A general model for the behaviour of offshore pipelines on sand

Un modèle généralisé du comportement des oléoducs sur sable

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ABSTRACT: Experimental observations from a number of centrifuge tests using calcareous sand and silica sand are presented. The tests were conducted mainly on untrenched pipelines, with control of vertical and lateral loads and measurement of the corresponding displacements. In parallel, a theoretical model has been developed on the basis of plasticity theory to simulate the response of a pipeline shallowly embedded in sandy soil under self-weight or by trenching. This model requires a total of twelve parameters to simulate pipeline response under various loading conditions. The model reproduces the key features of the measured load-displacement response of trenched and untrenched model pipelines in calcareous sand and silica sand under various loading conditions.

RÉSUMÉ: Des observations expérimentales à partir d'essais en centrifugeuse sur du sable calcaire et du sable silicieux sont présentées. Les essais ont été principalement menés sur des oléoducs non enfoncés, avec contrôle des forces verticale et transversale et mesure des déplacements correspondant. En parallèle, un modèle théorique a été dévelopé basé sur la théorie de la plasticité pour simuler la réponse d'un oléoduc peu profondément enfoncé dans du sable sous son propre poids ou par fossoyage. Ce modèle nécessite au total douze paramètres pour simuler la réponse d'oléoducs sous différents états de chargement. Le modèle reproduit les caratéristiques clés de la réponse chargement-déplacement des modèles d'oléoducs enfoncés et non-enfoncés dans des sables calcaire et silicieux sous différents conditions de chargement.

1 INTRODUCTION

The design of offshore pipelines in terms of the required submerged weight and any necessary anchoring, trenching or armouring to resist hydrodynamic loads is to a large extent driven by geotechnical considerations. Under this background, a series of experimental projects were carried out in the 80's and led to the development of an empirical pipe-soil interaction model (Lieng et al. 1988, Brennodden et al. 1989) for silica sand and soft clay. In this model, the lateral soil resistance was given by the following equation:

$$F = F_F + F_R \tag{1}$$

where F is the total resistance, F_F the sliding resistance and F_R the penetration dependent resistance.

This model has been incorporated in the current Pipeline On-Bottom Stability Design Guidelines (AGA/PRC 1993, PRCI 1998). However, due to the empirical nature of the model, the actual physics of pipe-soil interaction is not well understood (Hale et al. 1991). Non-physical behaviour may be exhibited when the model is used in certain realistic parameter ranges (Verley & Sotberg 1994), and difficulties are experienced when attempting to use the model for soil conditions that are different from those used in the original experiments (Zhang et al. 1999).

In an attempt at better understanding pipe-soil interaction, a number of model tests have been performed using calcareous sand and silica sand on the beam centrifuge at The University of Western Australia (Wallace 1995, Browne-Cooper, 1997, Zhang et al. 1999 & 2000a). A theoretical model has been developed on the basis of plasticity theory to simulate the response of a pipeline shallowly embedded in sandy soil. The model enables simulation of pipeline response under any loading path. It reproduces the key features of the measured load-displacement response of trenched and untrenched model pipelines in calcareous sand and silica sand under various loading conditions.

2 EXPERIMENTAL OBSERVATIONS

The general arrangement of the test apparatus is shown schematically in Figure 1. Tests were performed at 50 g on a centrifuge, using seabed calcareous sand and commercial silica sand. The particle size distribution of the materials is shown in Figure 2. Initial conditions of soil samples are summarised in Table 1. Detailed descriptions of the test apparatus and procedures are given by Zhang et al. (2000a) with the centrifuge facility described by Randolph et al. (1991).

Table 1. Soil conditions.

