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State-of-the Art Knowledge on Upheaval Buckling of Buried Pipelines

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ABSTRACT: This paper presents state-of-the-art knowledge on upheaval buckling, providing an overview on commonly used upheaval buckling soil models, latest uplift resistance results from experimental and numerical studies, investigations into the factors affecting the uplift resistance of soils and recommendations for design. The paper addresses the uplift resistance for both onshore and offshore pipelines. For onshore pipelines, the backfill soil cover could be dry, fully saturated or partially saturated. Thus, insight into the effects of degree of soil saturation on the uplift resistance is provided. For offshore pipelines, predicting the uplift resistance of buried pipelines has been a challenge due to uncertainty and randomness in the nature of soil cover created by various pipe burial techniques. This paper provides guidelines, supported by published literature, on the uplift resistance of different types of backfills such as sands, clays and blocky clays. An insight into the cyclic ratcheting mechanism, which is the driving mechanism leading to UHB pipeline failures, is also provided. It is expected that the paper will be a valuable source of information for designers and consultants undertaking pipeline designs both onshore and offshore.

KEYWORDS: Upheaval Buckling, Uplift resistance, Pipelines, Cyclic Ratcheting

1. INTRODUCTION

Pipelines are commonly buried underground to provide better thermal insulation, mechanical protection and environmental stability. In case of high pressure and high temperature (HPHT) pipelines, a sufficient burial depth is critical to mitigate against the upheaval buckling. The out-of-straightness (OOS) of the buried pipelines combined with the high axial compressive forces induced by the operating conditions causes the pipeline feature to mobilise upwards and this can eventually lead to structural failure known as "Upheaval buckling (UHB)", unless the uplift resistance of the soil is sufficient to resist the upwards movement of the pipeline. Therefore, understanding and correct evaluation of uplift resistance of soil is critical for safe and efficient design of HPHT pipelines both onshore and offshore. Typical upheaval buckling is schematically shown in Figure 1.

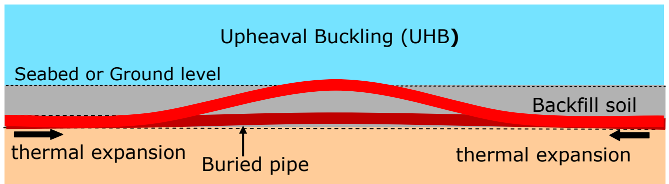


Figure 1 Typical upheaval buckling profile of a buried pipeline

2. OVERVIEW INTO UPLIFT RESISTANCE MODELS

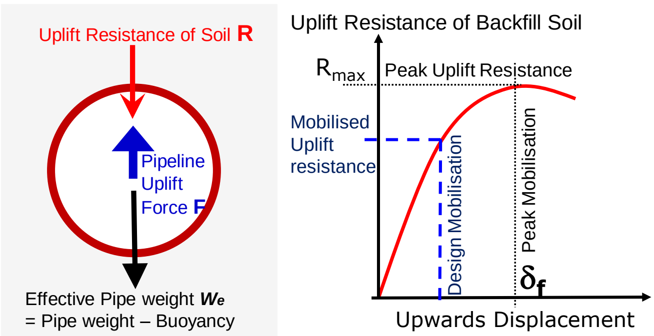
2.1 Basics

There are three vertical forces considered in UHB pipe design (Figure 2(a)) for buried pipelines susceptible for upward mobilization.

- Effective Pipe weight, W_e (Pipe weight – Buoyancy)
- Pipeline uplift force, F, due to operating temperature & pressure
- Uplift resistance of the soil R

It shall be noted that the effective pipe weight is constant and does not change with mobilization. The buoyancy force on the pipeline, whether from water or from soil in case of slurry clay or liquefied soils, is a constant and it is accounted for within the effective pipe weight and hence buoyancy is not part of the uplift resistance of soil.

Typical uplift resistance versus upwards pipeline displacement is shown in Figure 2(b). The pipe upward movement, or mobilisation, of the pipeline to achieve the desired soil uplift resistance is a vital design parameter for safe UHB design. 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 2(b).



(a) (b)

Figure 2 (a) Vertical forces on a buried pipeline experiencing UHB, (b) Soil uplift resistance with upwards pipeline mobilization

2.2 An Overview of Uplift Resistance Models

The present understanding on the uplift resistance of buried pipelines is based on analysis and experimental work by numerous researchers (Randolph and Houlsby, 1984; Pedersen, P.T. & Jensen, J.J. 1988; Selvadurai, A.P. 1989, Maltby and Calladine, 1995) (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; Baumgard, 2000; White et al., 2001; Bransby et al., 2001; and Cheuk et al, 2008; Finch, 1999; Finch et al.2000; Moradi & Craig, 1998, Wang et al.2009, Thusyanthan et al. 2008, Thusyanthan et al.2010, 2011). Thus, there are several analytical models proposed for pipeline uplift resistance in sands and clays. Drained and undrained uplift models are the most common models used in UHB designs. However, onshore pipelines are often buried in partially saturated soils and hence partially saturated uplift model is also applicable for UHB designs (Robert & Thusyanthan, 2018). Figure 3 below provides a summary of uplift models.

