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ON-BOTTOM STABILITY ANALYSIS OF SUBSEA PIPELINES UNDER COMBINED IRREGULAR WAVES AND CURRENTS

Guomin Ji
SINTEF Ocean[1]
NO-7450, Trondheim, Norway

Lanjing Li
Department of Mechanical and Structural
Engineering and Materials Science
University of Stavanger
NO-4036, Stavanger, Norway

Muk Chen Ong
Department of Mechanical and Structural
Engineering and Materials Science
University of Stavanger
NO-4036, Stavanger, Norway

ABSTRACT

The objective of the paper is to study the effect of different parameters regarding on-bottom stability of subsea pipelines under combined irregular waves and currents. The effect of friction coefficient is first investigated. The development of lateral displacement and penetration for three different friction coefficients are compared for sandy and clayey seabed respectively when applied wave and current conditions are kept same for all the cases. The friction coefficient affects the soil resistance force and further changes the initial time when the pipeline starts to move in the lateral direction. The accumulated displacement reduces for large friction coefficient and it results in less penetration. The total effect of the increasing friction coefficient depends on the competition between the increased friction force and the reduced passive soil resistance force. The pipeline usually crosses different types of soil along the route. Hence, different combinations of soil types along the route are applied in the analysis. The soil property at middle of the pipeline is found to be important when the boundary conditions at both ends are fixed. Three analyzing procedures, namely standard 3-hour procedure, the procedure recommended by PONDUS and the procedure recommended by DNV, could be used for on-bottom stability analysis under storm conditions. The comparison of these procedures shows that the procedure recommended by PONDUS is the most appropriate for the storm conditions. The procedure recommended by DNV considers the build-up of initial penetration before the storm; and it could be applied in the analysis when the penetration is stabilized after the start -up time (20% of 3-hour).

Keywords: On-bottom stability, friction coefficient, hydrodynamic load, storm condition and finite element analysis.

INTRODUCTION

After pipelines, umbilical and power cable are installed on seabed they are exposed to a complex pipe-fluid-soil interaction system and the major design criteria is how to ensure their on-bottom stability and it has long been an important study for offshore practice [4] [11].

The failure of on-bottom stability has recently been experienced in the Gulf of Mexico due to hurricanes Katrina, Rita and Ivan and it caused damage of pipeline, extensive lateral displacements (in the order of kilometres), collisions of pipeline with other subsea installations, and considerable loss of production [3].

The general design procedure is to determine a submerged weight capable of withstanding hydrodynamic loads through friction and passive soil resistance [2]. Alternatively stability can be ensured by burial, trenching, continuous rock dumping/covering or tension stabilization by intermittent interventions [10].

1. Earlier MARINTEK, SINTEF Ocean from 1st January 2017 through a merger internally in the SINTEF Group

To minimize costs and environmental impact from burial or seabed interventions, it is highly attractive to optimize on-bottom stability design. On the other hand, examples from the Gulf of Mexico highlight that maintaining safety in operations is essential.

The soil types, waves and current conditions as well as the structure itself are considered to be the main focus of the on-bottom stability analysis for a subsea pipeline. The response of the pipe is non-linear due to non-linear hydrodynamic forces and non-linear interaction between the pipe and the soil. In the past, lateral pipe-soil interaction has been modelled using a simple friction coefficient (also known as the Coulomb model). This model is very simplified and is an unrealistic method to model pipe-soil interaction because it does not consider non-linearity of the soil force, especially in large lateral displacement situations. Several research projects have been conducted in the past to refine the pipe-soil model. They include the model developed by Verley ([8], [9]) and SAFEBUCK Joint Industry Program (SAFEBUCK JIP) [1]. The PONDUS software that developed by SINTEF Ocean (previously known as MARINTEK) is one of the software that utilized the Verley's model. The SAFEBUCK JIP program is primarily concentrated on the lateral soil model during large displacement lateral buckling, and it examines the effect of the soil berm.

METHODOLOGY

PONDUS developed by SINTEF Ocean is a software focusing specifically on the dynamic lateral response of offshore pipeline subject to a combined action of wave and current on a horizontal seabed [5] [6]. The main features of PONDUS software are shown as following:

- Calculates wave kinematics from 3-D irregular waves for medium and deep water
- Calculates hydrodynamic force by load models
    - Morrison
    - Database force model [7]
    - Combination of Morrison and Database force model
- Uses 2-D beam elements with small deflection theory in finite element formulation
- Calculates soil resistance forces by soil models
    - Sand soil [9]
    - Clay soil [8]
- Computes the dynamic response of pipeline subjected to wave and current in time domain

The applied lateral soil model consists of two main components, namely Coulomb friction force and passive soil force. In general, the passive soil resistance is divided into 4 phases according to [2]

1) An elastic region where the lateral displacement is less than typically 2% of the diameter. The upper limit of the passive resistance in this stage is denoted as F_{r1}.
2) A region where significant lateral displacement may be experienced, up to half the pipe diameter for sand and clay soils in which the pipe-soil interaction causes an increase in the penetration and thus in the soil resistance. The upper limit of soil passive resistance is called breakout resistance F_{r2}.
3) After breakout, the resistance and penetration decrease.
4) When displacement exceeds typically one diameter, the passive resistance and penetration may be assumed constant. The soil resistance at this stage is denoted as F_{r3}.