Soil type	γ_d (kN/m^3)	D_r (%)	CPT gradient (MPa/m)
Medium dense calcareous sand	13.8	-	0.4
Very dense calcareous sand	15.2	-	1.2
Medium dense quartz sand	16.6	56	0.4
Very dense quartz sand	17.6	96	1.1

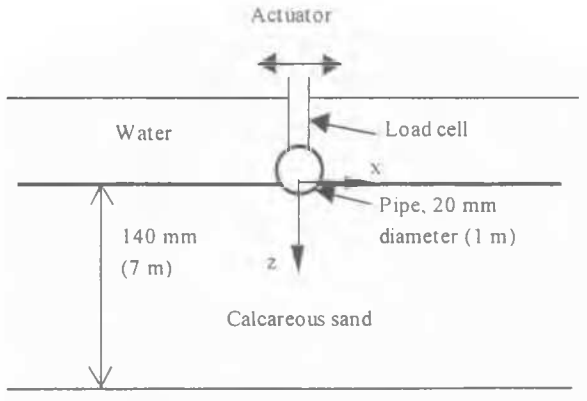

Figure 1. The general arrangement for pipe tests.

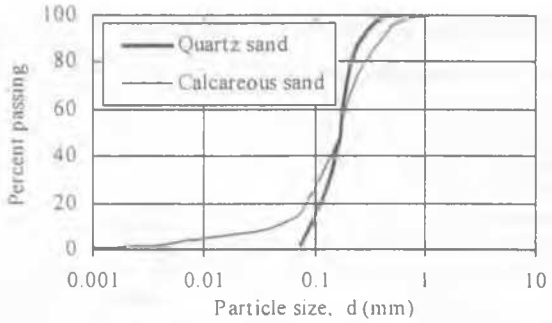

Figure 2. Particle size distribution for the soil samples.

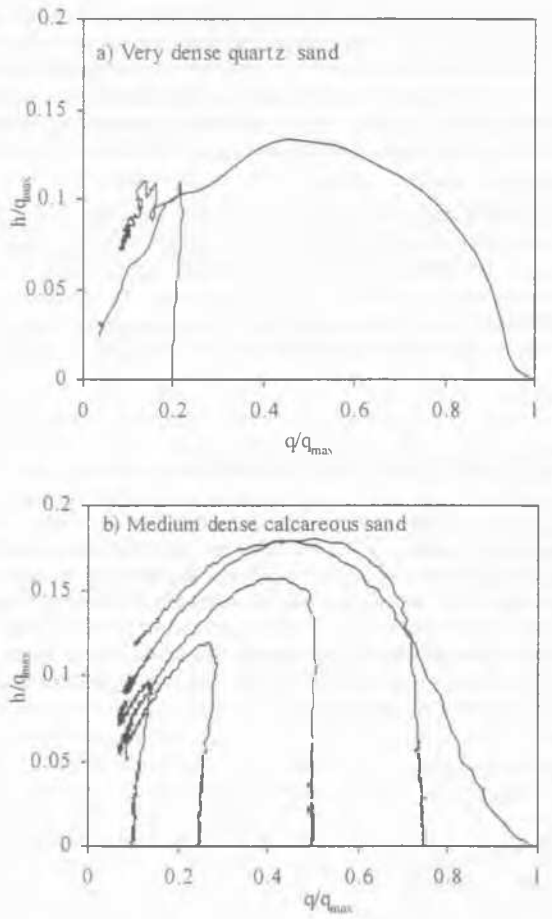

Figure 3. Typical results from sideswipe tests for pipes of different loading history.

Experimental data showed that the nominal vertical stress q (= vertical load per unit length/pipe diameter) required for penetrating the pipe increased approximately linearly with depth for all soil conditions. Test results suggested average vertical bearing moduli of 0.4 MPa/m, 0.35 MPa/m, 1.1 MPa/m and 1.2 MPa/m for the medium dense quartz sand, medium dense calcareous sand, very dense quartz sand and very dense calcareous sand, respectively. These are very close to the CPT gradients presented in Table 1.

Typical sideswipe test results (where the pipe was penetrated to a certain depth and then the vertical position of the pipe was held constant while displaced laterally) are shown in Figure 3. The data are plotted in normalised form with the nominal vertical stress, q, and nominal horizontal stress, h (= horizontal load per unit length/diameter), divided by the maximum vertical stress q_{max} prior to lateral movement. It appears from Figure 3 that the loading path is a parabola and has a positive intersect with the nominal horizontal stress axis, indicating some passive resistance of a partially embedded pipeline at zero vertical load. In some tests, after initial penetration of the pipe, the vertical load was reduced before pushing laterally. These tests have been termed "over-loaded" with an over-loading ratio $R = q_{max}/q$ defined. Results from these over-loaded sideswipe tests indicate that the vertical stress stayed approximately constant during the initial stages of the tests, prior to yield. The value of the peak horizontal stress was not affected by the applied vertical stress where $R < 2$. The horizontal stiffness of pipe response was not affected much by the over loading ratio.