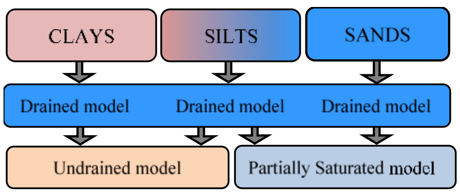


Figure 3 Uplift resistance models appropriate for different soils

UHB assessment in clays (cohesive soils) needs 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 applied 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 or undrained manner depends on the rate of loading primarily with respect to the permeability of the soil.

It is often misunderstood that the pipeline uplift is a relatively rapid phenomenon and only undrained soil response is applicable. This assumption implies that if undrained uplift resistance is enough to resist the pipeline, then the design is safe in the long-term. This assumption is not always correct. When the pipeline is first operational, it is true that the loading cycle take few hours (i.e the temperature of the pipeline increases 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.

Current guidelines do not provide the expected uplift resistance in frozen or thawed sandy backfills. Wang et al. (2011), using centrifuge experiments, compared the uplift resistance of fully-saturated and thawed sand backfills. The results showed that the peak uplift resistance for the thawed backfill case was approximately 12% lower than that of fully-saturated case. This highlights that fact in areas where backfill soils can experience freeze-thaw cycles, the uplift resistance of the backfill is affected by the weather changes.

2.2.1 Drained Uplift Resistance Model

Figure 4 schematically provides the uplift model commonly utilised for drained uplift resistance of a buried pipeline, where;

- R Peak uplift resistance per unit length of pipe
- γ' Submerged unit weight of backfill
- H Backfill height above TOP
- D Outer diameter of pipe incl. coating
- f Uplift resistance factor of backfill

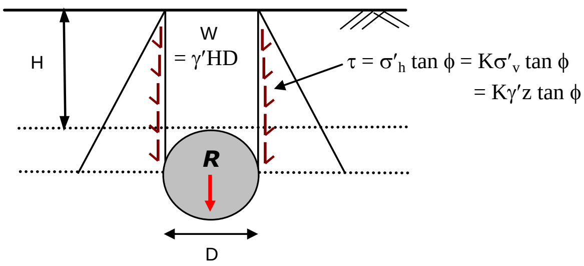


Figure 4 Uplift Resistance model

Figure 5, which is a strain plot from uplift experiment, provides strain fields applicable to vertical slip failure model.

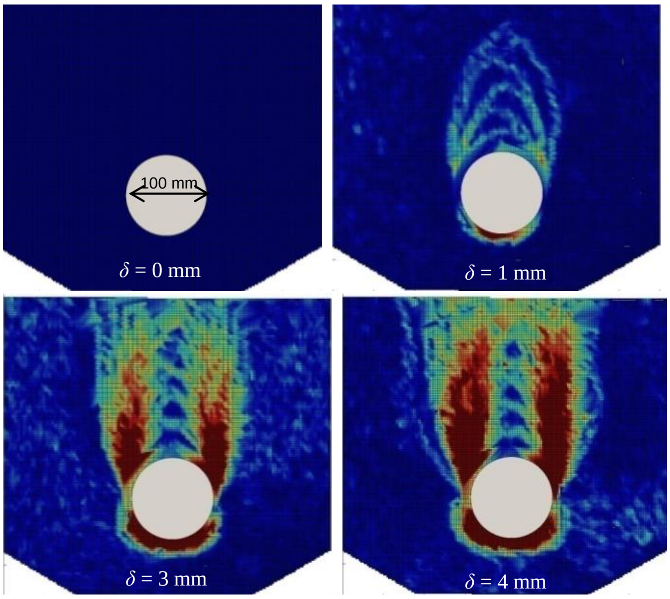


Figure 5 Strain plots from uplift experiment in saturated sands. δ is upwards pipe movement. (Thusyanthan et al. 2010)

There are three different versions of drained uplift model (Figure 4) that are commonly used in the industry. These are summarized below.

Model 1

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

$$\frac{R}{\gamma' \cdot H \cdot D} = 1.0 + f_s \left(\frac{H}{D}\right) \tag{1}$$

where the H is cover to top of pipe and f_s is uplift factor.

