

RELIABILITY ANALYSIS OF UPHEAVAL BUCKLING OF OFFSHORE PIPELINES

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ABSTRACT

Offshore pipelines are commonly buried in seabed for protection against damage, for better insulation and to prevent upheaval buckling induced by thermal and pressure loadings. The uplift resistance provided by the backfill soil is an important design parameter when determining the correct burial depth for a given pipeline. In this paper, the effect of variability in soil backfill stiffness and operation conditions on the performance of the pipeline upheaval behaviour is investigated. Variations in the soil backfill stiffness, pipe properties and the operational factors such as temperature and pressure are considered to assess the safety of the pipeline probabilistically. An optimized Latin Hyper Cube (LHC) sampling technique is used to draw the sample of soil stiffness, pipe properties and operational conditions from pre-assigned probabilistic distribution for each variable. Pipeline behaviour was simulated using elastic model, and the interaction was modelled using pipe-soil interaction elements using ABAQUS. The response surface method was used to establish approximate functional relationship between the input parameters and the output response. Reliability analysis of pipeline was performed using first order reliability method and simulation method. The results presented are useful to better understand the performance of offshore pipeline and probabilistic upheaval buckling assessment.

Keywords: offshore pipe, upheaval buckling, seabed-pipe interaction, probability, optimized Latin Hyper Cube.

1 INTRODUCTION

Offshore pipelines are major transportation systems in oil and gas field. The consequences of pipeline failures may not only result in heavy economic losses due to system shut-downs and repairs, but can also damage the environment due to loss of containment of oil or other chemicals. It is thus important that offshore pipelines are designed and operated safely (Jones *et al.*, 1991).

Offshore pipelines are often buried by ploughing or jet trenching. The ploughing or jet trenching operation cannot create imperfection free (flat) trenches, thus the pipelines will always have imperfections such as shown in Figure (1). These imperfections act as the triggering points for the upheaval buckling (Thusyanthan *et al.*, 2010).

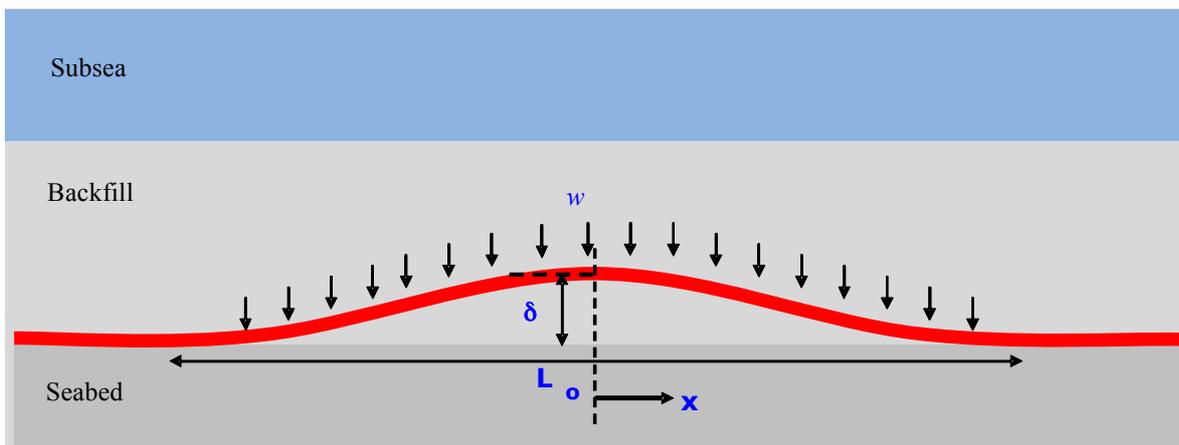


Figure 1: Schematic diagram of buried offshore pipeline profile.

The field observed feature lengths (L_o) and feature heights (δ) from several offshore projects reported in Thusyanthan *et al.* (2011) is shown in Figure 2.

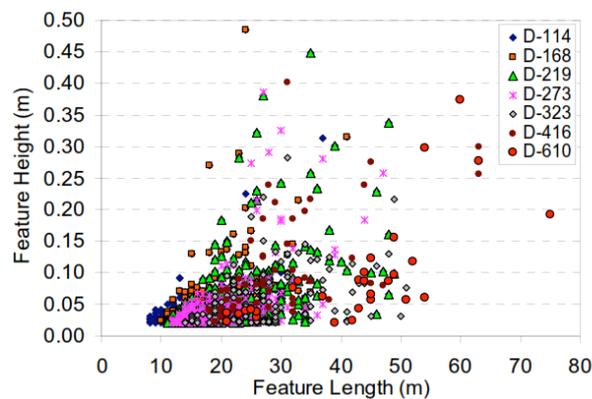


Figure 2: Observed imperfection feature properties in seabed profile on the basis of pipe diameter. (after Thusyanthan *et al.*, 2011)

The DNV RP F110 is the common industry recommended practice for designing offshore pipeline against global buckling. The DNV uses the mean values of soil uplift resistance and driving force (i.e., on effective axial load) in the design process as a deterministic method. The variability in the soil resistance and force is reduced by applying the partial safety factors for both on soil resistance (γ_{UR}) to reduce the available soil uplift resistance and axial effective load factor (γ_{UF}) on driving force. The appropriate values for partial safety factors (γ_{UR} and γ_{UF}) should be used in the design phase to increase the safety and the factors depend on the accuracy of field measurements and the targeted safety class. This conventional deterministic method is simple and straightforward but does not take into account the variability in appropriate manner (Schweiger *et al.*, 2001). Thus the methodology does not explicitly consider the effect of variability in backfill stiffness or operational conditions in the safety assessment against upheaval buckling. These aspects can be examined by probabilistic approach considering the variability in the inputs and assessing their effects on the overall upheaval buckling behaviour.