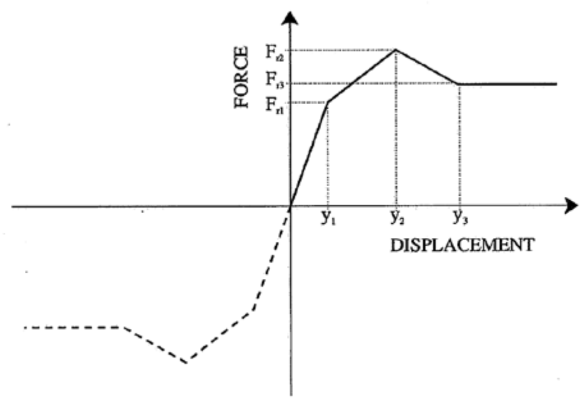


Figure 1 Force-displacement model for passive soil resistance F_r

The main theory of the soil model is described in [6], and the pipe displacement is expressed as sum of the elastic and plastic displacement:

$$\nu = \nu_e + \nu_p \tag{1}$$

Where ν_e – elastic displacement
ν_p – plastic displacement

In the elastic range the soil force is expressed as:

$$\boldsymbol{F_s = k_s \nu_e + \alpha_s k_s \dot{\nu}} \tag{2}$$

Where k_s – elastic soil stiffness (per unit length)
α_s – soil damping constant

In the plastic range the soil force is expressed as a sum of friction type force and a soil remaining force as follows:

$$F_s = F_f + F_r \tag{3}$$

$$F_f = \mu(w_s - F_l) \cdot sgn(\dot{v}) \tag{4}$$

$$F_r = D_s \cdot sgn(\dot{v}) \tag{5}$$

$sgn(\dot{v}) = +1\ when\ \dot{v} > 0$
$sgn(\dot{v}) = -1\ when\ \dot{v} < 0$

where $F_l = w_s$ if $F_l > w_s$
μ - constant friction coefficient (μ ≥ 0)
w_s- submerged weight of pipeline
F_l- lift force found from hydrodynamic force model
D_s- remaining force function

The remaining force function is a function of the plastic displacement and the number of oscillations and is also a function of the lift force.

The transition from elastic to plastic range is defined to taken place when:

$$|k_s v_e| = [\mu(w_s - F_l) + D_s] \quad (6)$$

And the transition from elastic to plastic range is defined to occur when the pipe velocity changes sign, that is when $\dot{v} = 0$.

In the elastic range the incremental form of the soil force is equal to:

$$\Delta F_s = k_s \Delta v + \alpha_s k_s \Delta \dot{v} \quad (7)$$

And in the plastic range

$$\Delta F_s = \frac{f}{1+f} k_s \Delta v - \frac{1}{1+f} \mu \Delta F_l \cdot sgn(\dot{v}) \quad (8)$$

where $f = \frac{\partial D_s}{\partial v_p} \frac{1}{k_s} \cdot sgn(\dot{v}) \quad (|f| < 1)$

The soil force in the plastic range is computed in two steps; first the pipe penetration due to pipeline movement, and then the corresponding force-displacement curve. The force-displacement curve for the penetration dependent force F_r is defined through the force levels F_{r1}, F_{r2} and F_{r3} and corresponding displacements y_1, y_2 and y_3 as shown in Figure 1.

Sand model

Initial penetration z_i for the pipe due to weight, w_s, is given by:

$$\frac{z_i}{d_h} = 0.037\, \kappa_0^{(-2/3)} \quad (9)$$

Where $\kappa_0 = \frac{\gamma_s d_h^2}{w_s}$

γ_s - submerged weight of soil

d_h - hydrodynamic diameter

The model is based on an empirical relation between development of pipe penetration as a function of the work done by pipe-soil interaction. This relation is given as:

$$\left(\frac{z_2 - z_i}{d_h}\right) = 0.23 \left(\xi \kappa_i^{-1} \left(\frac{y}{d_h}\right)^{-1/2}\right)^{0.31} \quad (10a)$$

$$\left(\frac{z_2 - z_i}{d_h}\right)_{\max} = 1.0 \left(\frac{y}{d_h}\right)^{1/2} \kappa_i^{-1/2} \quad (10b)$$

where $\xi = \frac{E}{\gamma_s d_h^3}$, $\quad E(t) = \int_0^t F_r ds$

$\kappa_i = \frac{\gamma_s d_h^2}{F_{ci}}$

y – instantaneous distance of the pipe from the origin

F_{ci} – the value of the vertical soil contact force at the instant maximum horizontal soil resistance, F_{r2}

E –energy (work) done by pipe on soil

The maximum resistance F_{r2} is given as:

$$\frac{F_{r2}}{\gamma_{s d_h^2}} = (5.0 - 0.15\kappa_i) \left(\frac{z_2}{d_h}\right)^{1.25}, \quad \kappa_i \le 20 \quad (11a)$$

$$\frac{F_{r2}}{\gamma_{s d_h^2}} = 2.0\kappa_i \left(\frac{z_2}{d_h}\right)^{1.25}, \quad \kappa_i > 20 \quad (11b)$$

where z_2 – maximum pre-break-out penetration

The increase penetration causes the force level F_{r2} to increase.

For displacement $y > y_2$, the origin is translated a distance $(y - y_2)$ and the penetration decreases until $y = y_3$, after which it remains constant.

If the pipe section continues to move in the same direction after break-out, there is a horizontal resistance (in addition to the friction) due to a mound of soil being pushed ahead of the pipe. The residual force, F_{r3}, will have effect on how far a pipe section will move after break-out whilst the pipe motion is still in the same direction.

Using same equation for F_{r2}, F_{r3} is expressed as an equivalent penetration after break-out, z_3, which is given as:

$$\frac{z_3}{z_2^*} = 0.82 - 3.2 \left(\frac{z_2^*}{d_h}\right) \qquad \frac{z_2^*}{d_h} \le 0.1 \quad (12a)$$

$$\frac{z_3}{z_2^*} = 0.5, \qquad \frac{z_2^*}{d_h} > 0.1 \quad (12b)$$

where z_2^* - maximum value of z_2 found in simulation up to the instant time considered

During the simulation, the model may go through many force-displacement cycles of different amplitude.

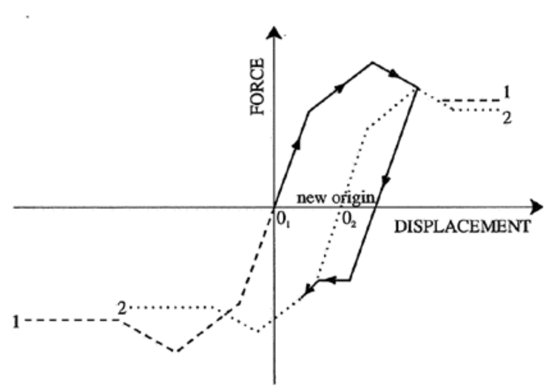


Figure 2 Force-displacement model. 1 cycle with amplitude

The displacement y_2, for which maximum break-out force F_{r2} occurs, is set to $0.5d_h$.

