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Upheaval Buckling Assessment Based on Pipeline Features

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Abstract

Upheaval buckling (UHB) is a common design issue for high temperature buried pipelines. This paper highlights some of the key issues affecting out-of-straightness (OOS) assessment of pipelines. The following factors are discussed; uplift resistance soil models, uplift resistance in cohesive soils, uplift mobilisation, ratcheting, uplift resistance at low H/D ratios and the correct methodology for load factor selection. A framework for determining ratcheting mobilisation is proposed. Further research is required to verify and validate this proposed framework. UHB assessment of three different diameter pipelines were carried out using finite element SAGE PROFILE package incorporating pipeline mobilisation and the results are compared with semi-analytical formulation proposed by Palmer et al. 1990. The paper also presents a summary of as-laid pipeline features based on projects over the past 10 years.

Introduction

Upheaval buckling (UHB) is a common design issue for buried pipelines when the out-of-straightness of the pipeline combined with the high axial compressive forces induced by the extreme operating conditions causes the pipeline to buckle upwards. In order to prevent upheaval buckling, the pipeline has to be buried deep enough such that the soil cover is sufficient to provide adequate uplift resistance. An out-of-straightness (OOS) assessment utilises the pipeline profile obtained from survey and appropriate soil modelling to calculate the soil or rock cover depths needed to mitigate UHB throughout the length of the pipeline. Figure 1(a) shows a typical schematic of upheaval buckling while Figure 1 (b) shows a pipeline in the field which has undergone upheaval buckling. Proper modeling of upheaval buckling requires in depth understanding of pipe-soil interaction and key factors affecting buried pipeline behaviour. Uplift soil models, uplift resistance in cohesive soils, uplift mobilisation, ratcheting, uplift resistance at low H/D ratios and correct methodology for load factor selection for UHB assessment are all discussed in this paper.

During pipeline Front-End-Engineering-Design (FEED), the semi-analytical formulation proposed by Palmer et al. 1990 (OTC paper 6335) has been one of the fundamental methods of assessing the required uplift resistance for the last 20 years. The feature geometry being limited to prop-type and inability to account for the pipeline mobilisation during operation are often seen as a limitation of this formulation. This paper investigates these issues and provides the results to improve the understanding of UHB with mobilisation.

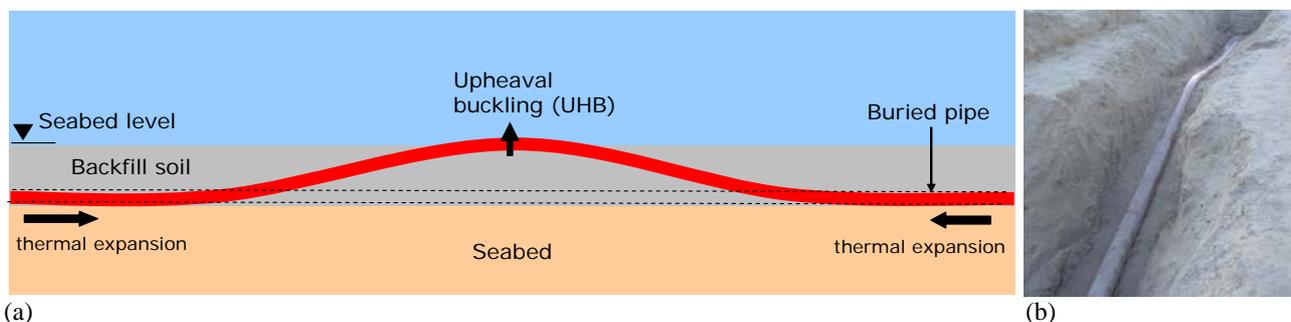


Figure 1: Upheaval Buckling (UHB) of a buried pipeline

Uplift resistance soil models (drained uplift resistance models)

Present understanding on uplift resistance of buried pipe lines is based on analysis (Randolph and Houlsby, 1984; Pedersen, P.T. & Jensen, J.J. 1988,) and experimental work by researchers (Vesic, 1971; Rowe and Davis, 1982; Hobbs, 1984; Randolph and Houlsby, 1984; Trautman *et al.*, 1985; Palmer *et al.*, 1990; Schaminée *et al.*, 1990; Dickin, 1994; Croll, 1997; White *et al.*, 2001; Bransby *et al.*, 2001; and Cheuk *et al.*, 2005; Finch, 1999; Finch *et al.* 2000; Moradi & Craig, 1998, Wang *et al.* 2009, Thusyanthan *et al.* 2008).

One of the early models to be used for prediction of peak uplift resistance, R , is given by Equation 1 (Schaminée *et al.*, 1990).

$$\frac{R}{\gamma' \cdot H \cdot D} = 1.0 + f \left(\frac{H}{D} \right) \quad \text{Equation 1}$$

where the H is cover to top of pipe and f is uplift factor (Schaminée).