On the other hand, in a probabilistic approach, the input parameters and loading are treated as continuous random variables and the performance of the structure resulting from different failure criteria is expressed in probabilistic framework as probability of failure (P_f) and/or reliability index (β). The development of reliability based design procedures has received considerable attention in the past. Several researchers recognised the importance and applicability of probabilistic approach in the engineering design and assessment (e.g., Babu and Srivastava, 2010 and Amirat *et al.*, 2009). In offshore pipeline, the most of the researches in probabilistic approach were focused on the effect of pipe corrosion on the performance of the system (e.g., Caleyó *et al.*, 2002) and risk-based maintenance of offshore oil and gas pipelines (e.g., Dey, 2004). There are not many studies found in the probabilistic approach applied to offshore pipeline, especially, on upheaval buckling design and assessment.

In this paper, the effect of soil stiffness, pipe properties and operational conditions uncertainties on the performance of offshore pipeline against upheaval buckling was investigated. The variability in the controlling parameter was modelled using probabilistic theory. The numerical model was developed in ABAQUS to simulate the effects of parameters' variability on upheaval resistance. Pipeline behaviour was simulated using elastic model, and the interaction was modelled using pipe-soil interaction elements. The response surface method was used to develop the response predictive tool using the numerical simulation results. The first order reliability method and simulation methods were used to obtain the reliability index and probability of failure of the pipeline system. Finally, the procedure to assess the safety of the pipeline was demonstrated using case studies.

2 PIPELINE CONSIDERATION

The properties of the pipeline which is considered for the current study is summarised in Table 1. The cover height of rock and clay backfill was designed on the basis of the safety factors of 1.5 and 1.8 respectively. It should be noted that higher safety factor is used for clay backfill design due to larger uncertainties associated with clay backfill soil than for rock. For these selected cases, cover heights to resist UHB in clay and rock were determined as 1.4 m and 0.8 m respectively.

Table 1: Pipeline properties

Parameter	Unit	Value
Pipeline Outside Diameter	m	0.900
Steel Pipeline Wall Thickness (mean)	mm	25
Steel Pipeline Density	kg/m ³	7850
Modulus of Elasticity - mean (at ambient)	GPa	204
Poisson's Ratio	-	0.3
Thermal Expansion Coefficient	C°-1	11.7x10-6
Pipeline Length	km	0.5
Corrosion Coating Material Thickness	mm	3LPP 0.25 (FBE) 0.25 (Adhesive) 2.50 (Polypropylene)
Density	kg/m ³	1440 (FBE) 926 (Adhesive) 920 (Polypropylene)
Concrete Coating Thickness	mm	101.6
Density	kg/m ³	3000

3 FINITE ELEMENT MODELLING OF PIPELINE

A series of finite element analyses was conducted using ABAQUS 6.11 to assess the pipeline deformation induced by its operating conditions. The model and its properties are explained in this section.

The tendency of the pipeline to buckle vertically when laid on an irregular seabed was numerically analysed using a 2D finite element model representing 500 m of pipeline. The numerical model was developed using ABAQUS 6.11 and a series of analyses were carried out to study the effect of variability in soil stiffness, pipe properties and operational conditions on UHB. The pipe was simulated elastically using 1.0 m long ABAQUS pipe elements PIPE21 (Abaqus, 2011).

The seabed surface was represented by a contact surface interacting with the pipe elements. The interaction between these elements was defined as a soft contact. The frictional behaviour between the pipe and the seabed was modelled as elastic perfectly plastic, with an effective friction factor of 0.5. This friction factor is applied to the combined load of instantaneous weight of the pipe and any applied overburden due to cover material used in pipe burial.

The resistance of the soil on top of the pipeline was simulated by elastic springs, resisting only upwards displacements, and a distributed load applied directly to the pipe elements. The behaviour of such springs (i.e., the pipe-soil interaction) is defined by the force per unit length at each point along the pipeline, caused by relative displacement between that point and the point on the far-field boundary.

3.1 ANALYSIS PROCEDURE

The analyses were undertaken by applying the following sequence of loads and boundary conditions:

Step 1: The pipe system was laid on a flat rigid seabed under flooded condition.

Step 2: The middle of the pipeline was raised by a prop from the flat seabed to the required imperfection height (the imperfection height is assumed as 0.5 m in this study).

Step 3: The imperfect seabed (with a geometrical configuration of 0.5 m imperfection as defined in Figure 3) was raised by applying a series of prescribed displacements according to an assumed auxiliary surface.

Step 4: The contact constraints between the pipe and the imperfect seabed surface was activated. As previously described, the interaction between the pipe and the imperfect seabed was defined with a soft contact in the direction perpendicular to the seabed surface and with a specified friction in the direction of the pipe axis.

Step 5: The prop support at the pipeline end was released and the pipeline was transferred to the imperfect seabed surface. As this is a soft contact, the imperfection height was reduced to a certain extent. At the same time, for the design and operating conditions, the flooded submerged weight of the pipeline was replaced by the operating submerged weight and the weight of the soil/rock cover was applied on top of the pipeline.

Step 6: The springs simulating the uplift resistance of the soil cover were then activated. In effect, this implied that at this stage, the vertical displacements in the soil springs were zeroed.

Step 7: The effective driving force, the temperature increase and the pressure were applied to the pipe to obtain the vertical deformation of the pipeline (see Figure 4).

A series of FE analyses (1000 in total) was conducted using the aforementioned UHB model to consider the variability in soil, pipeline properties as well as its operating conditions.