The value of y_3, i.e. where the resistance force become stable after break-out is taken as:

$$\frac{y_3}{d_h} = \frac{y_2}{d_h} + 0.1 + 3.3 \left(\frac{z_2^*}{d_h}\right), \qquad \frac{z_2^*}{d_h} \le 0.15 \quad (13a)$$

$$\frac{y_3}{d_h} = \frac{y_2}{d_h} + 0.6, \qquad \frac{z_2^*}{d_h} > 0.15 \quad (13b)$$

Clay model

Initial penetration z_i for the pipe due to weight, w_s, is given by:

$$\frac{z_i}{d_h} = 0.0025(S \cdot G^{0.3})^{4.2} + 0.067(S \cdot G^{0.3})^{0.75} \qquad (14)$$

where $G = \frac{S_u}{d_h \cdot \gamma_s}$

$$S = \frac{w_s}{d_h \cdot S_u}$$

S_u- remoulded undrained shear strength of soil

The maximum break-our force corresponding to a given pipe penetration z is taken as:

$$F_{r2} = 4.2 \cdot d_h \cdot S_u \cdot G^{-0.4} \left(\frac{z}{d_h}\right)^{1.25} \qquad (15)$$

The relation between pipe penetration and the work is given by:

$$\left(\frac{z_2}{d_h}\right) = 0.23\left(\xi \cdot S^2 \cdot \left(\frac{a}{d_h}\right)^{-1.5}\right)^{0.31}, \ \frac{a}{d_h} \geq 0.05 \qquad (16a)$$

$$\left(\frac{z_2}{d_h}\right)_{\max} = S \cdot G^{0.4}\left(\frac{a}{d_h}\right)^{0.2}, \ \frac{z_2}{d_h} \leq 0.3 \ , \ \frac{a}{d_h} \geq 0.05 \qquad (16b)$$

$$\left(\frac{z_2}{d_h}\right)_{\max} = 0.3 \ , \ \frac{z_2}{d_h} > 0.3 \qquad (16c)$$

If a/d_h is less than 0.05, then $a/d_h = 0.05$.

$\xi = \frac{E}{S_u \cdot d_h^3}$ $\qquad E(t) = \int_0^t F_r ds$

Where a - pipe oscillation amplitude (1/2 cycle)

The increasing penetration causes the force level F_{r2} to increase.

The point y_3 is determined through the expression:

$$\frac{y_3}{d_h} = 0.6\left(\frac{5.5}{\kappa} + 1\right) + \frac{y_2}{d_h} \qquad (17)$$

PARAMETER STUDY AND RESULTS

For the base case the pipe and soil properties used in analysis are presented in Table 1 and the material properties are listed in Table 2.

Table 1 Pipe and soil data

Pipe and soil data		
Friction coefficient, μ	0.6	-
Internal diameter of pipe, D_{in}	571.8	mm
Concrete coating, t_{con}	55	mm
Wall thickness, t_{wall}	19.1	mm
Corrosion allowance, t_{cal}	1.5	mm
Corrosion coating, t_{cc}	5	mm
Marine growth, t_{mg}	0	m
Pipe roughness, k/D	0.001	-

Table 2 Material properties

Steel density, ρ_{steel}	7800	kg/m^3
Concrete density, $\rho_{concrete}$	3000	kg/m^3
Oil density, ρ_{oil}	800	kg/m^3
Corrosion coating, ρ_{cor}	930	kg/m^3
Gravity coeff., g	9.81	m/s
Sea water density, ρ_{water}	1030	kg/m^3
Kinematic viscosity of salt water at 20°C, ν	1.05	mm^2/s

The environment data applied in the analysis is presented in Table 3.

Table 3 Environment data

Environment data	Return period		
	1 year	10 year	100 year
Wave height H (m)	10.3	12.6	14.8
Wave period T (s)	13.2	14.7	15.9
Current velocity (m/s) (1m above seabed)	0.36	0.51	0.66
Water depth (m)	104		

In the base case the length of pipe is 250 meters and the pipe is divided to 50 elements, and the boundary condition is defined as fellow:

• Fixed in translation and rotation at the left end.

• Fixed in rotation at the right end.

The response of pipe are taken from the right end (node 51), and the simulation time is set as 10800 seconds.

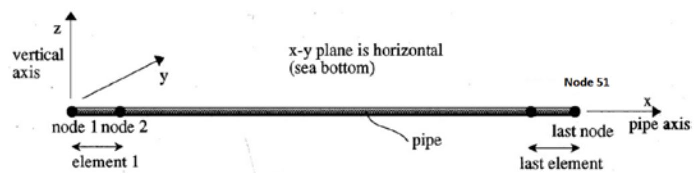


Figure 3 Illustration of PONDUS model

Soil friction coefficient

The stability of a pipeline is affected by the soil properties as well as hydrodynamic loading. Various kinds of failure or defect modes like pipeline walking and lateral buckling are considerably sensitive to the pipe-soil interaction. In this study, the effect of friction coefficients on stability of pipeline for both clay and sand soil are investigated to achieve a better understanding of pipe-soil interaction.

The combined loads of 10-year return period of current and 100-year return period of random waves are used in the simulation.

- ***Uniform soil type along route***

Based on the previous experimental studies on soil properties, the lower limit of the friction coefficient for clay is 0.2 while the upper limit is 0.6, therefore the variation of friction coefficient for clay soil is set to be 0.2, 0.4 and 0.6. The range of the friction coefficient for sand is between 0.4 and 0.8, and the friction coefficient is set as 0.4, 0.6 and 0.8 for the pipe-sand model.