Model 2

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

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

where H is cover to top of pipe and f_p is uplift factor. It should be noted that this uplift model uses weight and shear components of the corner soil regions from pipe center line to top of the pipe. The model in Eq. 1 (Schaminée *et al*., 1990) does not account for weight and shear components on top of pipe. Hence, it should be noted that the uplift factor in Eq. 1 and 2 are not interchangeable between the models and should always be used with model from which it was calibrated.

Model 3

ASCE (1984) and ALA (2001) uplift resistance model is based on uplift factor N_{qv} and cover height from center of the pipeline to surface, H_c, as provided below in Eq. 3.

$$R = \gamma' N_{qv} H_c D \tag{3}$$

As the uplift factor for each model is associated with that model, it is important that if uplift model is changed in design then the uplift factor value is also changed accordingly as per Eq. 4 and 5.

$$f_s = 0.1\left(\frac{D}{H}\right)^2 + f_p\left(\frac{H}{D}\right)\left(1+\frac{D}{2H}\right)^2 \tag{4}$$

$$\mathbf{N_{qv}} = \mathbf{1} - \frac{\boldsymbol{\pi}}{\mathbf{8}}\left(\frac{\mathbf{D}}{\mathbf{H_c}}\right) + \mathbf{f_p}\left(\frac{\mathbf{H_c}}{\mathbf{D}}\right) \tag{5}$$

2.2.2 Undrained Uplift Resistance Model

Undrained uplift model is provided below, (DNV-RP-F110).

$$R_{global} = \gamma'.\, H.D + \gamma' \cdot D^2 \cdot \left(\frac{1}{2} - \frac{\pi}{8}\right) + 2.\ \bar{s}_{ur}.(H + \frac{D}{2}) \tag{6}$$

Where S_u is the average undrained shear strength along the failure surface.

When a pipeline is buried deep (typically H/D >4), the flow around mechanism governs the peak uplift resistance. The peak uplift resistance of flow around mechanism can be calculated using the equations below.

$$R_{local} = \eta.\ N_C\,.s_{ur}\ .\ D \tag{7}$$

$\boldsymbol{\eta}$ Empirical factor based on field and model tests usually ranges from 0.55 - 0.8

Nc Theoretical bearing capacity coefficient calculated as follows,

$$N_C = 2\pi\left[1 + \frac{1}{3}\arctan\left(\frac{H + D/2}{D}\right)(1 + r)\right] \tag{8}$$

r Roughness factor for pipeline surface (0-1); r=0 for a smooth surface

The above equation is valid up to *(H+D/2)/D* ratio of 4.5. At deeper burial, the smooth and rough cases are approximately bounded by N_c values of 9 and 12, respectively.

For pipeline in cohesive soils, the upheaval buckling design needs to evaluate both drained and undrained uplift resistances. Thus, if the pipe is buried in CLAY soils, then the peak uplift resistance needs to be calculated using the following equation (DNV-RP-F110).

$$R = \min(R_{global}, R_{local}, R_{drained}) \tag{9}$$

3. PEAK MOBILISATION DISTANCE

3.1 Peak mobilisation in saturated and dry soils

Peak mobilisation distance in uplift resistance has been reported in many past research including DNV RP F110. The DNV (DNV RP F110) states, *"The uplift resistance R_{max} is assumed to be fully mobilised at a vertical uplift displacement δf, where δf is 0.005-0.01 times the height H. Note that δf seems to be independent of the ratio of H/D*". The above statement in DNV is mainly based on past research results which were mainly from laboratory experiments with a soil cover of less than 0.5m and pipeline diameter of typically less than 100mm. The research output was then reported as a cover range of H/D of 5-6. These results were then used to generalise the peak mobilisation distance as a ratio of H.

Thusyanthan et al. (2010), using large scale experiments of cover heights in excess of 1m, demonstrated that the peak mobilisation is much greater than mostly reported in past research based on small scale experiments and DNV guidelines. 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 by 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 (Eq. 10) to predict the peak mobilisation distance (δf) in loose sands in terms of H and D.

$$\frac{\delta_f}{D} = 0.02\, e^{\left[\frac{1}{2}\frac{H}{D}\right]} \tag{10}$$

The above equation is based on the available test data that included test data from dry, moist & submerged sand. Hence, any effect of soil saturation (dry, moist & submerged) on mobilisation is not distinguished in this equation.