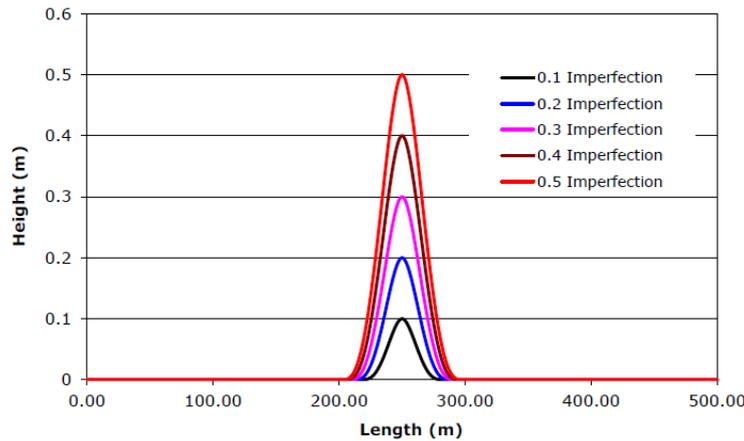


Figure 3: Irregular seabed imperfections.

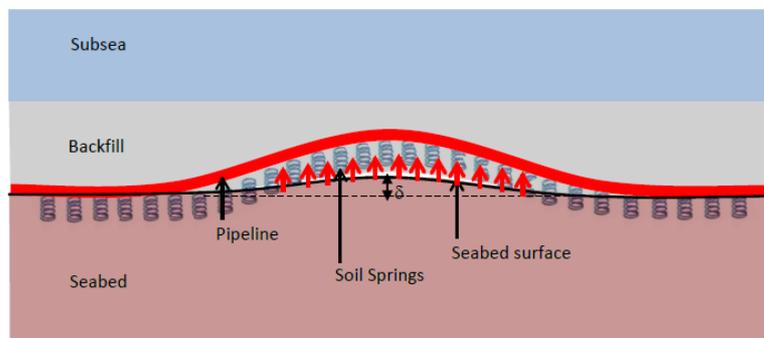


Figure 4: Schematic representation of the pipe in FE model.

3.2 BEHAVIOURAL MODEL FOR SEABED SOIL

Preliminary finite element assessments were also conducted to investigate typical load-displacement response of buried pipes in soft (i.e., clay) and hard (i.e., rock) soils. Here the soil was modelled using modified Mohr-Coulomb model which can capture the strain softening of soils during upheaval pipe displacements. Figure 5a shows a typical shear band formation induced by upward moving pipelines, obtained from FE analyses. After some parametric study, the behavioural model for pipeline upheaval responses in clay and rock were obtained as summarised in Figure 5b. Accordingly, in current study, it is assumed that the maximum resistance was achieved at 20 mm and 60 mm for a pipeline placed in clay backfill and rock respectively (as shown in Figure 5b). Moreover, it is also assumed that the peak mobilisations are not related with the initial state of the soils as was assumed in DNV RP F110. The assumed peak mobilisations are in line with the DNV RP F110 recommendations.

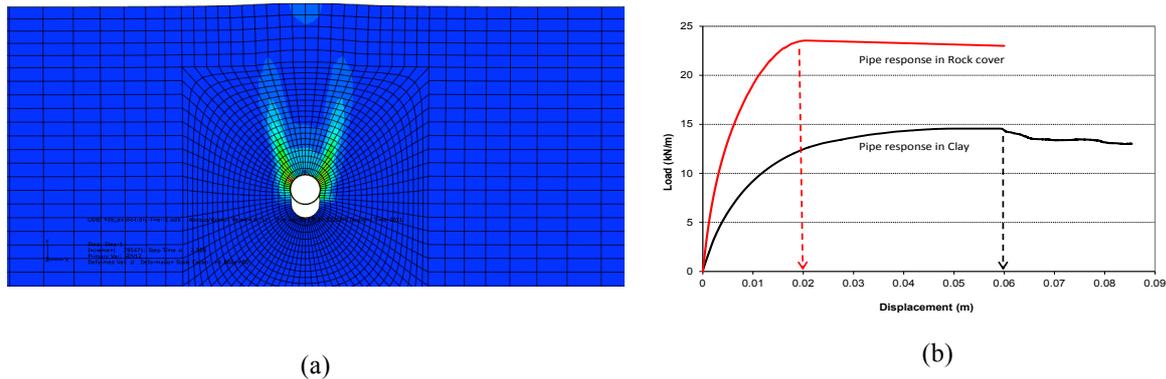


Figure 5: (a) Shear band formation induced by upward moving pipeline, (b) typical behaviour model for pipeline upheaval response

4 COMPUTATION OF PROBABILITY OF FAILURE AND SAFETY ASSESSMENT

Reliability is defined as the probability of success of a system under a given loading condition. In design, the reliability of a system component is evaluated with respect to one or more limit states. For example, in this problem, the limit state can be defined in terms of vertical deformation along the pipeline. Let's assume that the system is described by a set of basic variables x , (e.g., pipe material properties, soil stiffness and operational condition). The possible realisation of x can be separated into two sets on the basis of the considered limit state, namely the safe set for which the system is safe and the failure set for which the system fails or defined to be in a failure state. The surface separating the safe set and the failure set in the space of basic variables is denoted the limit state surface $G(x)$, and the probability of failure can be defined with respect to the limit state surface as given in Eq. (1).

$$P_f = P(G(x) \leq 0) = \int_{G(x) \leq 0} f_x(x) dx \tag{1}$$

where $f_x(x)$ is the joint probability density function for x . This expression is often referred to as the probability integral over the failure set. The complement, $1 - P_f$, is accordingly referred to as the reliability. The corresponding reliability index β is determined by

$$\beta = -\Phi^{-1}(P_f) \tag{2}$$

where Φ is the standard normal distribution function.

The probability of failure and the reliability index can be estimated by a reliability method which can be any amongst several available analytical methods such as first- and second-order reliability methods (i.e. FORM & SORM) as well as the simulation methods (Harr, 1987; Baecher and Christian, 2003).