DNV-RP-F110 recommends the use of the following uplift model (Equation 2) to predict the peak uplift resistance in cohesionless soils.

$$\frac{R}{\gamma' \cdot H \cdot D} = 1.0 + 0.1 \left(\frac{D}{H} \right) + f_p \left(\frac{H}{D} \right) \left(1 + \frac{D}{2H} \right)^2 \quad \text{Equation 2}$$

where f_p is uplift factor.

It should be noted that DNV-RP-F110 recommended uplift model uses weight and shear components of the soil regions just above the pipe surface but below top of pipe as shown in the Figure 2 (K is horizontal earthpressure coefficient at rest). The model in Equation 1 (Schaminée *et al.*, 1990) does not account for weight and shear components below top of pipe. Hence, it should be noted that the uplift factor in Equation 1 and 2 are not interchangeable between the models and should always be used with model from which it was calibrated.

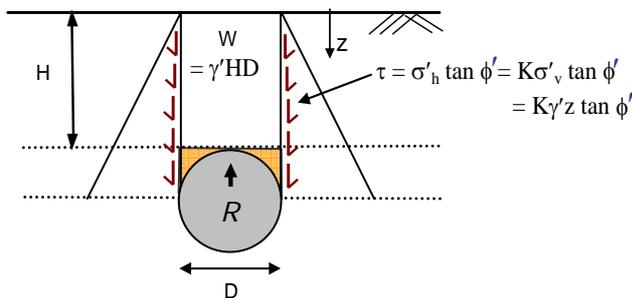


Figure 2: Uplift Resistance Model (drained)

Uplift resistance in cohesive soils

UHB assessment in cohesive soils has to consider both drained and undrained uplift resistance. Soil behaviour depends on the rate of loading (i.e rate of shearing of soil). If the rate of loading is greater than the rate at which pore water is able to move in or out of soil inter-particle voids, then the soil behaves in an undrained manner (i.e the volume change is zero, and the behaviour of the soil is independent of the stress level of the soil). If the rate of loading is slower than the rate at which pore water is able to move within the soil particles, the soil behaves in a drained manner (i.e frictional behaviour and the exhibited strength depends on the effective stress experienced by the soil). In summary, whether a soil (sand or clay) behaves in a drained manner or undrained manner depends on the rate of loading with respect to the permeability of the soil.

It is often mis-assumed that the pipeline uplift is a relatively rapid phenomenon. This implies that if undrained uplift resistance is enough to resistance the pipeline then the design is safe in the longterm. This is incorrect, when the pipeline is first operational, it is true that the loading cycle take few hours (i.e the temperature of the pipeline increase to operating temperature). The location of any features of pipeline starts to apply an upward loading on the soil cover. If the undrained uplift capacity of the backfill is able to resist this force then the pipeline's upward movement is restricted but the pipeline continues to apply this upward force on the soil as long as the pipeline is in operation. This means that if the drained uplift resistance of the backfill is lower than the undrained uplift resistance, the pipe will slowly move upwards with time. If this continues over the long term, the pipe will slowly move upwards and as the cover height decreases both drained and undrained uplift capacities of the backfill decrease making the upward movement easier (creeping upwards). The final failure can occur in an undrained manner. Thus, upheaval buckling assessment of cohesive soil should consider both drained and undrained behaviour of the backfill. DNV-RP-F110 does provide both undrained and drained uplift checks for pipelines in cohesive soil.

Uplift mobilisation distance

In order to prevent upheaval buckling, the pipeline has to be buried deep enough such that the soil cover is sufficient in providing adequate uplift resistance, Figure 3(a). The required upward movement, or mobilisation, of the pipeline to achieve the desired uplift resistance is a vital design parameter, in that pipeline integrity under operating conditions relies upon its value. An important point to note is that the mobilisation of the pipe often needs to be limited in order to limit the stresses in the pipeline, thus the available uplift resistance from the soil is not the peak uplift resistance but could be much lower as shown in Figure 3(b).