Almost all the past research experiments on uplift resistance were based on cover soils of less than 0.6 m due to practical reasons. At small cover depths, the soil dilatancy is higher. Thus peak uplift resistance would be reached at smaller mobilization due to high soil dilatancy contribution. The dilatancy contribution with depth can be demonstrated by viewing the angle of dilation versus depth for sands. The angle of dilation under plane strain conditions can be assessed using Bolton's formulation (Bolton 1986) and has been shown in Figure 6. It is clear from the figure that under typical laboratory experiments in which the cover height is likely to be less than 0.5 m, the dilation angle is much higher than at typical field cover heights. This is one of the key reasons for laboratory experiments to measure peak uplift at smaller mobilizations. Furthermore, the burial depths of oil and gas pipelines in the field are much greater than 0.5 m. For onshore pipelines, ASME B31.8 (ASME 2012) provides minimum cover depth to be 0.6 to 0.9 m depending on the location, and similarly PD8010-1 (British Standards 2004) states the minimum cover as 0.9 m. For offshore pipelines, it is often in the range of 1.5–2.5 m, where the minimum burial depth is mostly determined by protection and mitigation for upheaval buckling requirements. Thus, most of the published pipeline uplift experimental results, which are mainly based on shallow cover depths (Williams et al. 2013), are not directly applicable for field application and the results of such experiments should not be extrapolated.

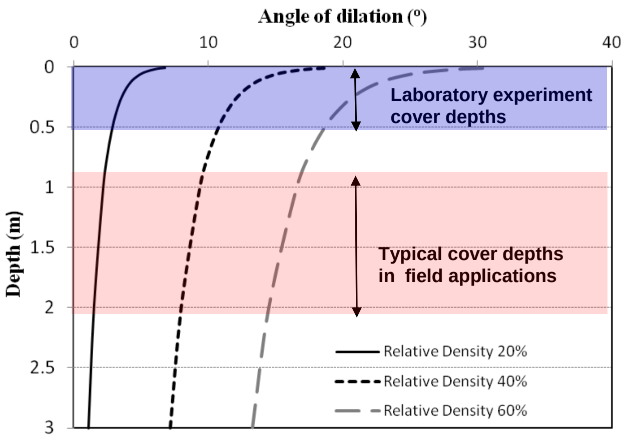


Figure 6 Variation of dilatancy with depth

A comprehensive data for peak mobilisation from experiments and FE results on peak mobilisation were presented by Robert and Thusyanthan (2014) and are re-presented in Figure 7.

3.2 Peak mobilisation in partially saturated soils

Peak-mobilization for pipes buried in partial saturation condition can be significantly different compared to pipes buried under dry/fully saturated conditions. There is incorrect tendency among scientific community to assume that the peak mobilization under partially saturated conditions due to suction can be smaller than dry/fully saturated conditions, thus biased to use the existing models developed under dry/saturated soils for conservative USB designs. However, recent studies by Robert and Thusyanthan (2017) showed that the peak mobilizations under unsaturated soil conditions can be higher than that in dry/fully saturated soil conditions. The study included two-dimensional finite element (FE) analyses conducted on the basis of steel pipeline buried in unsaturated finer sand (G_s=2.65, e_{max}=0.95, e_{min}=0.50 and D_{10}=0.13mm) with advanced constitutive model to capture the behaviour of partially saturated soil behavior (Robert, 2017). Results showed a clear dependence of saturation on the peak mobilizations (Figure 8). For pipes buried under shallower depths (i.e. H/D<5.7), peak mobilization under unsaturated condition was less than that under dry condition when degree of saturation was ~20%. This is due to suction induced high strength and stiffness of sand under low confining stresses. However, peak mobilizations of pipes buried at higher embedded depths under similar saturations exceed the mobilizations under dry condition. This is due to the over shadowed effect of suction by larger confining stress. These increased mobilizations under partially saturated backfill soil condition need to be captured in design to ensure that the pipeline integrity is not compromised by potential UHB failure.

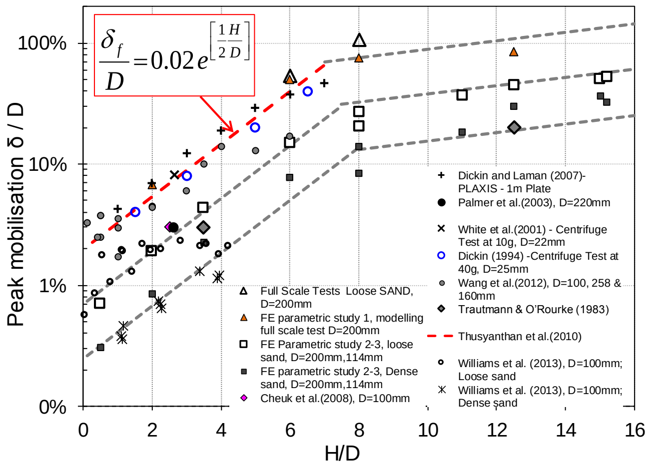


Figure 7 Summary of peak mobilization in uplift resistance (Results extracted from Robert & Thusyanthan 2014)

