Axial Soil Stiffness for Deepwater Pipeline Walking Analysis

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ABSTRACT

Deepwater pipeline global walking refers to the phenomenon that a pipeline, subjected to multiple start-up/shut-in thermal and pressure cycles, gradually moves toward one direction (usually the same to the flow direction). The fundamental mechanism of walking is rather complex and is affected not only by the thermal/pressure loading and pipeline structural behavior (e.g., lateral buckling), but also by the pipeline-soil interaction (PSI) which varies both spatially and temporally. The complex nature of walking makes the prediction of pipeline walking a challenging task, and the prevention of pipeline walking, which may be unnecessary, requires significant financial budget and increased risk exposure from offshore installations. Therefore, the motivation of this paper is to offer insights into PSI to help the engineering design of pipeline walking. The objectives of the paper are to re-visit the axial soil stiffness model (characterized by the axial mobilization displacement) recommended in the SAFEBUCK guideline and to propose a new model for the axial mobilization displacement for walking analysis. The recommended mobilization displacement is based on a simple energy method and is calibrated with a PSI analytical model, SAFEBUCK JIP model tests, in-house proprietary tests, and test results available in the literature.

Keywords: pipe-soil interaction, walking, axial stiffness, mobilization displacement

INTRODUCTION

Deepwater pipelines (in this paper, in-field flowlines and export pipelines are both termed pipelines for simplicity) are generally subjected to multiple high pressure and high temperature cycles due to start-up/shut-in operations during the design life, and thus build up a tendency to move globally toward one direction (usually the same to the flow direction), a phenomenon so-called pipeline walking. The fundamental mechanism of walking is rather complex and is affected not only by the thermal/pressure loading and pipeline structural behavior (e.g., lateral buckling), but also by the pipeline-soil interaction (PSI) which varies both spatially and temporally. Carr *et al.* (2006) provided analytical solutions to address the effects of the steel catenary riser (SCR) tension, the global seabed slope, and the thermal transient on the pipeline walking, and numerous studies have been conducted on the axial friction along the pipeline during the past two decades (Bruton *et al.*, 2008; Guha *et al.*, 2019; Hill *et al.*, 2012; Hussien *et al.*, 2020; Najjar *et al.*, 2007; Randolph *et al.*, 2012; Westgate *et al.*, 2018; White *et al.*, 2011; White *et al.*, 2010; White *et al.*, 2015).

Current state-of-the-practice for pipeline walking is to treat the soil axial resistance as an equivalent friction coefficient for input in the pipeline finite element (FE) analysis. The soil friction coefficient is usually simplistically assumed to be uniform along the whole pipeline and be velocity independent; however, design usually indeed considers both the undrained and drained behavior of the soil to reflect or partially reflect the pipeline movement history, and covers the low estimate to high estimate of the soil friction to reflect the soil spatial variability along the pipeline. Therefore, the previous theories and experimental programs mainly focused on how to accurately characterize the equivalent soil friction coefficient.

Another input to the pipeline structural analysis but received relatively less attention is the PSI axial stiffness (the ratio of the axial load transferred to the unit length of a pipeline to the axial displacement of that pipeline section). Current practice for this axial stiffness is usually to adopt a simple bi-linear model in SAFEBUCK and DNV-RP-F110 (2015), such that the soil axial resistance increases linearly with axial displacement to a point termed axial mobilization displacement (the minimum of the 1% of the pipe outer diameter and 5 mm), after which the maximum friction is mobilized and the axial resistance reaches the plateau. The mobilization displacement is crucial in the calculated pipeline walking rate, and hence impacts the pipeline walking mitigation strategy. In general, the calculated walking rate decreases with the mobilization displacement based on the literature (Hill *et al.*, 2012) and in-house experience. In essence, the PSI axial stiffness should be simpler than the axial t-z models for conventional piles under axial loading, because the submerged pipe weight, and hence the total normal stress applied to the soil around the pipe-soil interface is known, while in the case of piles, the horizontal stress remains unknown (Randolph *et al.*, 2012). Therefore, Randolph *et al.* (2012) anticipated the potential for a more complete theoretical treatment for this axial stiffness model, and used a simple hyperbolic form in determining the axial friction evolution. Subsequent analytical solutions for an elastic response were provided by Randolph (2013) and Guha *et al.* (2016).