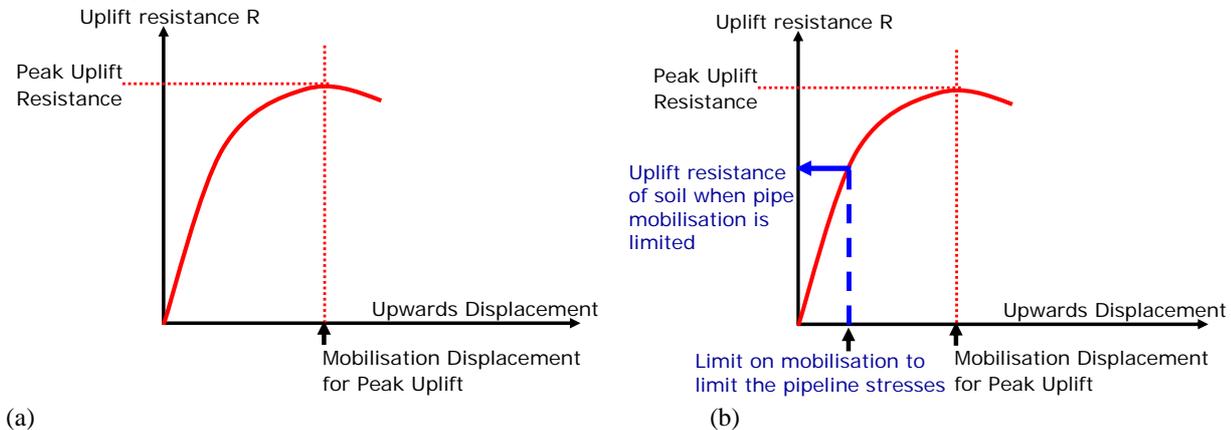


Figure 3: Typical curve of uplift resistance vs. upward pipeline displacement

The DNV states (Pg 44 of DNV RP F110), “The uplift resistance R_{max} is assumed to be fully mobilised at a vertical uplift displacement δ_f , where δ_f is 0.005-0.01 times the height H . Note that δ_f seems to be independent of the ratio of H/D ”. However, experimental full scale experiments (Thusyanthan et al. 2010, Figure 4) have demonstrated that the mobilisation is actually a function of (H/D) and DNV guideline underestimates the mobilisation distance. The effect of this under-estimation of mobilisation when combined with the use of tri-linear uplift resistance model, which is recommended by DNV, can lead to unconservative UHB designs. This has been demonstrated in Thusyanthan et al. 2010 using FE results. Based on the data from literature and full scale testing, Thusyanthan et al. 2010 proposed the following equation to predict the peak mobilisation distance (δ_f) in loose sands in terms of H and D . This is as given below (also shown in Figure 4),

$$\frac{\delta_f}{D} = 0.02 e^{\left[\frac{1H}{2D} \right]} \tag{Equation 3}$$

It should be noted that any effects of soil saturation (dry, moist & submerged) and soil density on mobilisation was not distinguished in this equation. This is being investigated and will be published in a future paper.

Uplift resistance at low covers

The current design practice for evaluating uplift resistance in cohesionless soils (sands and gravels) is based on DNV-RP-F110. As the DNV guideline does not specify how to evaluate uplift resistance for low H/D ratios (i.e $H/D < 2$), current industry practice is to ignore the contribution from shear altogether when H/D ratios are below ~ 1 (i.e uplift resistance is taken as the weight of the soil vertically above the pipe). The main reason behind this conservatism is the absence of available data at these low H/D ratios. This conservatism at low H/D ratios, for large pipes, can lead to large quantities of rock dump material being required as secondary backfill and thus costing the project millions of dollars. Wang et al. 2010 produced some full scale and centrifuge tests results which showed that the vertical slip model and the associated design equation are applicable for design scenarios with H/D ratios less than 1 (tests had been done on loose sand, dense sand and gravel).

Ratcheting in cohesionless soils

Pipelines undergo thermal cycles during start-up and shut-down stages. As the pipeline temperature increases, the axial force in the pipeline increases and when this axial force is above the buckling force for any out-of-straightness, pipeline features mobilise upwards till limited by the uplift resistance from the cover soil. When oil/gas supply is shutdown, the pipeline cools down and the pipeline tries reverting to its original shape and bear on the soil underneath. This downwards movement of the pipeline feature is resisted by the downward resistance of soil if the void below the pipe has soil infill. If mobilisation (upward movement) of the pipe during operational stage is not the same as downward movement during the shutdown stage, the pipe can experience incremental upward movement which can ultimately lead to upheaval buckling failure. This phenomenon is named pipe ratcheting. The onset of ratcheting is related to the stage when soil particles starts to fill the gap

beneath the pipe. The risk of ratcheting can be minimised by limiting the upwards movement of the pipe during operational stage such that it can fully come back to its original position during shutdown.