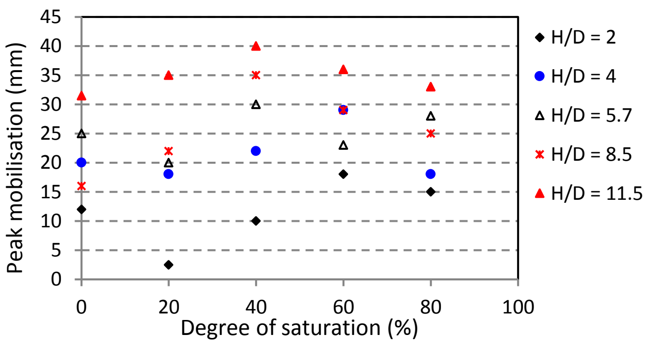


Figure 8 Peak mobilization vs degree of saturation for pipes buried at various depths for a fine sand, D= 114mm (Robert & Thusyanthan, 2017)

4. UPLIFT RESISTANCE IN OFFSHORE CLAYS

Offshore HPHT pipelines burial depth is determined by the uplift resistance of seabed backfill soils. In clayey backfills, the rate of uplift movement of the pipeline can affect the uplift resistance. When uplift resistance from backfill clay soil is insufficient to mitigate the UHB, rock dumping on top of backfill or on top of pipeline is often undertaken to increase the uplift resistance. Therefore, understanding the uplift resistance in backfill clays and rock-dump is essential for such designs.

Thusyanthan et al. (2008) provided uplift resistance results in offshore clay backfills with and without rock-dump based on centrifuge testing. The centrifuge tests were carried out at 30g using natural marine clay. The natural clay samples from offshore were characterised and reconstituted before testing. Field backfill conditions were simulated close to reality in the testing. In each of the tests, the resistance of soil cover, the vertical pipe displacement, and excess pore pressure changes at the pipe invert were measured. The backfill clays were from seabed with undrained shear strength of about 4 - 5 kPa at the mudline. Table 1 provides two test data from Thusyanthan et al. (2008).

Table 1 Tests Summary

Backfill cover	**Backfill Consolidation time**	**Peak Uplift Resistance R (kN/m)**
1m of CLAY	2 months	3.25 (slow) 4.75 (fast)
1m of Rock on top of 1m of CLAY	1 month CLAY & 1 month Rock	9 (slow) 12 (fast)

Figure 9 and 10 provides clear evidence that the suction below the pipeline plays a key role in the uplift resistance in clayey backfill. The magnitude of the negative excess pore pressure depends on the rate of pipeline upwards movement.

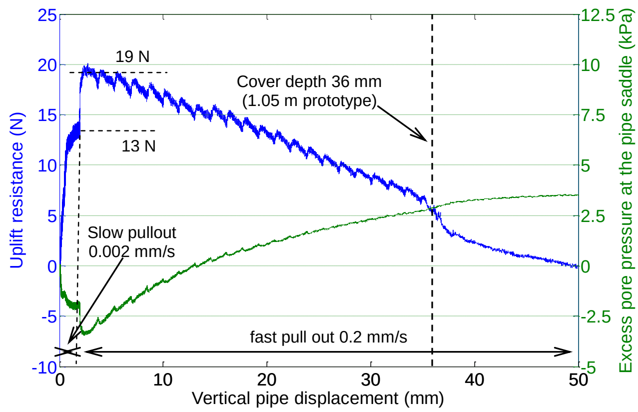


Figure 9 Uplift resistance of a 8.7mm (0.261m in prototype) under clay backfill (30 g centrifuge test data provided in model scale), data from Thusyanthan et al. (2008)

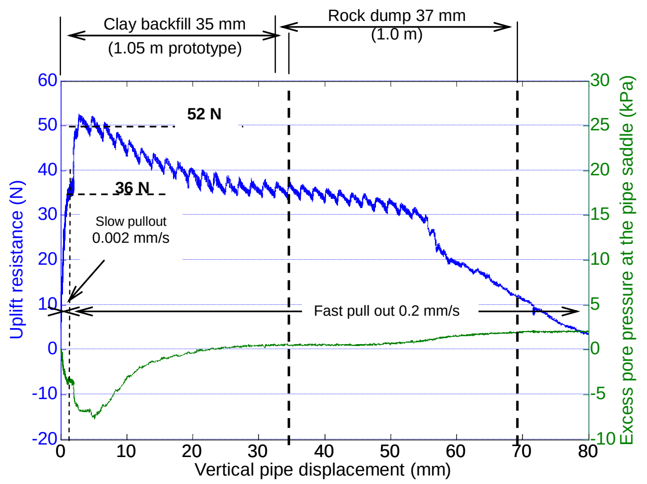


Figure 10 Uplift resistance of a 8.7mm (0.261m in prototype) under 1m clay and 1m rock backfill (30 g centrifuge test data provided in model scale), data from Thusyanthan et al. (2008)

Figure 11 summarises the test data together with the various components that make up the uplift resistance. It is evident that the shear resistance of the rock is not mobilised in the experiments. Thus if rock dump is placed on top of soft clay backfills, the shear resistance in the rock is not recommended to be included in the design unless project specific testing has shown it can be included.