In this study, both the analytical methods (i.e., FORM) and simulation method (i.e., Monte Carlo simulation using Latin Hypercube Sampling technique) were used to study the reliability of offshore pipeline. The FORM, Monte Carlo simulation and Latin Hypercube Sampling technique are briefly explained in the sections below.

In First Order Reliability Methods (FORM), the limit state function $G(x)$ is linearized to compute the first two moments (i.e., mean and variance) as a function of x . However, in the vast majority of real situations, the limit state function is not linear. In this case, the first two moments of $G(x)$ cannot be determined on the basis of the corresponding moment of x only, and their joint distribution is needed. The computation of reliability index for nonlinear $G(x)$ can be found elsewhere (e.g., Pinto *et al.*, 2004).

FORM requires a simplified relationship for $G(x)$ is required which is some time either difficult to establish or the established relationships are too complicated to perform the analytical reliability methods. The relationship can be established using the concept of response surface method (RMS). Then, the response surface model, with combination of FORM approach, can be used for the calculation of reliability index values using the following expression [Baecher and Christian (2003)]

For uncorrelated normally distributed R and S

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{3}$$

For uncorrelated lognormally distributed R and S

$$\beta = \frac{\ln \left[\frac{\mu_R \sqrt{1 + CoV_S^2}}{\mu_S \sqrt{1 + CoV_R^2}} \right]}{\sqrt{\ln \left[(1 + CoV_S^2)(1 + CoV_R^2) \right]}} \tag{4}$$

where R is the resistance (or capacity) of a system, S is the load (or demand) on the system and CoV is the coefficient of variance.

Alternatively, the family of Monte Carlo simulation (MCS) techniques can be used to estimate the probability of failure. More detail on MCS can be found in Ang and Tang, 1984. The advantage is that the methods are relatively simple and easy to execute, and give a solution which converges towards exact results when a sufficient number of simulations are carried out. A drawback is the excessive computational time required, especially if small failure probabilities are to be estimated. In order for the estimate of P_f to be sufficiently reliable, it is required that the sample dimension be of the order $10 P_f^{-1}$.

5 PROBABILISTIC MODELLING OF PIPE AND SOIL PROPERTIES AND OPERATIONAL CONDITIONS

The elastic modulus of backfill soil (E_s), pipe wall thickness (t), pipe elastic modulus (E_p), operational temperature (T) and pressure (P), product density (γ_p) are considered as uncertain variables. Table 2 shows the respective mean values and corresponding coefficient of variation (CoV) of the parameters together with assigned probability distribution on the basis of published literature (Babu and Srivastava, 2010; and Rajeev and Tesfamariam, 2012, b). The response of pipeline is affected by the uncertainties associated in soil and pipe properties and operational conditions. The uncertainty in the response of pipeline can be derived using several methods such as computationally intensive Monte Carlo simulation with random sampling. However, the uncertainty in the response can be efficiently quantified using Optimized Latin Hypercube Sampling (OLHS) (Park, 1994). OLHS provides a stratified sampling scheme rather than the purely random sampling, as it provides more efficient means of covering the probability space. For a unit hypercube of dimension n (number of uncertain parameter) that contains N_s (number of simulation) data points, there are N_s sub-cubes each that has $1/N_s$ volume. Thus, the side length of each sub-cube is $\sqrt[n]{1/N_s}$, which is selected as the minimal inter-point distance. In this study, 6 number of uncertain parameters (n) (see Table 2), and 500 number of simulation (N_s) are considered. The OLHS is sampled from $N_s \times n$ matrix of independent random numbers $R \in [0, 1]$. Figure 6, for example, illustrates optimised distribution of R -values of uncertain parameters (n_1 - n_2) and (n_1 - n_3).

Table 2: Soil, pipe and operational condition parameter uncertainty.

Parameter	Unit	Distribution	Mean	CoV(%)
Diameter	mm	Fixed	900	-
Thickness	mm	Lognormal	25.0	6.0
Elastic modulus	GPa	Lognormal	204.0	6.0
Soil Stiffness				
Clay	kPa	Lognormal	200	30
Rock	kPa	Lognormal	1600	10
Temperature	°C	Truncated Normal	75 [50 to 120]	15
Pressure	MPa	Normal	20.0	20
Product Density	Kg/m ³	Lognormal	200	10

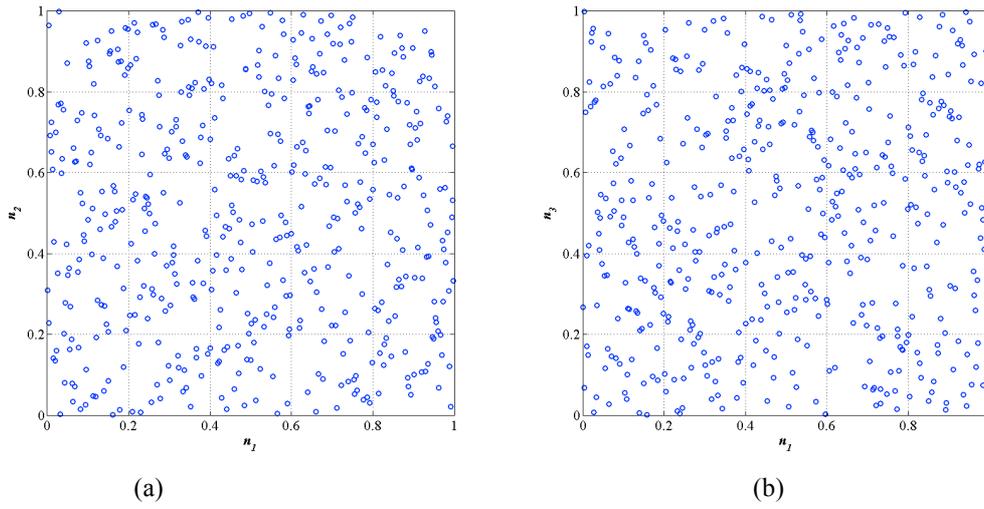


Figure 6: Distribution of uncertain parameters R-value after sampling using OLHS.