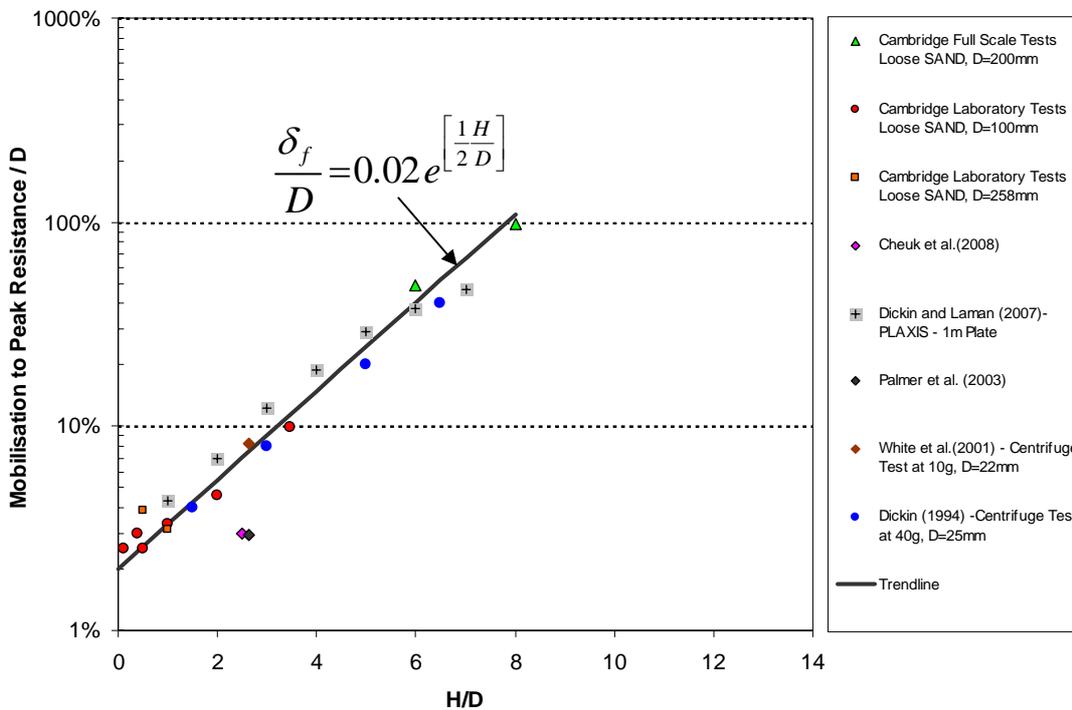


Figure 4: Peak mobilization (δ_f) vs. cover height to top of pipe (H), both normalised by pipe diameter (D), proposed by Thusyanthan et al. 2010

The literature on pipeline ratcheting is very limited. Upheaval failure attributed to ratcheting was reported by Nielsen et al. (1990). Finch (1999) provided some experimental data of ratcheting behaviour from full scale testing of a 0.22m diameter pipe buried to a depth of 0.58m with fine sand (submerged unit weight 8.8kN/m^3). Experimental uplift results from dry sands (fine and coarse sand) were presented by Cheuk et al. (2008) for a pipe diameter of 0.1m and cover of 0.25m. The test results of Cheuk et al. (2008) showed that a cavity with sloping sides forms beneath the pipe during the mobilization of peak resistance. For fine sands, particles started to infill this cavity after 3mm ($\sim 10D_{50}$) of pipe movement, leading to irrecoverable upward pipe displacements. For the coarse sand, this infilling mechanism occurred at a pipe displacement of 9 mm ($\sim 4D_{50}$).

The onset of ratcheting is related to the stage when soil particles start to fill the void beneath the pipe. This onset depends on soil particle size, friction angle of the soil, dry/partially saturated or submerged soil and pipeline diameter. For clayey soils, shear strength and time duration of the uplift cycle will also influence the onset of infilling. It should be noted that the onset of infilling below the pipe is one of the key requirements for ratcheting but it does not on its own lead to ratcheting, i.e. the pipeline downward force during shutdown, if sufficient enough, could move the infill soil and revert to original pipeline profile.

Infilling of soil is certain to occur (in uncemented sands and drained conditions in clay) and both sides of the infill will meet at the centre when the pipe has moved upwards such that it is above the angle of repose lines as shown in Figure 5. Thus based on the geometry shown in Figure 5 and assuming angle of repose (α) to be 32° (critical friction angle of sand), we can calculate the mobilisation (δ) at which the infilling from both side of the pipe meet at the centre. This value is $9\%D$. If we assume that when both sides of the infilling meeting at the centre is a clear criterion for ratcheting, then we can expect ratcheting to occur for mobilisation greater than $9\%D$ in sands with angle of repose of 32° (note that it will be $13.5\%D$ if the angle of repose is 38°).

Initiation of the infilling of soil can occur much earlier than the $9\%D$ mobilisation, but this may not lead to ratcheting and currently there is limited data to verify this. The results from Cheuk et al. (2008) on initiation of infilling in fine sand suggests that $3\%D$ may be the lower limit for the onset of infilling and hence ratcheting.

Based on the limited data available on ratcheting and the presumption that ratcheting is certain to occur at $9\%D$ mobilisation, we could propose three different regimes for ratcheting in relation to mobilisation and pipeline diameter as shown in Figure 6.