- *Clay soil*

The envelope curve of displacement for different friction coefficient is shown in Figure 4 and the maximum displacement decreases as the friction coefficient increases. The envelope curve increases rapidly at the location near the left end, and it becomes relatively stable after the node 15.

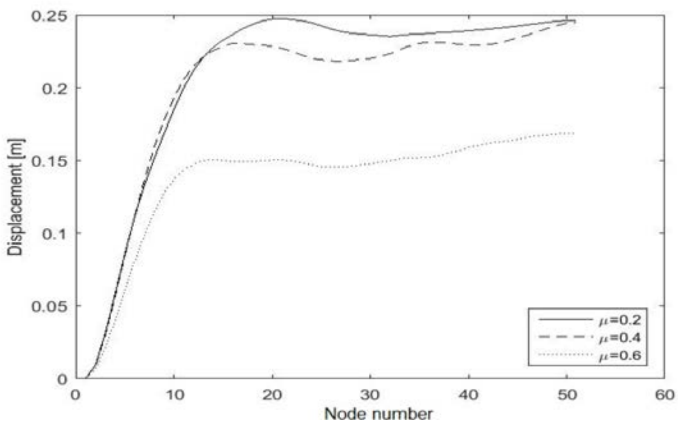
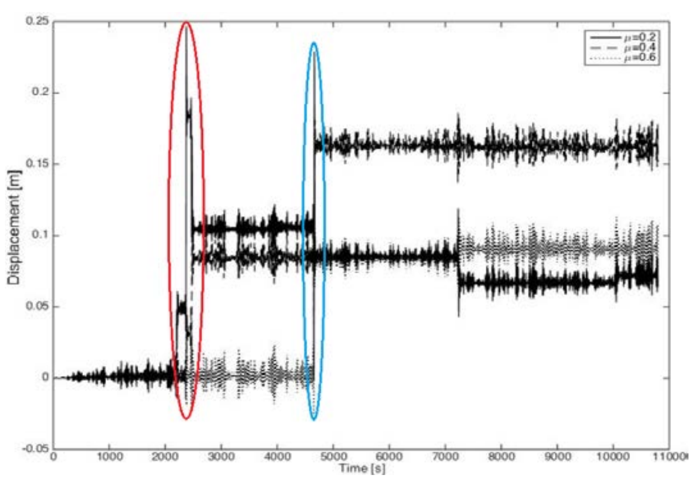


Figure 4 Envelope curve of displacement along the length of pipe (clay model with different friction coefficients)

Based on the development of the envelope curves, the time series of the displacement at the end node (node 51) is plotted in Figure 5. The oscillation of the displacement is caused by the oscillation in random waves. The displacement changes significantly at time intervals 2300-2500s and 4600-4700s and the zooming plot is also shown in Figure 5.



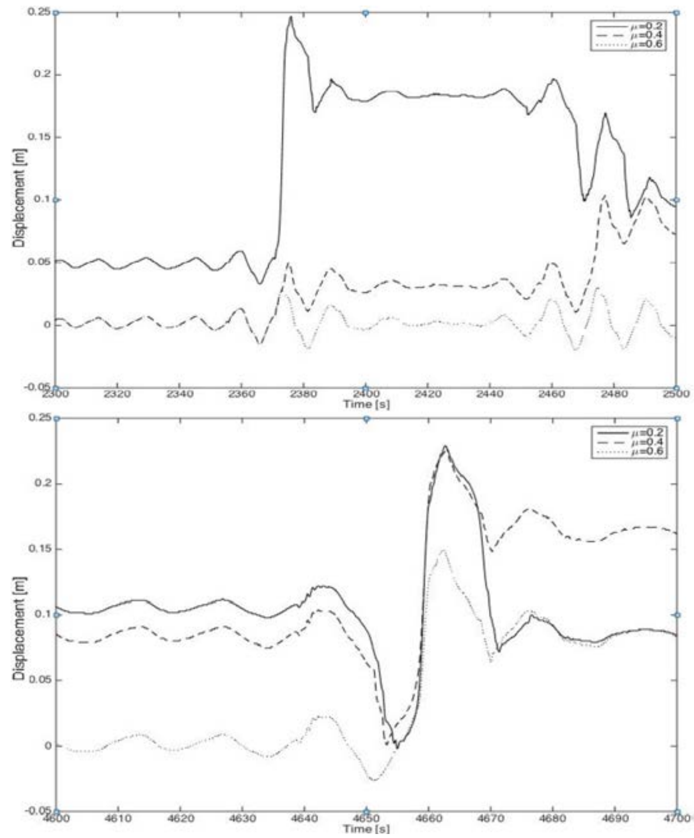


Figure 5 Time series of displacement at node 51 (clay model with different friction coefficients)

The development of relative penetration, which is non-dimensional parameter defined as penetration/d_h, at node 51 is shown in Figure 6. The initial relative penetrations are same for three different friction coefficients and pipe shows no movement. At the beginning, the soil force is only provided by the friction force, and it results that the smaller the friction coefficient is, the earlier the pipe starts to move. The development of relative penetration is general corresponding to the development of displacement.

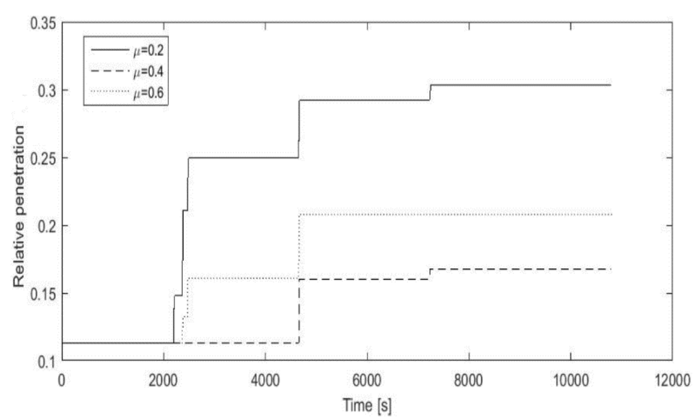


Figure 6 Time series of relative penetration at node 51 (clay model with different friction coefficients)

- *Sand soil*

The envelope curve of displacement for different friction coefficient is shown in Figure 7, and the time series of the displacement and relative penetration at the node 20, 32 and 30 where the maximum displacement occurs is plotted in Figure 8 and Figure 9 for friction coefficient of 0.4, 0.6 and 0.8 respectively.