Typical probe test results (where the vertical load was kept constant as the pipe was pushed laterally) for normally loaded pipes showed that all loading curves levelled out at nominal horizontal stresses close to the applied vertical stress. The development of lateral soil resistance observed is fairly consistent with that reported by Lyons (1973). The ratio of ultimate lateral resistance to vertical load measured in Lyons' tests was between 0.7 and 1.0, as reported by PRCI (1998).

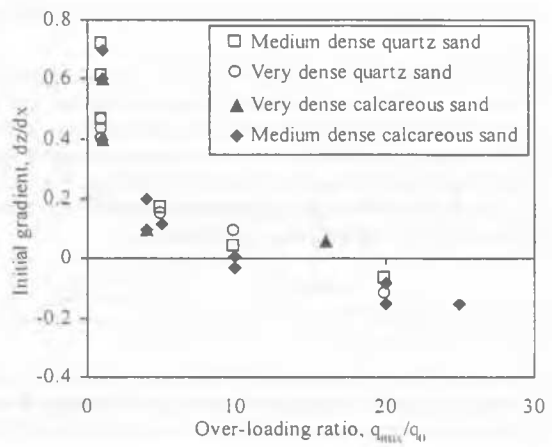

Figure 4. The initial gradient of pipeline movement from probe tests.

The gradient of initial pipe movement did not appear to be affected by stress level or embedment and had an average value of about 0.5 for normally loaded conditions. The initial displacement gradient (dz/dx) decreased significantly with increasing over loading ratio, as shown in Figure 4. The data indicate downward movement under lateral loading when R is less than 10 and upward movement when R is greater. The experimental data clearly indicated coupling between vertical and horizontal responses, meaning that the pipe undergoes vertical movement, when pushed horizontally. This concept has been mentioned in the PRC design guidelines although it was not implemented there because of a shortage of experimental data.

3 MODEL FORMULATION

Within the same framework as that presented by Zhang et al. (1999) the response of a pipeline can be fully described in nominal vertical and horizontal stress (q-h) space. The definition of model parameters can be found in Table 3.

The loading history of a pipeline is represented by the bounding surface, F = 0. This surface, best fitted with sideswipe test results, is described using a parabolic equation, as shown in Figure 5.

$$F = h - \mu\left(\frac{q}{q_{max}} + \beta\right)(q_{max} - q) = 0 \qquad (2)$$

A yield surface $f_b = 0$ is proposed to enclose the small elastic domain in q-h space. It takes the form of a parabola with a size much smaller than that of the bounding surface, such that

$$f_b = h - h_N - \mu\left(\frac{q-q_N}{rq_{max}} + \frac{1+\beta}{2}\right)\left(\frac{1+\beta}{2}rq_{max} - (q-q_N)\right) = 0 \tag{3}$$

The sizes of the bounding and the yield surfaces are defined by the hardening law derived from vertical loading tests, which is expressed using a linear equation, Figure 5.

$$\delta q_{max} = \frac{k_{ve}k_{vp}}{k_{ve}-k_{vp}}\delta z^{p} \tag{4}$$

The yield surface, $f_b = 0$, is simultaneously dragged with the loading point and moves around within the bounding surface like a bubble in accordance with a kinematic hardening rule (Zhang, 2000).

The plastic displacement vector is determined from the bullet shaped plastic potential surface.

$$G = h - \mu_t\left(\frac{q}{q_{max}} + \beta\right)^{m}(q_{max} - q) = 0 \tag{5}$$

For any stress path, the plastic hardening modulus, K, varies from its initial value of K_{max}, to the value of K_C when the yield surface contacts the bounding surface. The following expression is adopted to perform this transition.

$$K = K_C + (K_{max} - K_C)\left(\frac{\delta}{\delta_{max}}\right)^{\rho} \tag{6}$$

where $K_{max} = \lambda\ k_{vp}$ is assumed, δ is the transformed distance between the load point A and the conjugate point C, as shown in Figure 5, and δ_{max} = the maximum value of δ.