In order to better determine the PSI mobilization displacement, in the first part of this paper, an analytical model is proposed by extending the linear elastic solutions from Guha *et al.* (2016) to consider the nonlinearity of the stress-strain relation in the soil element, then a simplified axial load – displacement relation is proposed by directly linking to the soil element stress – strain curves with the same concept of mobilizable strength design (MSD) proposed in Osman *et al.* (2007). In the second part of this paper, the above established nonlinear analytical model is combined with selected SAFEBUCK JIP model test data, in-house proprietary tests, and test results available in the literature to determine the mobilization displacement using an energy-based approach. The findings from this paper are expected to provide rational guidance on the selection of the soil mobilization displacement for pipeline walking analyses.

PIPELINE AXIAL LOAD-DISPLACEMENT MODEL

Using the same model as Randolph (2013) and Guha *et al.* (2016), the pipeline load-displacement response is investigated under the plane-strain condition. Figure 1 shows the model schematic for the analytical solution. The solution derivation is based on a half embedded pipe with an outer diameter of "D'" for convenience. For a pipe with an embedment less than half of its diameter D as shown in right of Figure 1, simplifications can be made by treating the contact width $Dsin\theta_D$ (where θ_D is the subtended angle as defined in Figure 1) as D' and then virtually move the pipe downward to have a hypothetical half embedment. For a pipe with more than a half D embedment, the derivation is more complicated and is not addressed in this study; however, the normalized axial-displacement response of a pipeline with more than half D embedment is similar to the one with less than half D embedment from practical experience.

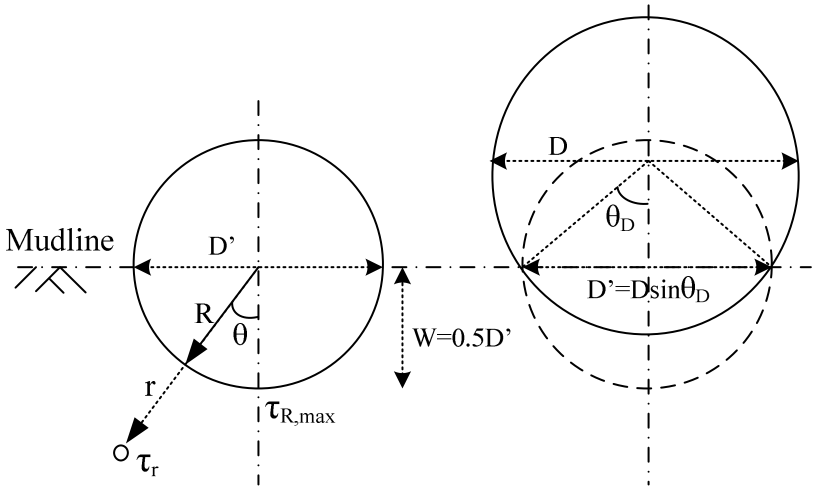

Figure 1 Schematic of a partially embedded pipeline

ANALYTICAL DERIVATION OF AXIAL LOAD-DISPLACEMENT RESPONSE

Focus of this paper is limited to a pipeline embedded in clays. In Figure 1, consider the general point that locates a distance (radius) r and an angle θ from vertical, the shear stress (in the direction perpendicular to the plane) τ_r can be reasonably assumed to be $\tau_r = \tau_{r,max}cos\theta$, where $\tau_{r,max}$ is the maximum shear stress below the pipe invert at a distance r below the pipe center. Therefore, from the equilibrium requirement (Randolph, 1977), the shear stress in any points in the soil can be linked to the maximum shear stress at the pipe invert, $\tau_{R,max}$, as below:

$$\tau_r = \tau_{R,max}\frac{R}{r}cos\theta \qquad 1$$

where $R = 0.5D'$. The shear stress-shear strain relationship adopted is nonlinear based on a total stress analysis method, and is assumed to follow a power law suggested by Vardanega and Bolton (2011) as shown below:

$$\frac{\tau_r}{S_u} = a\left(\frac{\gamma}{\gamma_n}\right)^b \leq 1.0 \qquad 2$$

where S_u is the undrained shear strength; γ is the shear strain; γ_n is a reference shear strain; a and b are the non-dimensional fitting parameters. For typical deepwater soft clays, due to the disturbance during the pipeline installation and the subsequent consolidation, S_u can be reasonably assumed to increase linearly with depth with zero strength at the mudline, i.e., $S_u = kz$, where k is the strength gradient and z is the depth below the mudline.

Combining Equation 1 and 2 with $S_u = kz$ gives

$$\gamma = \gamma_n\left(\frac{\tau_{R,max}R}{ak}\right)^{1/b} r^{-2/b} \qquad 3$$

The shear strain can be linked to the soil deformation by $\frac{du}{dr} = \gamma$, where u is the radial gradient of the axial displacement. Thus, the axial displacement δ_x can be obtained as

$$\delta_x = \int_R^\infty \frac{du}{dr} dr \qquad 4$$

Combining Equation 3 and 4 gives

$$\delta_x = \gamma_n \left(\frac{\tau_{R,max} R}{ak}\right)^{1/b} \frac{\mathrm{b}}{2-b} R^{1-2/b} \qquad 5$$

The axial force, F, exerted on the pipeline can be obtained from the integration of the shear stress at the pipe-soil interface as follows:

$$F = 2\int_0^{\pi/2} \tau_{R,max} cos\theta \, dR = 2R\tau_{R,max} \qquad 6$$

Limiting $\tau_{R,max}$ to the shear strength at the interface, combining Equation 5 and 6, and revoking $R = 0.5D' = 0.5Dsin\theta_D$ gives

$$\frac{F}{F_u} = \left(\frac{4-2\mathrm{b}}{bsin\theta_D}\right)^b \cdot a \left(\frac{\delta_x/D}{\gamma_n}\right)^b \qquad 7$$

where F_u is the maximum axial force that can be exerted on the pipe.

For soils with a constant profile of S_u, similar derivation as shown in the above can be repeated gives

$$\frac{F}{F_u} = \left(\frac{2-2\mathrm{b}}{bsin\theta_D}\right)^b \cdot a \left(\frac{\delta_x/D}{\gamma_n}\right)^b \qquad 8$$

For practical applications, Equation 7 and Equation 8 do not differ significantly considering the uncertainties in the soil strength. Hence, it is suggested the normalized axial load-displacement response is taken the "average" of Equation 7 and 8 as:

$$\frac{F}{F_u} = \left(\frac{3-2\mathrm{b}}{bsin\theta_D}\right)^b \cdot a \left(\frac{\delta_x/D}{\gamma_n}\right)^b \qquad 9$$

Equation 9 resembles Equation 2 in a way that the pipeline axial load-displacement macro behavior takes the same form as the stress-strain relation in the fundamental element level. This result is consistent with the mobilizable strength design (MSD) concept in Osman *et al.* (2007) that the structural load-displacement response may be determined directly from the fundamental stress-strain relationship.

DEEPWATER APPLICATION

Equation 9 is then used to predict the axial load-displacement responses of pipelines in deepwater soft clays. Typical normalized stress-strain relation for deepwater soft clays from direct simple shear (DSS) tests can be found in Chen *et al.* (2019) and is re-produced in Figure 2. The best fit to the data in Figure 2 gives a=0.5, b=0.3, and $\gamma_n = 0.8\%$. The calculated normalized axial load F/F_u – normalized axial displacement δ_x/D response is shown in Figure 3 (for a half diameter embedment). Also shown in Figure 3 are the calculated responses using a linear elastic approach (Guha *et al.*, 2016), the laboratory 1-g model tests conducted at The University of Texas (UT) at Austin (Hussien *et al.*, 2020), the in-situ measurements of the "SMARTPIPE" (White *et al.*, 2010), and the bi-linear model in SAFEBUCK III (SAFEBUCK and DNV-RP-F110, 2015).