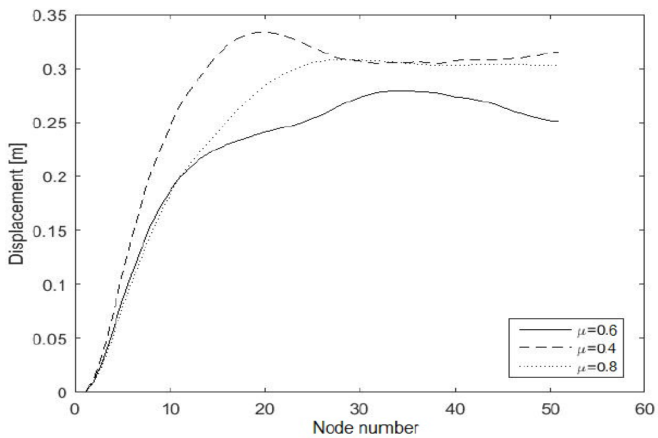
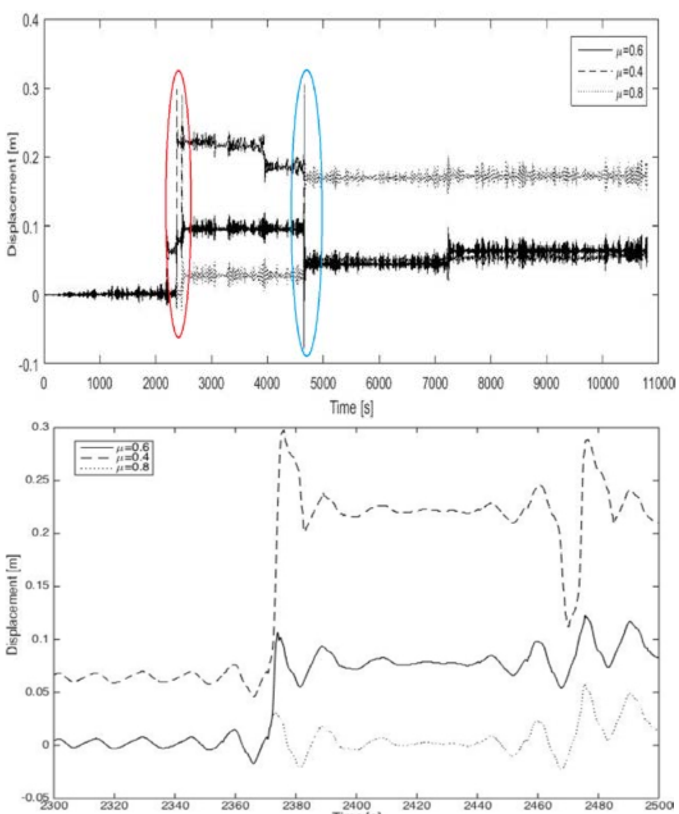


Figure 7 Envelope curve of displacement along the length of pipe (sand model with different friction coefficients)



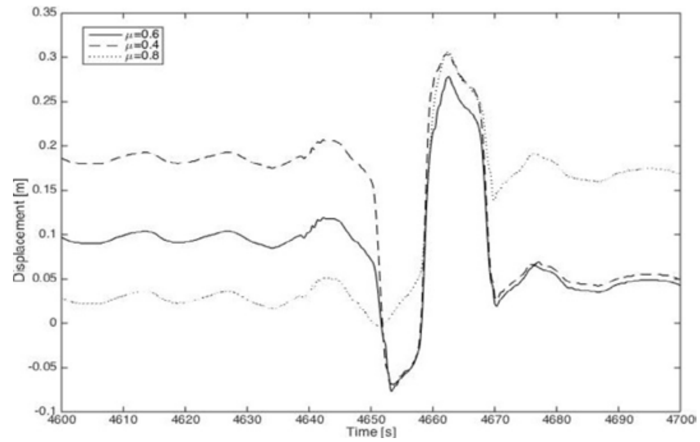


Figure 8 Time series of displacement at node 51 (sand model with different friction coefficients)

Summary of the results for uniform clay and sand soil with different friction coefficient is shown in Table 4. The results shows that the soil reaction is a non-linear behaviour, and the relative penetration contributes in a non-linear way to the stability of the pipeline.

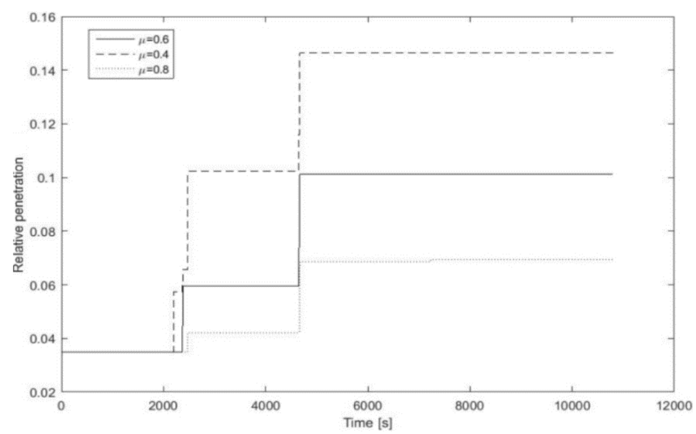


Figure 9 Time series of relative penetration (sand model with different friction coefficients)

Table 4 Summary of results for the uniform clay and sand soil with different friction coefficients

Case No,	soil	μ	Max. displacement (m)	Max. soil resistance force (N)	Max. relative penetration (m/m)
1	clay	0.2	0.25	1600	0.30
2		0.4	0.24	1650	0.16
3		0.6	0.17	1700	0.21
4	sand	0.4	0.34	2500	0.15
5		0.6	0.28	2300	0.11
6		0.8	0.31	2400	0.07

The similar features for both clay and sand soil are as following:

- The friction coefficient affects the initial soil resistance force, and further determine the initial time when the pipe starts to move in lateral direction. The envelope curve of the displacement along the pipeline is changed for different friction coefficients.
- The penetration is related to the accumulated displacement of pipeline which is reduced for large friction coefficient. The penetration and accumulated displacement tends to become larger as the friction coefficient decreases.