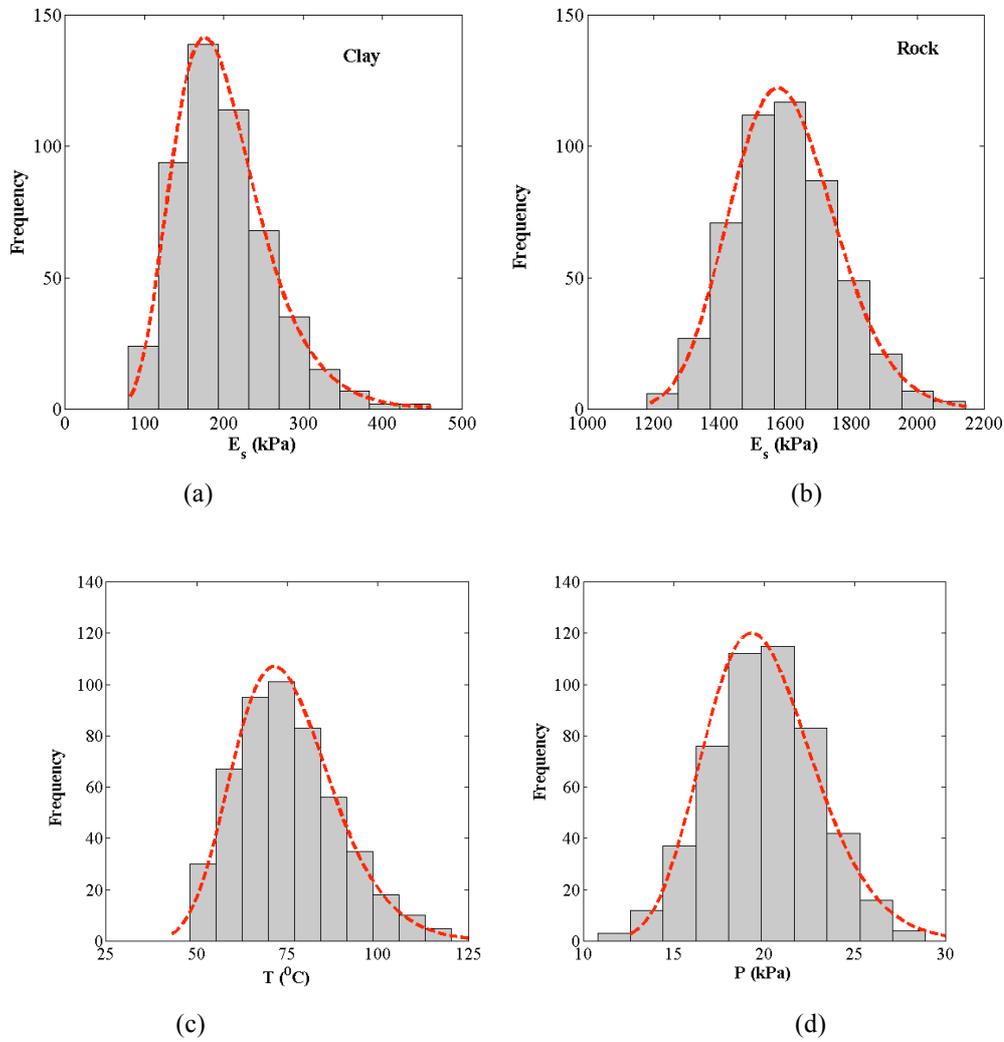


Figure 7: Randomly sampled (a) clay stiffness, (b) rock stiffness, (c) applied temperature and (d) applied pressure.

Each element of uncertain parameters x is then mapped according to:

$$x_{ij} = F_{x_j}^{-1}(R_{ij}) \tag{5}$$

where $F_{x_j}^{-1}$ is the inverse CDF for parameter j . Each row of x contains different set of sampled parameters, from which statistical sample of soil, pipe and operational conditions are obtained and the sample pipe models are generated and subjected to sampled operational conditions. All variables are assumed to be statistically independent of each other, i.e., the correlation between any two variables is assumed to be negligible.

Figure 7 shows the histograms of frequencies of the sampled clay and rock stiffness and operational condition (i.e., temperature and pressure). Also shown are the probability densities from which the values have been sampled.

6 RESULTS AND DISCUSSION

The reliability assessment of offshore pipeline buried in clay soil and rock was carried out using the results obtained from FE analyses. Figure 8 shows typical results (i.e. pipe deformations) obtained from FE analyses for pipe buried in rock under different operating conditions of temperature (T) and pressure (P). The limiting pipe deformation value to eliminate the failure of pipeline is 60 mm and 20 mm for clay and rock respectively (as explained in section 3.2). It can be seen from example FE results shown in Figure 8 that pipe exceeds limiting deformation (20 mm for rock) at the operating temperature of 112 whereas it is in safe operation in the other case.

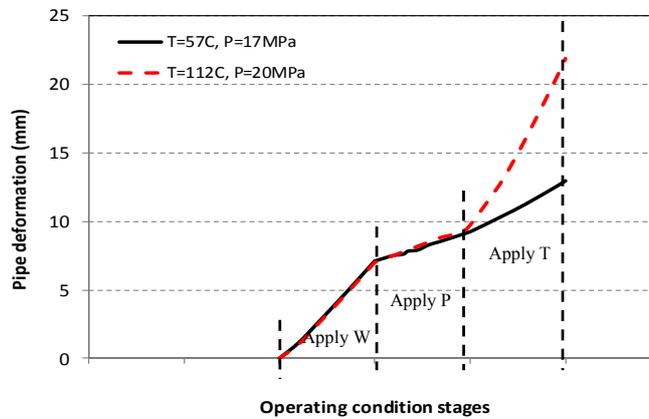


Figure 8: Examples of FE results for pipe deformation in Rock Cover.