- Mobilisation of more than $9\%D$ - Ratcheting
- Mobilisation in-between $3\%D$ to $9\%D$ - Ratcheting possible
- Mobilisation less than $3\%D$ - Ratcheting highly unlikely

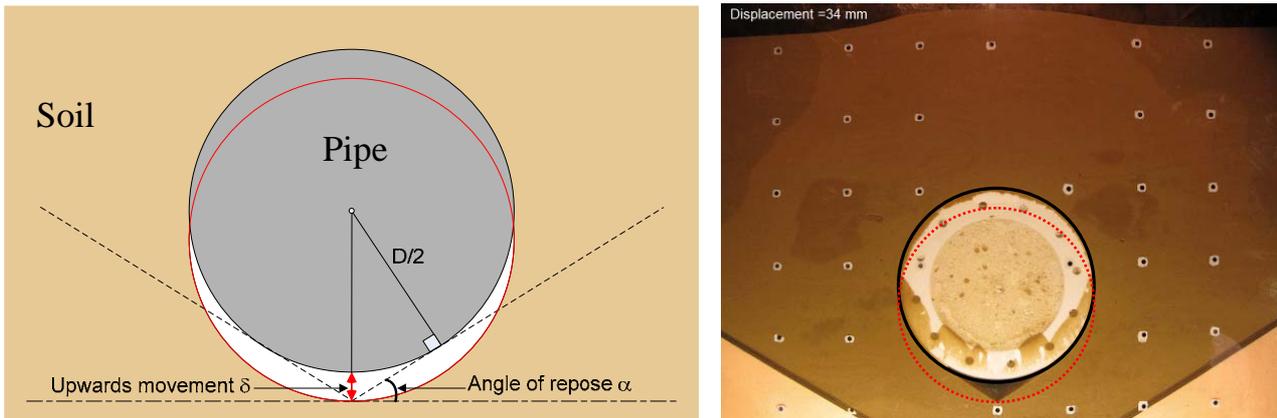


Figure 5: Pipeline uplift mobilisation and angle of repose

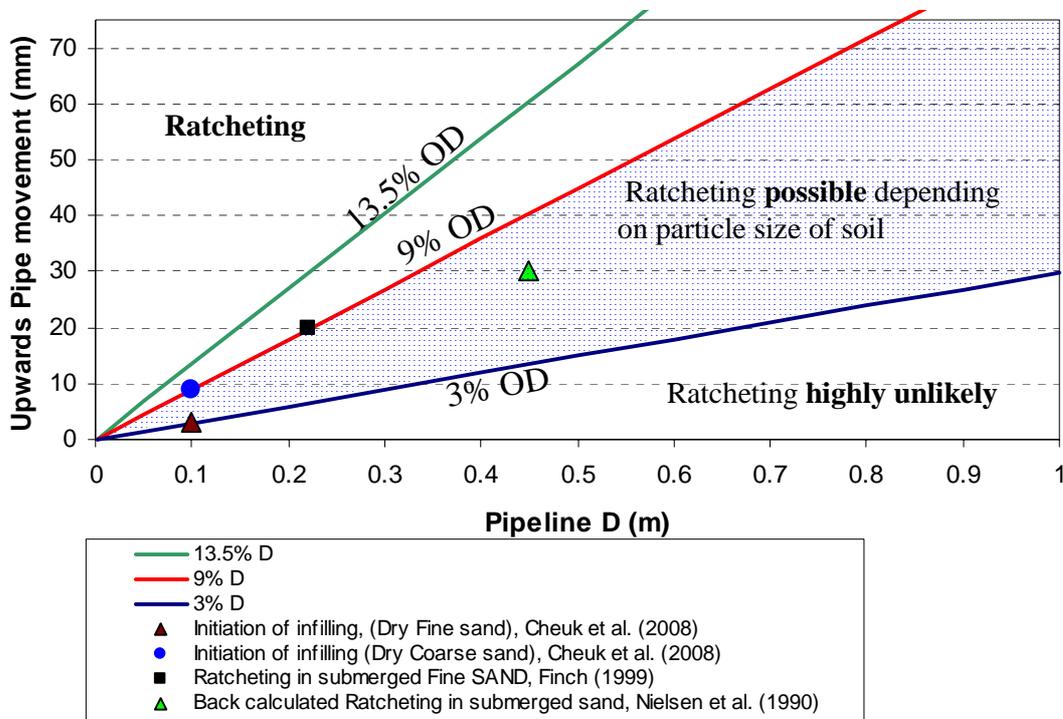
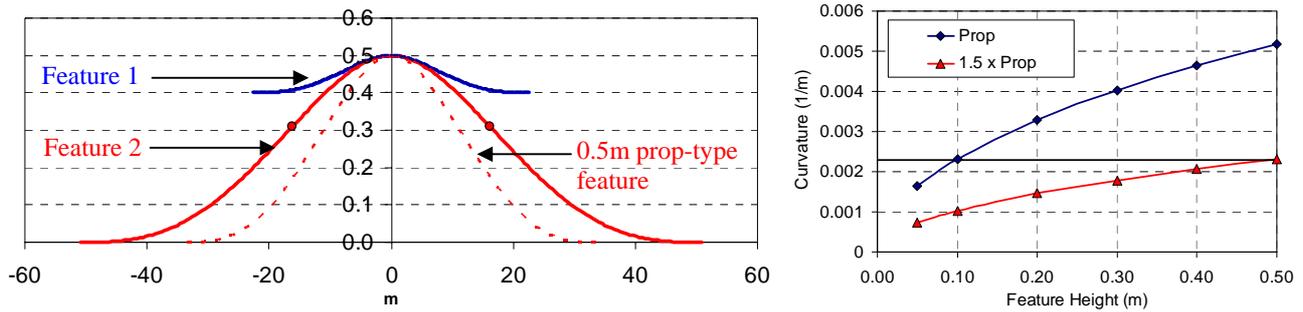