As shown in Figure 3, the nonlinear model proposed in this study (Equation 9) predicts the mobilization displacement is about 1% of the diameter, which coincidently exactly agrees with the best estimate of the bi-linear model recommended by SAFEBUCK III. The linear model using a constant shear modulus G with a ratio of $G/S_u = 44$[1] gives a larger mobilization displacement than the nonlinear model, around 3.5% of the diameter. The mobilization displacements from the selected UT tests vary from about 4% to 6% of the diameter (108 mm), which are closer to the prediction from the linear elastic model, but are significantly greater than the predicted value from the nonlinear model. The in-situ measurements of the mobilization displacement from the "SMARTPIPE" differ substantially during the first sweep and the subsequent sweeps. The first sweep gave a mobilization displacement of about 5% of the diameter (225 mm), while the 6th sweep which was conducted with a higher moving velocity gave a much larger mobilization displacement of about 27% of the diameter.

The discrepancies from the model predictions (both linear and nonlinear), the recommended practice in SAFEBUCK III, and the model tests (both 1-g laboratory tests and in-situ tests) signify the complexity of the accurate prediction of the mobilization displacement. It appears now that the mobilization displacement increases with the pipe moving velocity, and hence the mobilization displacement tends to be smaller for a drained response than that for an undrained response, as can be seen from the UT model tests and the "SMARTPIPE" tests. However, the current observation is by no means the final conclusion, and alternative theory may be developed to more accurately determine the mobilization displacement. One such idea is to link the mobilization displacement directly to the time of mobilizing the pipe instead of relating to the distance of the pipe has travelled (White *et al.*, 2010). Another practical way for design, without further advanced theories, is to compile the available model test data and theoretical model predictions in a statistical way to best capture the current understanding. One such initiative is proposed in this paper below.

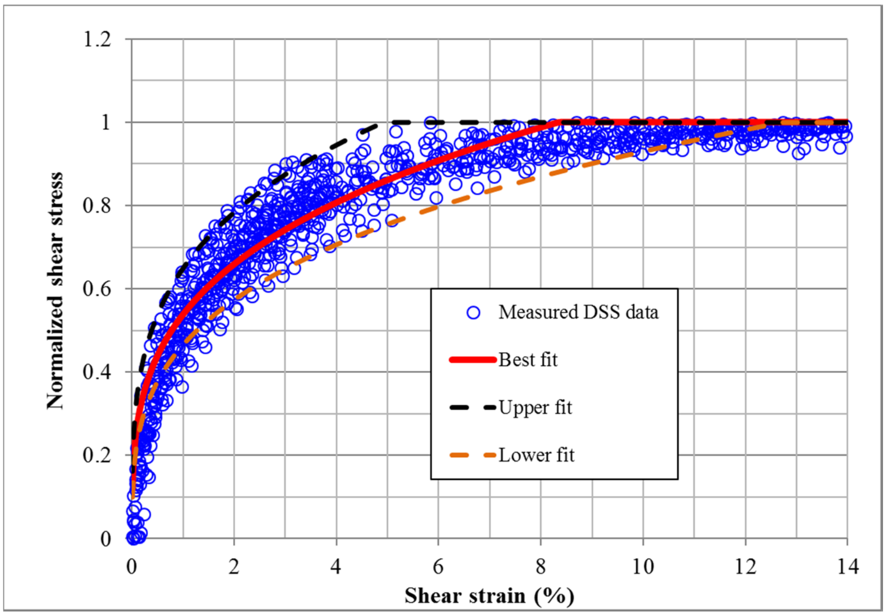

Figure 2 Typical stress-strain relation for deepwater clays

[1] The value of $G/S_u = 44$ was selected is based on the best estimate of the stress-strain relation in Figure 2 by equating the specific energy (the area under the stress-strain curve) of the linear elastic response to that of the nonlinear response.