The main difference between clay and sand model is that sand soil is stiffer and less permeable than clay soil, and the penetration depth is subsequently larger for the pipe-clay model than for the pipe-sand model.

- ***Multiple soil types along route***

Considering the length and the complicity of the seabed environment in a pipeline route, a combined soil model with multiple soil types in different sequence along the route is simulated in this section. The same properties as the base case are applied in the simulation and the friction coefficient for sand and clay soil is 0.6 and 0.2 respectively.

Table 5 Cases for multiple soil types along route

Case	
s50c50	50% sand and 50% clay
c30s40c30	30%clay-40%sand-30%clay
s30c40s30	30%sand-40%clay-30%sand

The envelope curve of displacement for case s50c5 is shown in Figure 10. The maximum displacement at sand section and at clay section occurs at node 16 and 40. The relative penetration at node 16 and 40 is shown in Figure 11 and the maximum relative penetration in the sand section is smaller than the one in the clay section.

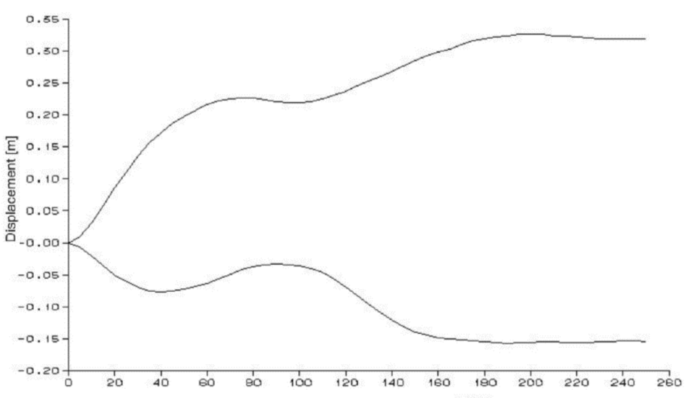


Figure 10 Envelope curve of displacement along the length of pipe (s50c50)

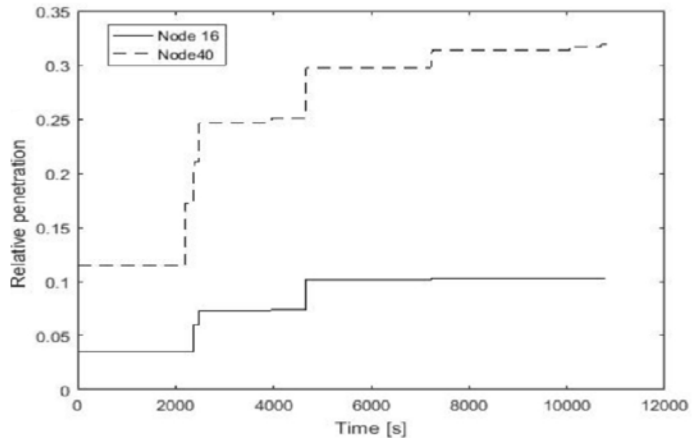


Figure 11 Time series of relative penetration at node 16 and 40 (s50c50)

The envelope curves of displacement for case c30s40c30 and s30c40s30 are shown in Figure 12. The maximum displacement occurs approximately at node 18 for both cases, but the magnitude of maximum displacement is quite different. The relative penetrations at node 18 for both cases are shown in Figure 13 and the maximum relative penetration in s30c40s30 is larger than the one in c30s40c30.

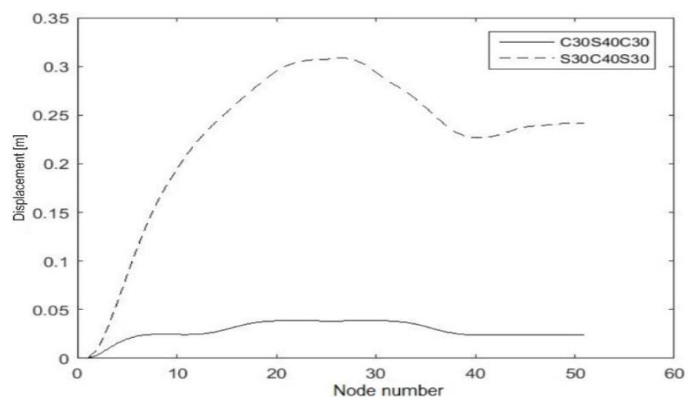


Figure 12 Envelope curve of displacement along the length of pipe (c30s40c30 and s30c40s30)

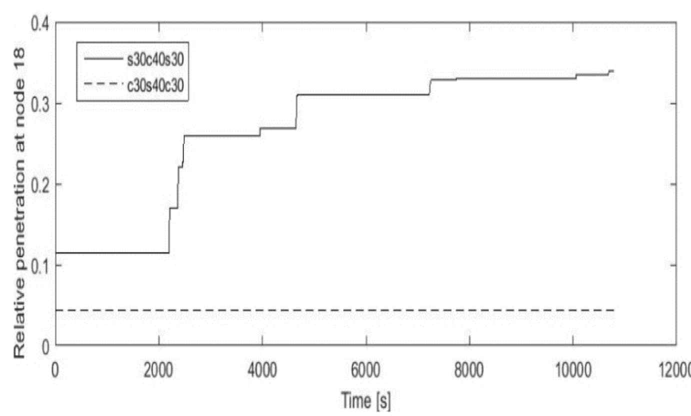


Figure 13 Time series of relative penetration at node 18 (c30s40c30 and s30c40s30)