Figure 6: Proposed regimes for ratcheting based on pipeline diameter and upward mobilization of pipe

Choice of Load factor selection for UHB/OOS assessment

The driving force for the UHB is the axial force in the pipe, induced by temperature and pressure of the content, pipe-soil interaction (axial) and the pipeline end conditions. The resisting force for the UHB depends on factors such as material properties of the pipeline, pipeline dimensions, ovality, out-of-straightness (measured by the survey) and the soil resistance. There is uncertainties associated with all of the factors that contribute to resisting UHB and in order to take this into account; a reliability assessment is carried out prior to any UHB or OOS assessment. This reliability assessment captures all the uncertainties and provides appropriate load factors to be used in the assessment. The axial force is then multiplied by appropriate load factor before UHB/OOS assessment is carried out. The selection of load factor depends on factors such as pipeline out-of-straightness (feature height and wave length), survey accuracy and probability of failure required by design (generally 10^{-3}).

It should be noted that the appropriate load factor to be utilised in the OOS assessment cannot be solely based on the pipeline curvature. This is because there are infinite combinations of feature heights and wave lengths that can yield the same curvature at the crest. This fact is illustrated below with an example.



(a) Two pipeline features with same curvature at the crest
Figure 7: Pipeline features and corresponding peak curvature

(b) Curvature vs. Feature height

Figure 7 (a) shows two pipeline features (Feature 1 and Feature 2), feature 1 is a prop feature with a height of 0.1m and feature 2 is a $1.5 \times$ prop feature with a height of 0.5m. It is to be noted that both features have almost identical curvature (0.0023) at the crest, as shown in Figure 7 (b). Thus, if we base the load factor selection solely on curvature, we will end up using the same load factor for both features which is highly overconservative for feature 2. This can be seen from the following case. **Table 1** provides the results of a structural reliability assessment of a pipeline ($D=0.32\text{m}$) for a given standard deviation accuracy of the survey. If curvature based selection of load factor is used, a load factor of 1.7 will be assigned for both feature 1 and feature 2 as both have curvature of 0.0023. However, the appropriate load factor for feature 2 ought to be 1.3 (note 1.3 is still conservative as feature 2 is a 1.5 prop feature). Thus the OOS assessment with load factor selection based on curvature alone could be overly conservative and lead to excessive rock dump.

Table 1 : Typical Reliability Assessment Results

Feature Height (m)	Peak Curvature of Prop feature (1/m)	Load Factor
0.05	0.0016	1.8
0.1	0.0023	1.7
0.2	0.0032	1.6
0.3	0.0040	1.5
0.4	0.0046	1.4
0.5	0.0052	1.3

Furthermore, extra caution is required when it comes to the determination of the minimum soil height (background cover) required to mitigate against undetectable features due to the survey inaccuracies and data smoothing. Curvature based approach might not assign the appropriate background cover to secure the small undetectable features. Hence, it is recommended to consider the imperfection heights have to be considered in conjunction with curvatures.

Semi-empirical method of UHB susceptibility assessment

During FEED stage of the pipeline design, designers often use semi-analytical methods to assess the pipeline susceptibility to UHB. The method proposed by Palmer et al. 1990 (OTC paper 6335) has been one of the main methodologies for UHB assessments in the absence of FE analysis. This method is a very elegant and simple solution to the UHB problem, however it does have some limitations and these limitations are sometimes overlooked by the design engineers. The Palmer et al. 1990 did state, "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 pipe wall".

Two questions are often raised with the use of semi-empirical formulation of Palmer et al. 1990; the applicability of the formulation for non-prop features and the effect of pipeline mobilisation on the results. The next section investigates these limitations and presents results to improve the understanding of UHB of non-prop features with pipeline mobilisation. The reader is referred to Palmer et al. 1990 for details of the formulation but Equation 4 and 5 (P_{eff} is axial force in the pipeline, w is the downward force per unit length) presents the normalisation proposed by Palmer et al. 1990. A typical pipeline geometry shown in Figure 8.