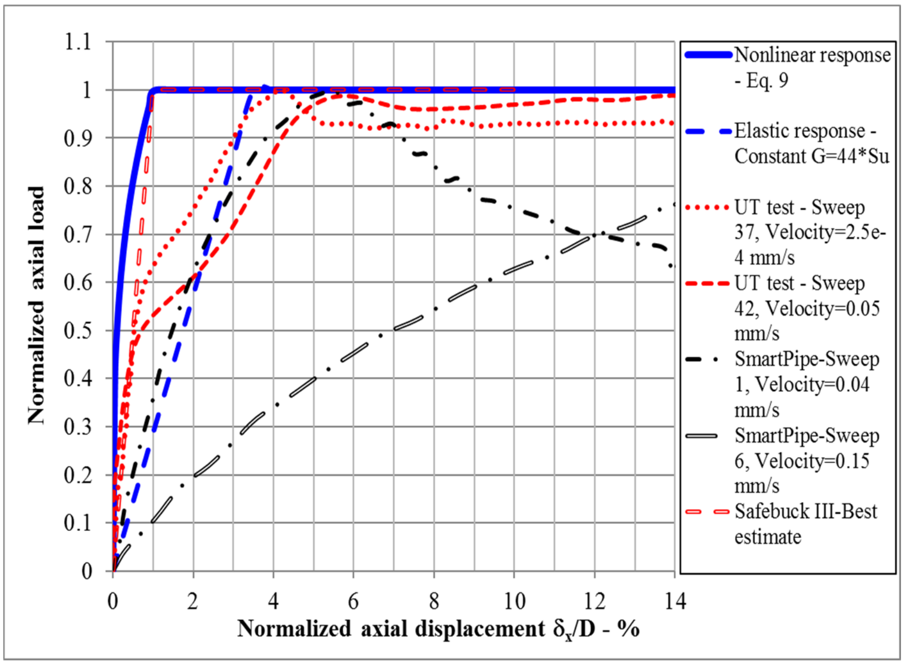

Figure 3 Normalized load-displacement response: predictions and model test results

ENERGY BASED DETERMINATION OF MOBILIZATION DISPLACEMENT

The axial load-displacement response for PSI is in general nonlinear as can be seen from Figure 3. In order to be consistent with pipeline structural analyses that generally require a linear elastic perfectly plastic model for the soil friction, the bi-linear model in SAFEBUCK III simplifies the nonlinear responses largely by focusing on the initial tangential part of the measured responses for the low and best estimate of the mobilization displacement, while the recommended high estimate was intended to capture the displacement mobilizing the residual displacement observed in the model tests. The best estimate of the mobilization displacement in SAFEBUCK III is the minimum of 5 mm and 1% of the diameter.

It is judged that the mobilization displacement determined from the initial tangential part of the load-displacement curve for an equivalent bi-linear model tends to be small. A better way to determine this mobilization displacement for an equivalent bi-linear model is from an energy-based approach as described below. From the pipeline walking point of view, a system with a bi-linear soil model can be approximately treated to be equivalent to the one with a nonlinear soil model if the energy absorbed by the soil with a bi-linear model is the same to that absorbed by the soil with a nonlinear response. This reasoning is illustrated in detail in Figure 4. Suppose the actual load-displacement response of a pipeline is nonlinear as shown by the solid curve, then the equivalent bi-linear model is depicted by the dash curve such that the area below the nonlinear curve is the same to the area below the bi-linear model to ensure the energy absorbed by the soil from the two models are the same. In this way, the walking rates of the two system are expected to be similar as the energy absorbed by the soil is identical.