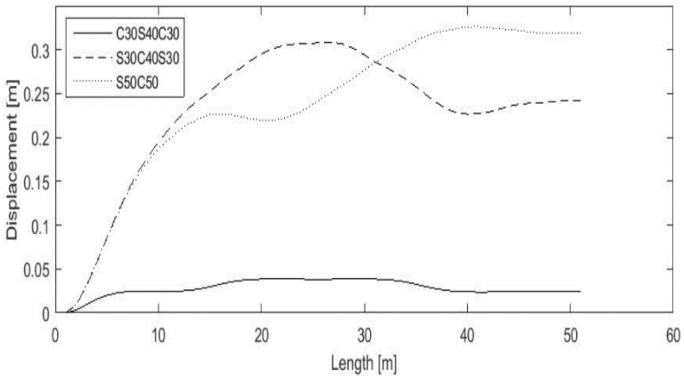


Figure 14 Envelope curve of displacement along the length of pipe (s50c50, c30s40c30 and s30c40s30)

The comparison of envelope curve for all three cases is shown in Figure 14. The results indicates that the movement of pipe significantly increases when there is sand section located at the end with fixed boundary condition. When sand soil is the dominant soil type, the magnitude of maximum displacement is quite similar for different soil combinations. However when the percentage of clay soil increases, the maximum displacement will be reduced considerably, not only for the whole pipeline but also for the local sections. Furthermore, nodal displacement, soil force and relative penetration appears to be quite stable and oscillates with a constant amplitude in C30S40C30. This is probably due to the relatively small movement at the end nodes as the clay soil provides high soil resistance force.

COMPARISON OF DIFFERENT PROCEDURES

In the design lifetime of pipeline it is likely to experience storm condition and thus it is more critical to maintain the on-bottom stability during storm. The penetration is an important parameter in the assessment of the on-bottom stability. Therefor it is important to find the stabilized penetration depth of the pipeline at the pre-storm situation. Three available procedure to assess the on-bottom stability under storm condition will be compared in this paper for clay soil. In each procedure, analyses were performed for seven realizations with random seeds.

1) Standard 3-hour simulation procedure with default initial penetration calculated by PONDUS

The envelope curve of displacement is shown in Figure 15. The mean maximum displacement plus one standard deviation for seven realizations is about 5.07m at the right end of the pipe, and is smaller than 10 times of the outer diameter of the pipe. It suggests that the pipe is stable when considering the $L_{stable} < L_{10}$ [2] displacement criteria. However it seems that the displacement will still develop and the conclusion may be conservative. It should be noted that the instability in this context refers to an "accumulated damage".that may also get contribution from storms that are less severe than the design storm in a normal analysis. Nevertheless, the standard procedure tends to underestimate the displacement.

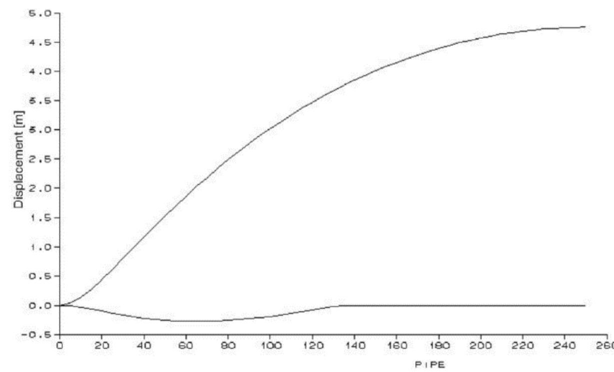


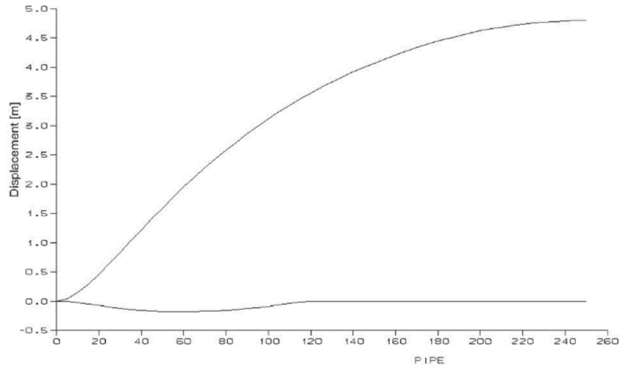
Figure 15 Envelope curve of displacement along the length of pipe (one realization, Standard 3-hour simulation procedure)

2) Procedure recommended by the PONDUS user manual

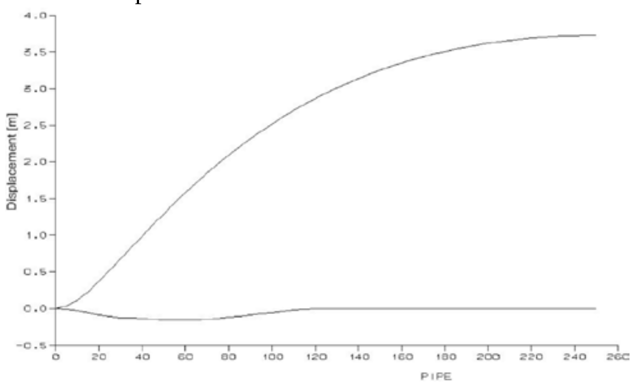
The procedure is penetration-oriented and is built up step by step by substituting the initial penetration depth taken from the previous simulation result. According to PONDUS manual [5], the waves are sorted into two sets, the first with 800 waves and the other with 500 waves.

- Step 1. 800 waves with linear increasing hydrodynamic forces.
- Step 2. 500 waves with short start-up period (100s), use the penetration found in Step 1 as initial penetration.
- Step 3. If the penetration is still under development, repeat the analysis with 500 waves and penetration from previous step as the initial penetration.
- Step 4. Perform the design storm analysis when the penetration is stabilized.