6.1 SENSITIVITY ANALYSIS

In order to quantify the relative importance of individual parameters on the response of pipeline considered in this study, the sensitivity analysis was conducted. These analyses help to determine: (1) what parameters require additional consideration for strengthening the response prediction, thereby reducing excessive simulations; (2) what parameters are insignificant and can be eliminated from the simulation; (3) what inputs contribute most to response variability; and (4) what parameters are most highly correlated with the response (Hamby, 1994).

In this study, first-order-second-moment (FOSM) analysis was carried out to perform the sensitivity study. For this purpose, all variables were assumed to be statistically independent of each other, i.e. the correlation between any two variables was assumed to be negligible. Furthermore, the relationship between the response variables and the random variables are approximated to be linear.

Let Z be a random variable representing a response parameter in the following form:

$$Z = h(x_1, x_2, \dots, x_N) \tag{6}$$

The random variable X_i can be characterised by its mean, μ_x and variance σ_x^2 . Now expanding Z using Taylor series and keeping only the first order terms, one can write

$$Z \approx h(\mu_{x_1}, \mu_{x_2}, \dots, \mu_{x_N}) + \sum_{i=1}^N (x_i - \mu_{x_i}) \frac{\partial h}{\partial x_i} \tag{7}$$

Thus, mean μ_Z and variance σ_Z^2 of Z can be approximated as

$$\mu_Z \approx h(\mu_{x_1}, \mu_{x_2}, \dots, \mu_{x_N}) \tag{8}$$

$$\sigma_Z^2 \approx \sum_{i=1}^N \sigma_{x_i}^2 \left(\frac{\partial h(x_1, x_2, \dots, x_N)}{\partial x_i} \right)^2 + \sum_{i=1}^N \sum_{j \neq i}^N \rho_{x_i, x_j} \frac{\partial h(x_1, x_2, \dots, x_N)}{\partial x_i} \frac{\partial h(x_1, x_2, \dots, x_N)}{\partial x_j} \tag{9}$$

where ρ_{x_i, x_j} is the correlation coefficient between x_i and x_j . The partial derivative of $h(x_1, x_2, \dots, x_N)$ with respect to x_i can be calculated numerically using the central difference method as follows:

$$\frac{\partial h(x_1, x_2, \dots, x_N)}{\partial x_i} = \frac{h(x_1, \dots, \mu_i + \Delta_{x_i}, x_N) - h(x_1, \dots, \mu_i - \Delta_{x_i}, x_N)}{2\Delta_{x_i}} \tag{10}$$

In this case, simulations are performed varying each input parameter individually to approximate the partial derivatives as given in Eq.(10). Since the correlation between any two uncertain parameters is neglected, the variance is calculated by ignoring the terms associated with ρ_{x_i, x_j} in Eq.(9).

Figure 9 shows the relative variance contribution of different uncertain parameters for the vertical deformation of pipeline. It is observed from Figure 9 that the variation in the response is controlled mostly by temperature (relative variance more than 75%) in both clay and rock. The deformation also shows considerable sensitivity to pressure. This reveals that the operating conditions have major impact on mobilising the pipe towards allowable limits. Though the variation in soil stiffness properties has less significance compared to operational conditions, it is an important parameter to design the pipeline to avoid failures due to upheaval buckling (i.e., determination of cover height mainly depends on the soil stiffness).

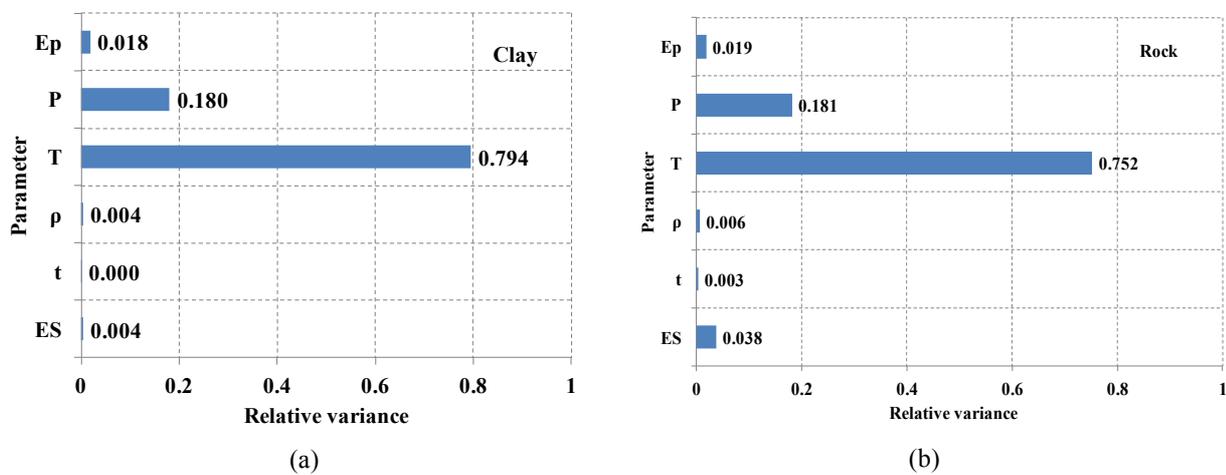


Figure 9: Sensitivity analysis of random parameters on the response (a) clay and (b) rock.

This observation indicates that temperature and pressure need more careful characterisation in order to avoid the failures due to extreme variation in those parameters.