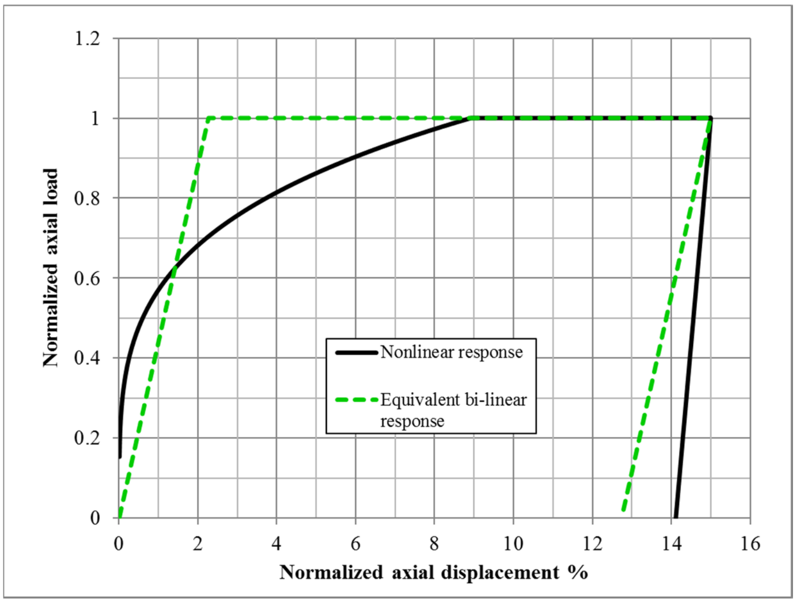

Figure 4 Equivalent bi-linear model for nonlinear response using an energy method

The above energy-based methodology is adopted to re-interpret "SMARTPIPE" test results, the nonlinear analytical model prediction, the in-house model test results, and the selected SAFEBUCK JIP model test data to determine the mobilization displacement for the equivalent bi-linear soil model. For the convenience of the mathematical manipulation, Equation 9 is used to fit the nonlinear axial load-displacement response. Because the soil exhibits a higher stiffness immediately after a loading reversal, the unloading part of the nonlinear model is simplistically treated to be linear with an elastic stiffness 10 times greater than the elastic stiffness of the bi-linear model. Then the mobilization displacement for the equivalent bi-linear model is determined at the displacement fully mobilizing the soil resistance at the first time. In total, 53 cases were analyzed with the determined mobilization displacements range from 1.3% of the diameter to 21% of the diameter, and the average value is about 8% of the diameter. The cumulative distribution function (CDF) of the mobilization displacement is shown in Figure 5, with a low estimate (5^{th} percentile value), a best estimate (50^{th} percentile value), a high estimate (95^{th} percentile value) of 1.4%, 6.2%, 20% of the diameter, respectively.