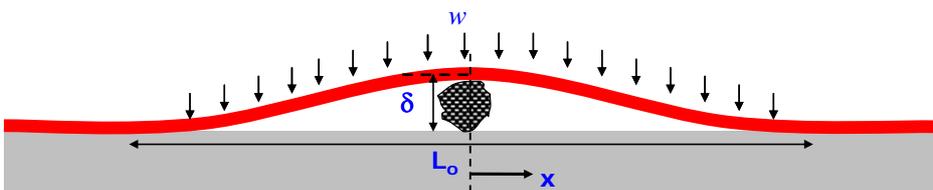


Figure 8: Pipeline geometry under a prop

$$\phi_L = \frac{L_0}{2} \cdot \sqrt{\frac{P_{eff}}{EI}} \tag{Equation 4}$$

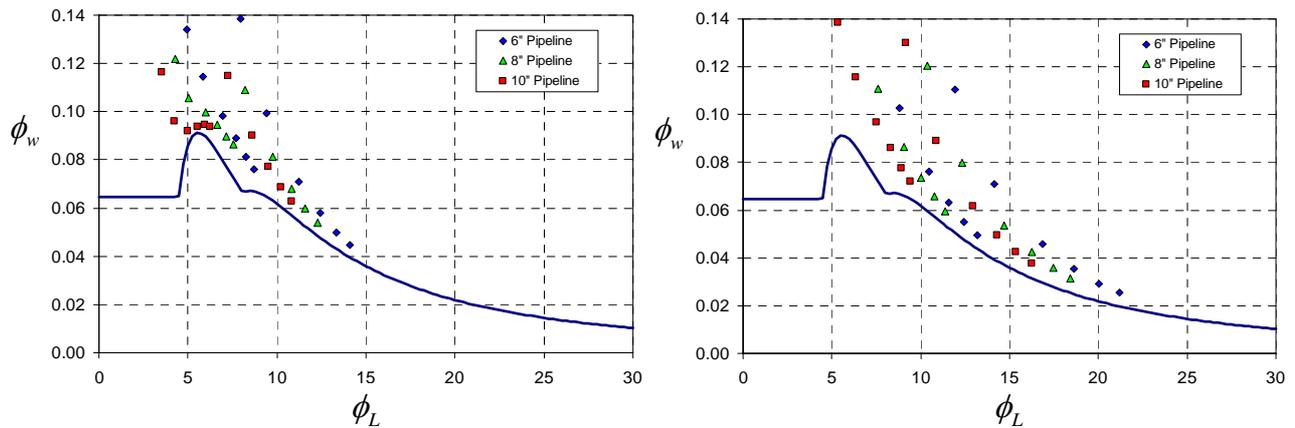
$$\phi_w = \frac{w \cdot EI}{\delta \cdot P_{eff}^2} \tag{Equation 5}$$

Parametric Study

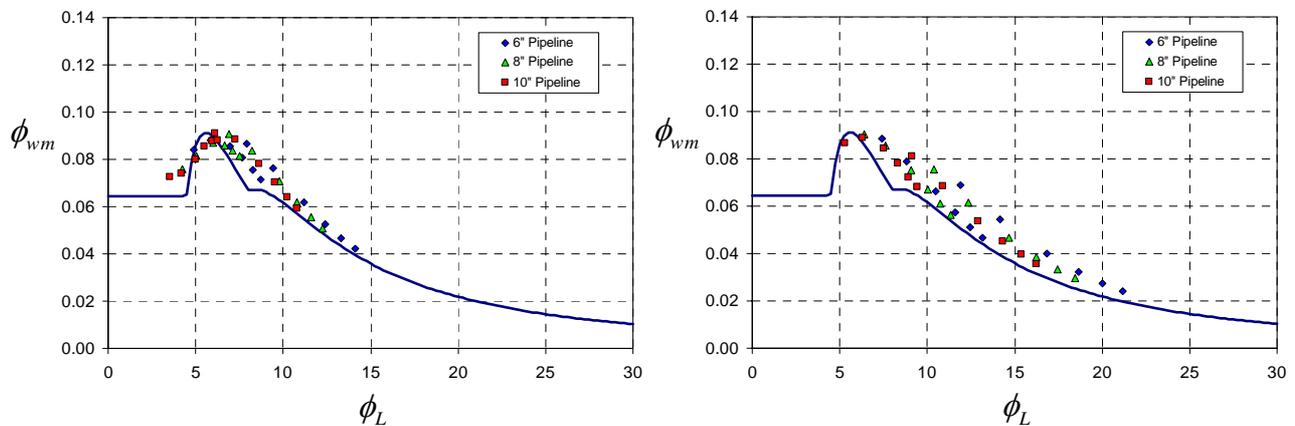
An UHB analyses were performed for three different pipelines diameters (6", 8", and 10") using finite element SAGE PROFILE package. Symmetrical and continuously supported idealized features ranging between 0.05m to 0.5m were considered with three different wavelength ratios (namely, 1.0, 1.5, and 2.0) based on pipelines flooded weights. Four different pipelines mobilisation displacements (10mm, 20mm, 30mm, and 40mm) were adopted in the operational analyses cases for each feature. Pipelines propensity to upheaval buckling were then assessed for the different pipelines' imperfection sizes under two different effective axial forces (high and low). Analyses results were normalized as per Palmer et al. (1990) non-dimensional variables to enable the comparisons. Figure 9(a) shows the UHB assessment results for prop-type features of all three pipelines with 30mm mobilisation. Figure 9(b) shows the UHB assessment results for 1.5 × prop-type features of all three pipelines with 30mm mobilisation. It can be seen that the results are not in agreement with the original solution line of Palmer et al. (1990).