6.2 RESPONSE VARIABILITY AND PREDICTION EQUATION USING RANDOM SAMPLING

As stated above, in this study, the random sample generation is done by Latin hyper cube method following Stein’s approach (1987). Latin hypercube method generally requires less number of realisations in comparison to the simple random sampling in order to have statistically meaningful results. Using Latin hypercube sampling techniques, unbiased estimate of mean and probability distribution function of the responses can be obtained with a smaller variance, in comparison to the simple random sampling of same sample size. In order to find the right sample size, the numerical analysis was carried out using 500 number of samples. Figure 10 shows the sample size versus the mean and dispersion (i.e., standard deviation) of vertical pipe deformation normalised by values corresponding to the sample size of 500. The figure shows that the mean and dispersion of responses tend to converge as the sample size increases and the total 500 number of simulation chosen is adequate.

Further, the response predictive tool was developed for pipe buried in both clay and rock by applying response surface method using the simulation results. Response surface methods (RSMs) is a collection of mathematical and statistical techniques for solving problems in which the goal is to optimize the response of a system or process using input variables (x_1, x_2, \dots, x_N) , subject to observational errors (Montgomery, 2001). Response surfaces are smooth analytical functions and are most often approximated by linear function (first order model) or polynomial of higher degree (such as the second-order model). The second-order polynomial response surface has the form:

$$y = \beta_0 + \sum_{i=1}^n \beta_i x_i + \sum_{i=1}^n \beta_{ii} x_i^2 + \sum_{i=1}^n \sum_{j=1}^i \beta_{ij} x_i x_j \tag{11}$$

where y is the response parameter (i.e., pipe deformation), and β_0 , β_i and β_{ij} are the regression coefficients. Estimates of the coefficients can be obtained by fitting the regression equation to the response surface values observed at a set of data points.

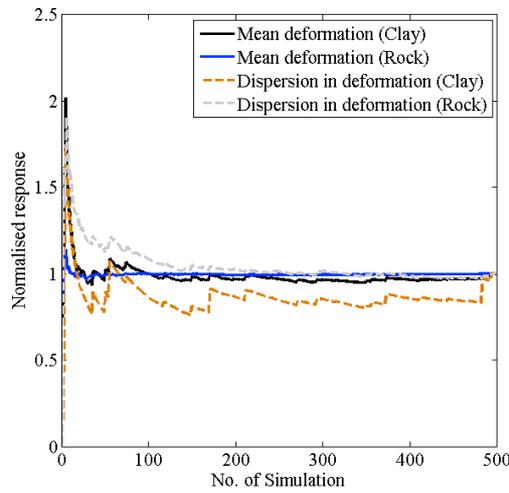


Figure 10: Sample size versus normalised response.

As stated above, a functional relationship between the response and the input variables is required to apply the analytical reliability method. Using the MC simulation data, the first order response surface model was developed from the regression analysis for both the pipe buried in clay and rock. The response surface method has been successfully used in reliability analysis of buried pipe, laterally loaded piles and seismic performance of buildings (e.g., Sivakumar Babu and Srivastava, 2010; Rajeev and Tesfamariam, 2012a). The vertical deformation response predictive tool developed for pipe buried in clay and rock is given below together with coefficient of determination (R^2). The predictive equation for rock shows a very good correlation with the numerical model prediction with R^2 of 0.99. The equation to predict the deformation in clay shows reasonable correlation with the numerical model prediction with R^2 of 0.65. However, the prediction in clay can be improved by using different form of equation.

$$\delta_{clay} = -24.84 - 7.8 \times 10^{-3} t \text{ (mm)} + 4.8 \times 10^{-2} E_p \text{ (GPa)} - 1.8 \times 10^{-2} E_s \text{ (kPa)} + 0.42 T \text{ (}^\circ\text{C)} + 0.58 P \text{ (Pa)} - 4.5 \times 10^{-3} \rho_p \text{ (kg/m}^3\text{)} \quad R^2 = 0.65 \quad (12)$$

$$\delta_{rock} = 2.63 - 4.8 \times 10^{-3} t \text{ (mm)} + 2.46 \times 10^{-2} E_p \text{ (GPa)} - 2.19 \times 10^{-3} E_s \text{ (kPa)} + 0.14 T \text{ (}^\circ\text{C)} + 0.19 P \text{ (Pa)} - 1.06 \times 10^{-2} \rho_p \text{ (kg/m}^3\text{)} \quad R^2 = 0.99 \quad (13)$$

6.3 PROBABILITY OF FAILURE AND RELIABILITY INDEX

Based on the pipe deformation simulated using the numerical model and the random samples, the factor of safety was calculated for pipe buried in clay and rock separately using the corresponding limiting pipe deformation values (i.e., 60 mm for clay and 20 mm for rock). The FOS is determined by dividing the corresponding limiting pipe deformation value by the displacement computed from FE analysis. The distribution of computed factor of safety (FOS) is shown in Figure 11. The FOS varies between 0.60 to 4.15 with the mode of 2.62 for pipe buried in clay and, for the pipe in rock, FOS varies between 0.84 to 1.69 with the mode of 1.26. Figure 11 also shows the fitted normal distribution. From the total of 500 simulations, 6 and 27 simulations lead to the response greater or equal to the limiting deformation (i.e., $FOS \leq 1$) in clay and rock, respectively.