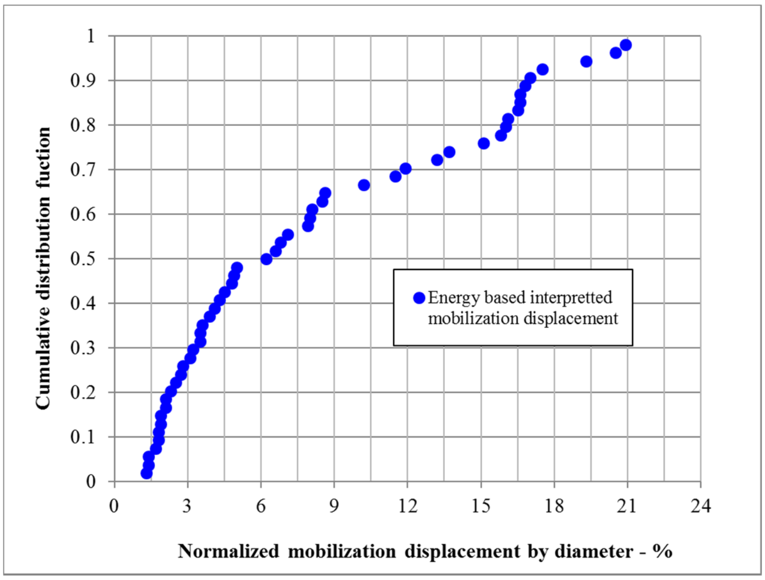

Figure 5 Cumulative distribution function for the mobilization displacement

DISCUSSIONS AND RECOMMENDATIONS

The determination of the mobilization displacement to be used in an equivalent bi-linear soil friction model for pipeline walking analyses is complex due to the nonlinear nature of the soil behavior, the drainage condition of the soil, the pipeline moving velocity, and the significant change of the pipe-soil interface when forming a shear band. In addition, the mobilization displacement may change over the time scale during the cyclic and consolidation history. The analytical solution developed in this paper for the nonlinear load-displacement response is elegant in theory, but appears to not agree well with the model test results. The linear elastic solution with an appropriate shear modulus (which is determined based on the same specific energy for the nonlinear and linear response) seems to bring the prediction closer to the model test observations, but still away from satisfaction. In addition, it has been generally agreed that first cycle response is different from the subsequently cycles in that a significant peak value can be observed in the first cycle, while the peak value in the subsequently cycles is not very noticeable. However, the consolidation time between the cycles adds additional complexities in this conclusion. Alternative theories may be used to illustrate the mechanism of the mobilization displacement, e.g., the mobilization of soil resistance is dependent on the time instead of a specific distance (White *et al.*, 2010) based on several in-situ model tests; however, no consistent trend is observed from other model tests, and the theory needs to be more refined.

Another pitfall in determining the mobilization displacement is the failure to differentiate the elastic versus plastic displacements mobilizing the soil resistance. Because the soil response is highly nonlinear, by the time the soil mobilizes the ultimate resistance, there are substantial plastic deformations occurred that upon unloading, that plastic displacement will remain. However, in typical pipeline analyses, the soil frictional model is linear elastic perfectly plastic both for loading and unloading, then the plastic part of the mobilization displacement cannot be simply treated as the elastic mobilization displacement to be used in the pipeline analysis; otherwise, the walking rate would be underpredicted. This phenomenon is well captured in the current SAFEBUCK guideline: "…*the structural analysis should model the elastic mobilisation distance which is the stiff and recoverable part of the axial response, rather than the plastic irrecoverable part, to avoid underestimation of walking*" (SAFEBUCK and DNV-RP-F110, 2015). The proposed energy-based methodology in this paper overcomes this pitfall of elastic versus plastic displacements, because the method ensures the energy absorbed by the nonlinear model and the bi-linear model is the same.

Based on the above discussions, below is the list of recommendations for the current practice:

1. It is recommended to use the energy-based method proposed here to determine the mobilization displacement from nonlinear axial load-displacement responses.
2. In the absence of project-specific assessments/tests, it is recommended to use a mobilization displacement of 6.2% of the pipe outer diameter in the linear elastic perfectly plastic soil axial friction model for the best estimate, and it is suggested to update the SAFEBUCK guideline accordingly. Sensitivities may be performed on the low estimate and high estimate of the mobilization displacement of about 1.3% and 20% of the pipe outer diameter, respectively.
3. It is recommended to further compile the available test data to further refine the statistics on the recommended mobilization displacement in this paper.
4. It is recommended to further develop theories to reconcile the different observations and sometimes even contradicting evidence from different model tests in the determination of the mobilization displacement, e.g., the effect of drainage condition, consolidation, and the cyclic evolution.

CONCLUSIONS

This paper presents a nonlinear analytical solution for the load-displacement response of a pipeline under axial loading, which is a significant generalization from the current available elastic solution. The analytical solution is then applied to predict the axial load-displacement responses measured in the model tests. Discrepancies between the predictions and measurements are presented and discussed. The paper then proposes an energy-based method to determine the mobilization displacement to be used in the linear elastic perfectly plastic soil friction model in pipeline structural analyses. The analytical solutions, the in-house model test results, and the selected SAFEBUCK JIP model test data were then compiled to statistically determine the mobilization displacement. The paper concludes the complexities in the accurate determination of the mobilization displacement, lists the common pitfalls in interpreting model test results, and recommends a mobilization displacement of 6.2% of the pipe outer diameter as a best estimate in the absence of project-specific assessments. The paper also offers areas to be further researched in the proper determination of the mobilization displacement.

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