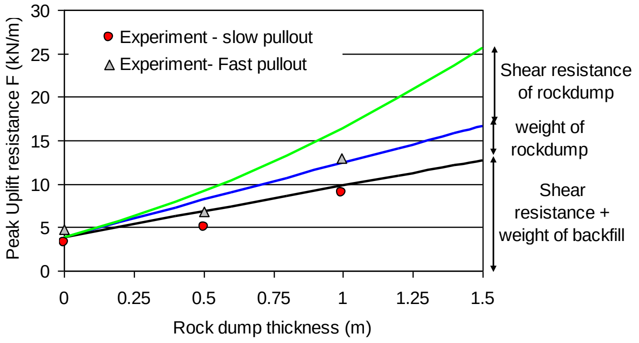


Figure 11 Variation of uplift resistance with rock-dump thickness. (submerged weight of clay backfill $\gamma' = 6$ kN/m^3, submerged weight of rock $\gamma'_r = 10$ kN/m^3, clay cover $H_b = 1.05$ m, uplift factor of clay backfill $f_p = 0.25$, uplift factor of rock $f_{rp} = 0.4$, D = 0.261), data from Thusyanthan et al. (2008)

The uplift resistance factor to be used for offshore blocky clays is variable and often project specific. Wang et al.(2009) has provided a data base of uplift factors that were experimentally obtained for blocky clays. The backfilling method plays an important role in resulting uplift factor. Seabed soil classification is also critical in correctly predicting nature of backfill soils. It shall be noted that for the same soils, use of different soil classification standards can results in different soil classification. Thusyanthan (2012) has provided a comprehensive summary of different soils classifications and how same soils can be classified differently by different classification standards.

5. CYCLIC RATCHETING IN SANDS

High Pressure High Temperature (HPHT) pipelines operate under extreme operating conditions than the surrounding ambient conditions, causing the pipeline to expand, while the backfill on top of the pipeline restraints its movements. These pipelines are installed at ambient temperatures and out-of-straightness features are unavoidable under buried conditions. As the pipeline tend to expand under high axial forces, any OOS features present in the pipeline bedding surface leads to upward mobilizations, leading to upheaval buckling failures unless the backfill material provides sufficient resistance against the upward pipeline movement. If the pipeline upwards mobilization is beyond a critical limit, the gap underneath the pipeline is filled up by soil particles from around the pipeline. Thus, when pipelines cool down during temporary suspension, it cannot return to their original position. As pipeline undergo many thermal cycles of start-up and shut-down during the lifetime, pipeline can progressively move upwards in steps during each of the stop/start cycles. This phenomenon is known as pipeline ratcheting.

This incremental upward movement of the pipeline, if not mitigated, would eventually lead to the UHB failure of the pipeline. Thus, ratcheting is the fundamental driving mechanism behind most UHB pipeline failures in sands. Current pipeline guidelines such as the DNV-OS-F101, DNV-RP-F110 and DNVGL RP F114, while stating that ratcheting should be addressed in design, do not offer a comprehensive criterion for design against ratcheting. Hence the conventional design wisdom is to avoid it completely by limiting the mobilization, typically 20mm in sands. However, detailed understanding of the ratcheting is still not available.

5.1 Uplift Ratcheting Model in Cohesionless Soils

The out-of-straightness (OOS) features in a buried HPHT pipeline which has mobilized upwards during operating conditions will tend to move downwards during shutdowns as the pipelines tend to shorten axially. In the absence of axial pipe-soil friction and soil migration underneath the pipe, the pipeline should return to its original as-laid position, creating an average downward bearing pressure on the soil equivalent to the submerged weight of the pipe (with contents) divided by the contact area. In reality, however, both axial friction and infilled soil in the gap beneath the pipeline may restrict the pipeline and prevent it from returning pre-operation profile. It is difficult to evaluate the exact bearing pressure of the pipeline on the base soil when it has completely cooled down. Therefore, it is reasonable to assume that net soil resistance always returns to zero during cyclic ratcheting as shown in Figure 12. However, when the pipeline cools down during a shut-down, due to pipeline contraction, there could be a downwards force more than the submerged weight of the pipe (with contents) per unit length. This means that the pipe could push down into the infilled soil during every cycle. Thus ratcheting does not necessarily start due to infilling alone. The ratcheting would only occur if the pipeline is unable to push down into the infilled soil to reach almost the original pipeline profile. This model is demonstrated in Figure 12. Thus, proposed ratcheting framework in Figure 12 is fundamental for ratcheting mitigation designs. The ratcheting initiates when the pipeline mobilisation is beyond δ_{crit} as in the figure.

