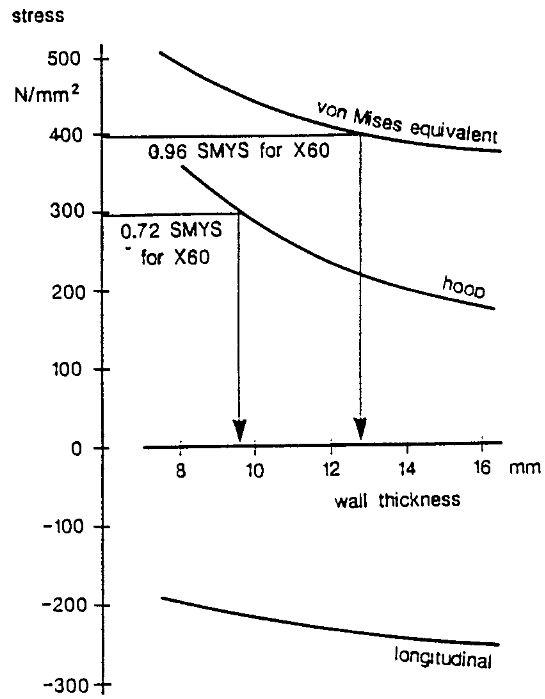
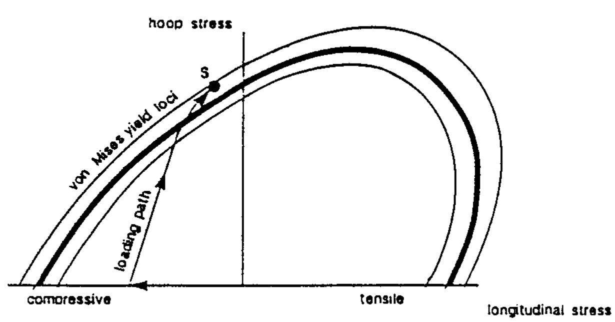
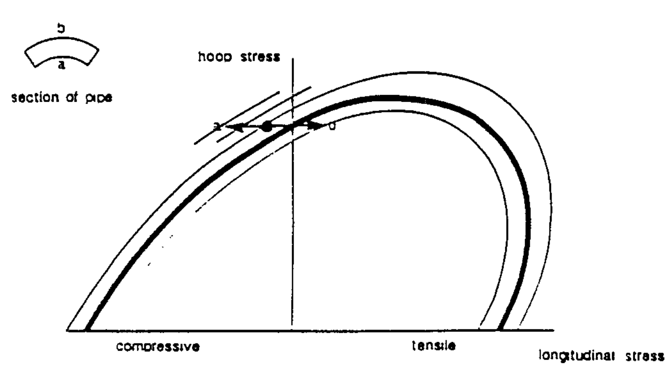


Figure 7 Example of effect of strain-based design



a stress path during loading beyond yield
assumes: no bending, no residual tension, temperature increase applied first, pressure increase second



b incremental stress changes induced by bending from initial state represented by point S

Figure 8 Implications of strain-based design for stress and yield in pipeline