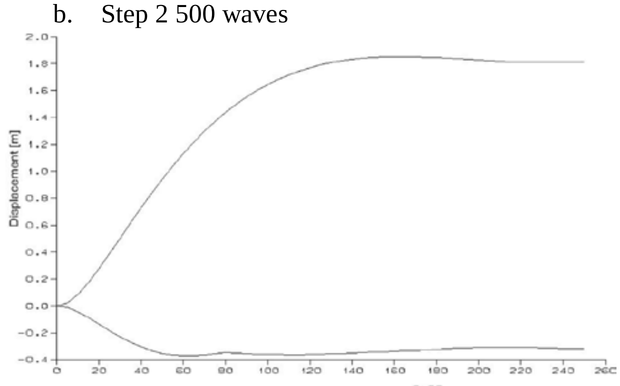
The envelope curve of displacement is shown in Figure 16. The displacement for PONDUS recommended procedure is almost zero for 3-hour storm condition, and the analysis results of step 4 indicates the pipe will be stable in the design storm when the stabilized penetration from Step 3 is considered as the initial penetration.



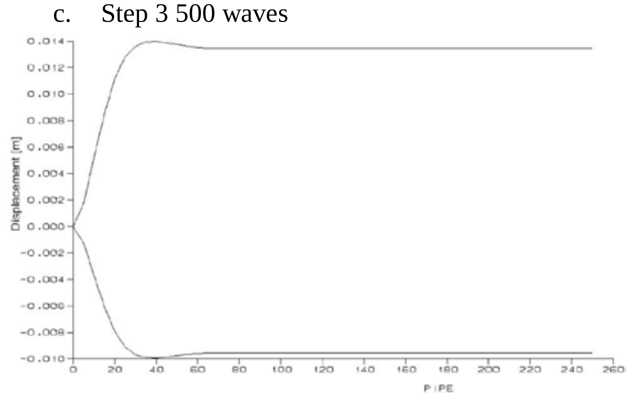
a. Step 1 800 waves



b. Step 2 500 waves



c. Step 3 500 waves



d. 3-hour storm

Figure 16 Envelope curve of displacement along the length of pipe (one realization, Procedure recommended by the PONDUS)

3) Procedure recommended by DNV-RP-F109

According to DNV [5], the storm sea scenario can also be modeled by introducing a linear ramp function on wave induced particle velocity and acceleration, so that the load increases from zero to full load during approximately the first 20 per cent of the analysis. The pipe will be subjected to moderate waves with small displacement that leads to increased penetration and passive resistance. As the 3-hour simulation is considered in the analysis the start-up period in PONDUS simulation is set to be 20 percent of 3-hour (2160s).

The envelope curve of displacement is shown in Figure 17. The mean maximum displacement plus one standard deviation is about 5.2m, and the results satisfy the L_{10} displacement criteria [5] for the pipeline, but it is not sufficient to draw a conclusion on whether the displacement of the pipeline will further increase if the simulation time increases. It is also important to find out whether the penetration becomes stable at 2160s in the analysis.

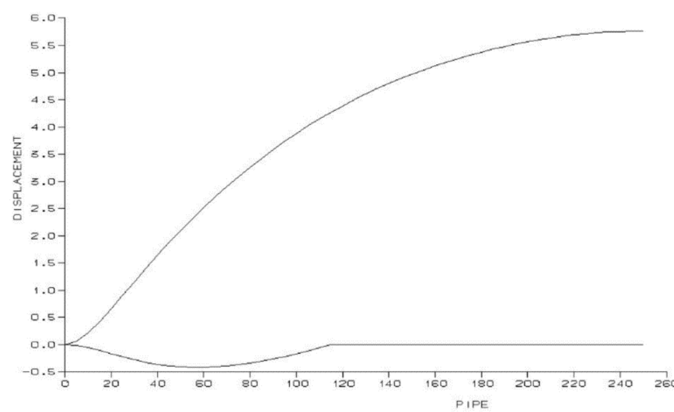


Figure 17 Envelope curve of displacement along the length of pipe (one realization, Procedure recommended by the DNV)

Considering the total steps of each procedure, the PONDUS recommended procedure is obviously the most sophisticated one, as the initial penetration is deducted step-by step with several iterations. The comparison of three procedures are summarized in Table 6, and the pipeline is stable during the storm condition for procedure 2. Although the displacements during 3-hour simulation with design storm fulfil the L_{10} displacement criteria for procedure 1 and 3 it is still difficult to make a conclusive assessment whether or not pipeline will be stable under storm circumstance. The procedure recommended by DNV considers the build-up effect of the initial penetration before the storm. It could be applied in the analysis when the penetration is stabilized after the start-up time (20% of 3-hour).

Table 6 Summary for three procedure

Procedure No.	Displacement during 3-hour simulation with design storm (m)	Max. relative penetration (m/m)
1	5.07	0.040
2	0.02	0.114
3	5.2	0.039

The effect of boundary condition is also investigated and same cases were study except the both ends of the pipe is set to free. The summary of the results is shown in Table 7.

Table 7 Summary for three procedure with free ends boundary condition

Procedure No.	Displacement during 3-hour simulation with design storm (m)	Max. relative penetration (m/m)
1	11.0	0.040
2	0.03	0.115
3	11.5	0.039

CONCLUSIONS

The main conclusions from the present study are listed as follows:

1. Compared to the simple Coulomb friction model, the applied pipe-soil models are proven to be more realistic. The passive resistance force due to the penetration contributes to the on-bottom stability of the pipeline.
2. When considering the energy-based soil model, the penetration is related to the accumulated displacement of pipeline. The smaller the friction coefficient is, the earlier the pipe will start to move. Moreover, the development of penetration and accumulated displacement tends to become larger as the friction coefficient decreases. The competition between friction force and passive resistance force results in that the development of displacement is changed for the different friction coefficients.
3. When considering the multiple soil types along the route, the maximum displacement is mostly dependent on the soil type at end nodes which is fixed, and when there is clay soil under the both ends of the pipeline for current numerical model, the maximum displacement will be reduced substantially.
4. Comparison of different procedures shows that the penetration depth during the pre-storm condition appears to be very important for the assessment of the on-bottom stability during the design storm condition for clay soil. Procedure recommended by the PONDUS is proven to be the most realistic and feasible procedure for the on-bottom stability assessment for a design storm situation.

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