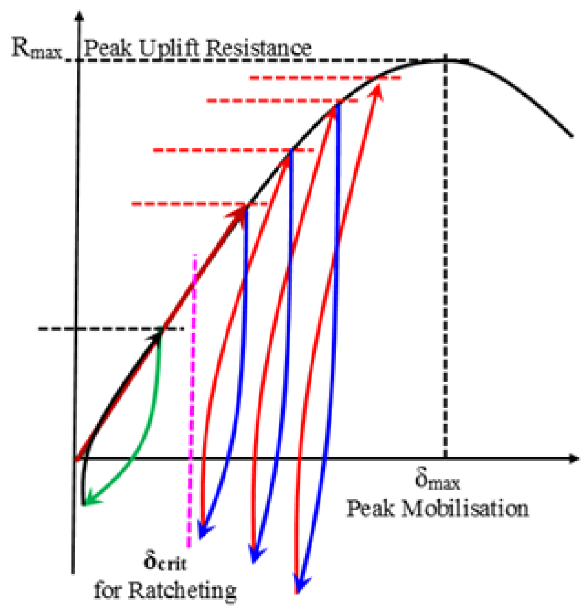


Figure 12 Pipeline ratcheting concept in uplift resistance vs mobilization plot, ratcheting initiates only when pipeline is unable to push down on the infilling to reach its original profile, Thusyanthan et al. (2017)

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.

Thusyanthan et al. (2017) presented a frame work for ratcheting initiation based on pipeline diameter and mobilisation as presented in Figure 13. Based on the limited data available on ratcheting and the presumption that ratcheting is certain to occur at 9%D mobilisation for sand with critical state friction angle of 32°, Three different regimes for ratcheting in relation to mobilisation and pipeline diameter are presented as below.

1. Mobilisation of more than 9%D
   -Zone A- Ratcheting

2. Mobilisation in-between 3%D to 9%D
   -Zone B- Ratcheting possible

3. Mobilisation less than 3%D
   -Zone C- Ratcheting not likely

However, even in Zone A and B, ratcheting does not occur if the pipeline downward force during shutdown is sufficient enough to compress the infilled soil and revert to original pipeline profile. Thus, the amount of possible downwards movement of pipe during shut-down is a critical value. If the pipe mobilisation is below this value, the ratcheting would not occur. This line is named "Pipeline downwards Line (PDL)" and it is shown in Figure 13 with a typical value of 20mm (this value is not fixed). This value is dependent on pipeline properties, operating conditions and OOS feature. Ratcheting does not occur if the mobilisation is below PDL. Thus, the fourth region where ratcheting cannot occur is region below PDL.

4. Region below PDL line -Ratcheting is not possible

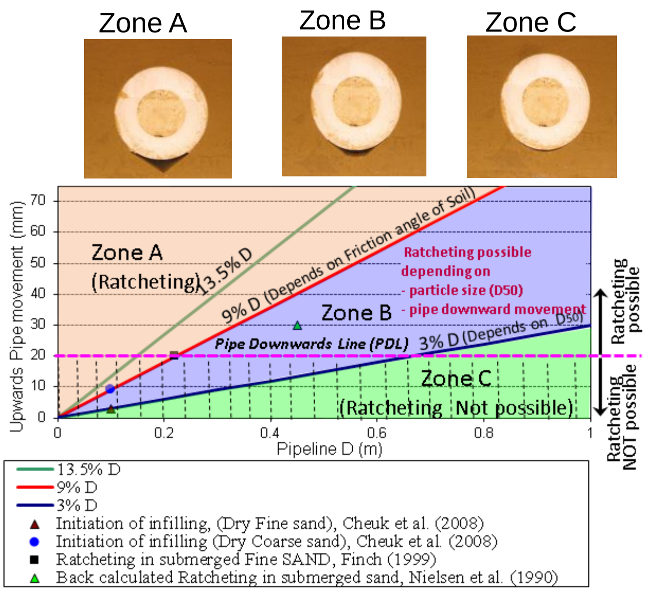


Figure 13 Ratcheting Framework (Thusyanthan et al.2017)