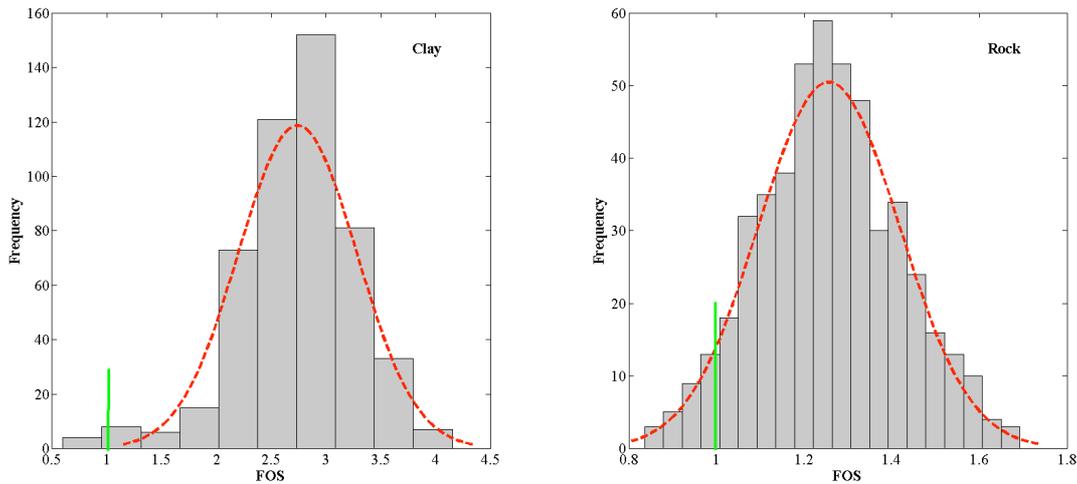


Figure 11: Computed FOS (a) clay and (b) rock.

Using the response predictive tool given in Equations 12 & 13 the mean and the CoV of the limit state function were evaluated as 23.22 mm and 33.64% respectively in clay and the corresponding values are evaluated as 16.19 mm and 13.25% in rock. Therefore, the reliability index computed for allowable limits for deformation (i.e. 60 mm in clay and 20 mm rock) is **3.06** and **1.67** for the pipe buried in clay and rock respectively by assuming the parameters based on the uncorrelated lognormal distribution (i.e. using Equation 4). The corresponding probability of failure is **0.0011** and **0.0475** for clay and rock respectively. The result of reliability index and the probability of failure obtained for pipe buried in clay are above the acceptable limit (i.e., medium safety level). However, for the pipe buried in rock, the reliability index and the corresponding probability of failure indicate that the performance of the pipeline is not in the acceptable limit (i.e. a low safety or in the range of unsatisfactory to poor performance).

Further, a normal distribution was fitted to the computed FOS from the numerical analysis and the cumulative distribution density (CDF) function was derived. The probability of failure was calculated using CDF (i.e., the CDF value corresponding to FOS = 1). However, to quantify epistemic uncertainty associated with the sampling process, 500 samples of FOS each were re-sampled 500 times from the computed FOS using bootstrapping method (Efron and Tibshirani, 1986) and corresponding CDF were generated to evaluate the probability of failure. The procedure yields a sample of P_f from which mean and dispersion can be evaluated. The dispersion is a measure of the uncertainty associated with the sampling. The CDF developed from bootstrapping is shown in Figure 12. The computed mean value of probability of failure is **0.00067** with the CoV of 56.0% for the pipe in clay and **0.054** with CoV of 12.9% for the pipe in clay. The mean value of probability of failure computed using bootstrapping method is very close to the one computed using the analytical method for both clay and rock.

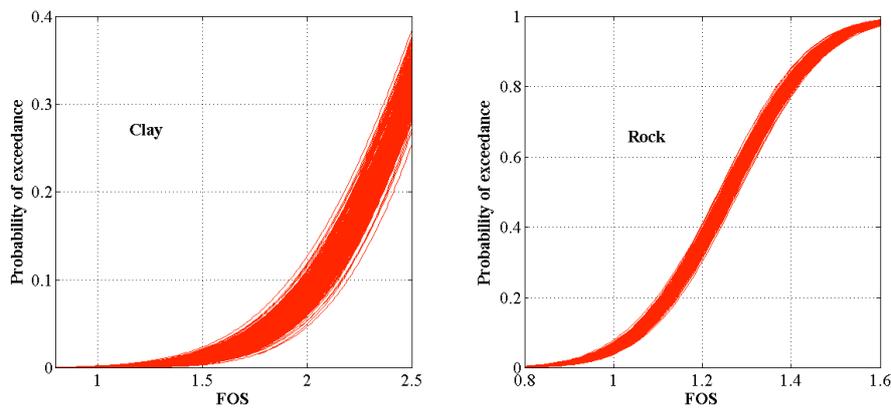


Figure 12: Computed CDF from bootstrapping clay (left) and rock (right).

7 SUMMARY AND CONCLUSION

The reliability analysis of upheaval buckling in offshore pipeline is investigated in this paper. The pipeline buried in clay and rock was considered for the analysis. The variability in soil stiffness, pipe material and geometrical properties and operational conditions were incorporated. The finite element model of pipeline system was modelled and 1000

simulations were carried out. The analysis results show that, even though the stiffness can be dominant in pipe mobilisation (i.e., design consideration), the variability of operating conditions (temperature and pressure) govern the pipe deformation compared to the soil stiffness variation. Soil stiffness, which can be a function of relative density, determines initial pipe peak mobilisation for a given operating condition (hence important to characterise), however, the variation of operating conditions has more significance on pipe mobilisation. Therefore, the accurate variation in operational conditions should be considered in the design process for upheaval buckling. Further, the probability of failure of pipe buried in clay soil is less than in rock mainly due to the higher load factor used for clay in the designing stage (i.e. 1.8 for clay and 1.5 for rock). However, the dispersion of computed probability of failure is higher in clay soil than in rock.

A proper consideration of probabilistic approach and the numerical analysis is useful to examine the performance of offshore pipelines under upheaval buckling in a rational way, which has been demonstrated in the present paper. However, the assumption on the type of distribution significantly affects the reliability of the pipeline performance. It should be noted that the observation and results presented in this paper cannot be directly applicable to other problems. Further research on the effect of variation of soil properties on the upheaval behaviour is needed for better understanding of the upheaval behaviour in different soil conditions.

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