In order to account for the pipeline mobilisation in normalisation, the normalised factor ϕ_w was re-defined as ϕ_{wm} as shown in Equation 6 below and the results of Figure 9(a) and (b) were replotted in Figure 10(a) and (b). It can be seen that the results are now more in line with the original solution line, however there is still a 10%-30% difference in ϕ_{wm} .

$$\phi_{wm} = \frac{w \cdot EI}{(\delta + mob) \cdot P_{eff}^2} \tag{Equation 6}$$



(a) prop-type feature with 30mm mobilisation (b) 1.5 × prop feature with 30mm mobilisation
Figure 9: Numerical Results from SAGE compared with original Palmer et al. (1990) solution.



(a) prop-type feature with 30mm mobilisation (b) 1.5 × prop feature with 30mm mobilisation
Figure 10: Numerical Results from SAGE compared with modified normalization.

Development of OOS Database

An OOS database is being developed to collect the pipeline survey data of over 100 pipelines that have been trenched and buried in the seabed over the last 10 years. This database will have a wealth of information regarding OOS features of pipelines. It will also have the results of upheaval buckling assessment carried out using finite element SAGE PROFILE package. The database is currently under development and pipeline data of 35 pipelines have been fully captured so far. Figure 11 presents the OOS features from the 35 pipelines of various diameters.

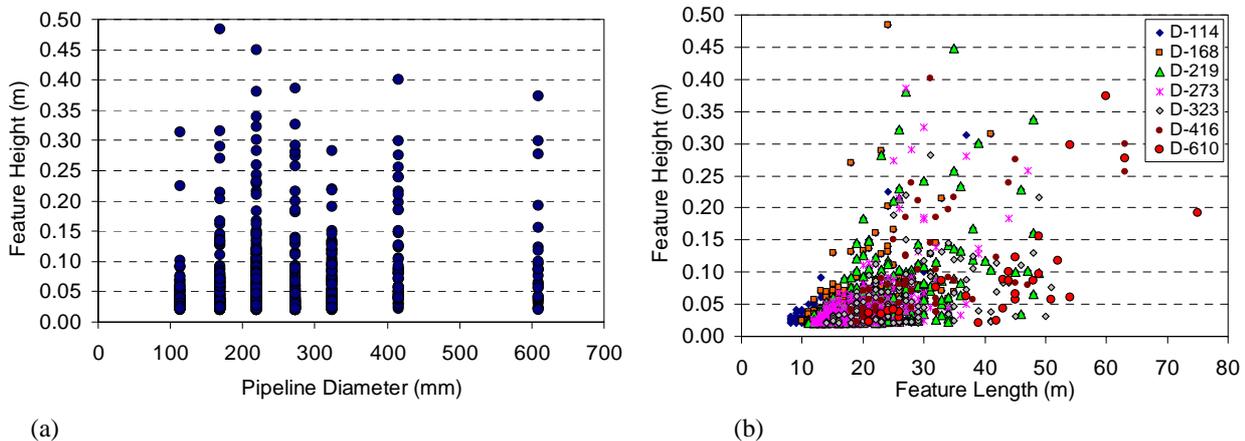


Figure 11: Pipeline features from OOS database of 35 pipelines

Conclusions

Some of the fundamental issues that needs to be considered in performing UHB/OOS assessment have been discussed in this paper. These include;

- Uplift resistance soil models and the importance of correctly modeling the uplift mobilisation were highlighted.
- A framework for pipeline ratcheting in cohesionless soils is proposed supported by limited experimental data. Further research is required to verify and validate the proposed framework.
- It has been shown the load factor selection solely based on pipeline curvature can result in highly overconservative soil/rock covers.
- UHB assessment using SAGE PROFILE FE package of prop and non-prop features including mobilisation has been compared with the semi-empirical formulation proposed by Palmer et al. (1990). It has been shown that a modified normalisation is required to bring the finite element UHB results closer to the original solution. Even then, there is a difference of 10%-30% on the normalised values.

Acknowledgement

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