6. PARTIALLY SATURATED UPLIFT RESISTANCE MODEL

Robert and Thusyanthan (2018) proposed an analytical model (Eq. 11 & Table 2) for predicting uplift resistance of pipes buried in partially saturated soils. The model was derived using large scale tests and finite element (FE) analyses which were conducted on the basis of steel pipeline buried in unsaturated finer sand (G_s=2.65, e_{max}=0.95, e_{min}=0.50 and D_{10}=0.13mm) that was simulated using unsaturated Nor-Sand model (Robert et al, 2016). The proposed model is a function of initial soil suction (s), soil relative dilatancy index (I_R, Bolton 1986), effective soil stress at the pipe level (σ'), cover height (H) and pipe diameter (D). As can be seen from the 3-D state boundary surface of the model (Figure 14), the suction induced apparent cohesion effect of finer sands can dramatically increase the uplift resistance of the pipeline when compared to dry condition. The peak dimensionless uplift resistance can increase by a factor of ~1.5-2 in partially saturated sand compared to that of dry condition depending on the initial degree of saturation and relative dilatancy index of the backfill soil. The proposed model can be used to determine dimensionless peak uplift resistance (N_q) for pipelines buried at H/D <11.5 in dry as well as partially saturated sands (similar to tested sands) for $s/\sigma' \leq 2.0$.

$$N_q^{unsat} = \frac{a + b\left(\frac{s}{\sigma'}\right) + c\left(\frac{s}{\sigma'}\right)^2 + d\left(\frac{s}{\sigma'}\right)^3 + e\left(\frac{H}{D}\right) + f\left(\frac{H}{D}\right)^2}{1 + g\left(\frac{s}{\sigma'}\right) + h\left(\frac{s}{\sigma'}\right)^2 + i\left(\frac{H}{D}\right) + j\left(\frac{H}{D}\right)^2} \quad (11)$$

Based on Robert and Thusyanthan (2018), Figure 14 provides an overview of the model in terms of H/D and normalise suction S. Figure 15 presents how the peak uplift factor *fp* varies with degree of soil saturation. It is evident that peak uplift factor is exhibited when the degree of saturation range is 20%-40%.

Table 2 Model constants for the proposed tool to predict uplift resistance of buried pipes

Constant	Value		
	I_R=0.1	I_R=3.0	I_R=4.5
a	0.5044	3.8535	1.2156
b	14.5832	10.9518	50.3785
c	-7.5164	-5.9988	-7.4688
d	0.8595	1.3350	2.6580
e	0.6173	-0.4025	1.6601
f	-0.0508	0.0123	-0.1265
g	1.0562	0.5202	2.3092
h	-0.4501	-0.1699	0.1374
i	-0.0690	-0.1402	0.0544
j	-0.0006	0.0054	-0.0094

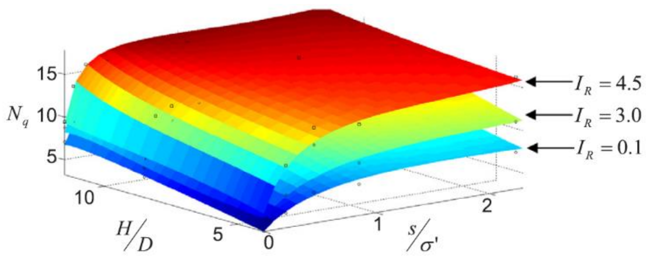


Figure 14 Proposed analytical model response for uplift pipeline behaviour in sandy soils, Robert and Thusyanthan (2018)

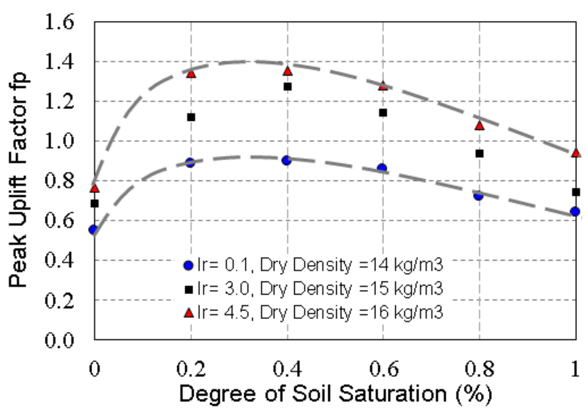


Figure 15 Effect of degree of soil saturation on peak uplift factor fp (equation 2), Robert and Thusyanthan (2018)

7. CONCLUSION

High pressure and high temperature (HPHT) pipelines can fail by upheaval buckling unless proper design has been implemented to mitigate UHB failures. This paper has provided the state-of-the-art knowledge on the uplift resistance of buried pipelines. Common uplift models used in design on sands and clays have been presented together with insight into effects of partial saturation in uplift resistance in sands. These models can be used within probabilistic modelling which can consider variations of multiple influencing factors. The UHB design shall ensure that mobilisation utilised in design is less than δ_{crit} for ratcheting initiation as shown in Figure 12. Furthermore, the uplift resistance used in design needs to be corresponding to the design mobilisation. It is recommended to utilize the outlined approaches in this paper for reliable and safe UHB designs.

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