The plastic stiffness interpolation parameter ρ changes the rate of the reduction in plastic modulus. Appropriate values for this parameter must be determined from curve fitting of sideswipe or probe test results under over loaded ($q_{max}/q_0 > 1$) conditions. Because the transformed distance $\delta/\delta_{max} < 1$, higher values of ρ will give weaker response.

As such, the incremental load-displacement relation can be obtained using the following expression.

$$\begin{Bmatrix}\delta q\\ \delta h\end{Bmatrix} = \left[D^{ep}\right]\begin{Bmatrix}\delta z\\ \delta x\end{Bmatrix} \tag{7}$$

where D^{ep} is the elastic plastic matrix which can be derived within the framework given by Zhang et al (1999), noting the difference in K values.

From Equations 2, 4, 5 and 7, key performance ratios of pipe-soil interaction in sideswipe and probe events can be obtained through algebraic manipulation. The expressions for some of these ratios are listed in Table 2.

4 MODEL PERFORMANCE

4.1 *Model parameters*

All parameters for the model are described in Table 3. These parameters can be obtained by curve fitting from a set of pipe tests. Some of them may be derived from conventional soil mechanics theory (Zhang, 2001). For calcareous sand, the calibration gives: $m = 0.18$, $\mu = 0.4 + 0.65z_0/D$, $\beta = 0.06$, $\mu_t = 0.6$, $k_{vp} = 350$ kPa/m, $k_{ve} = k_{he} = 20k_{vp}$, $K_{max} = 200k_{vp}$, $\rho = 2$ and $r_0 = 0.01$.

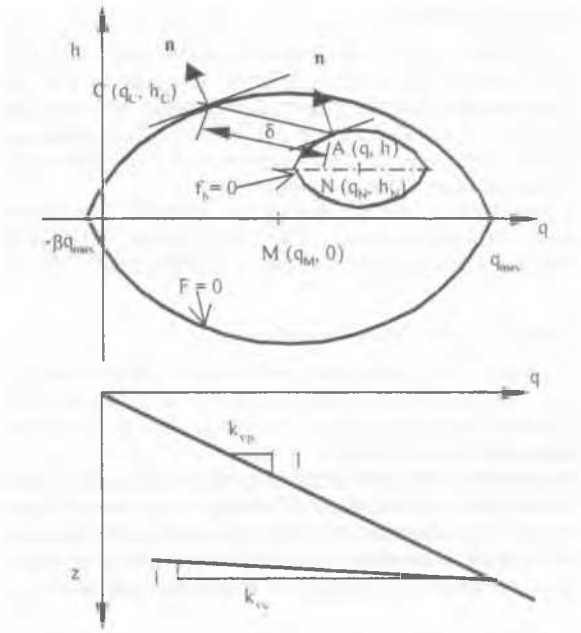

Figure 5. Schematic of the two-surface model.

Table 2. Key performance ratios given by the theoretical model.

Sideswipe	Probe
$\frac{h_{max}}{q_{max}} = \frac{\mu}{4}(1+\beta)^2\frac{1}{1-0.5(1+\beta)k_{vp}/k_{ve}}$	$\frac{h_{max}}{q_{max}} = \frac{m\mu(1+\beta)^2}{(1+m)(m-\beta)}$
$\frac{q@h=h_{max}}{q_{max}} = \frac{1-\beta}{2-(1+\beta)k_{vp}/k_{ve}}$	$\frac{\Delta z}{z_0} = \frac{1+\beta}{m-\beta}$
$\frac{h_{res}}{q_{max}} = \frac{m\mu(1+\beta)^2}{(1+m)^2}$	$\left(\frac{dz}{dx}\right)_{initial} = \frac{\mu_t(1+\beta)^m}{1+\chi k_{vp}/k_{ve}}$
$\left(\frac{dh}{dx}\right)_{initial} = \frac{\chi}{1+\chi}k_{he}$*	$\left(\frac{dh}{dx}\right)_{initial} = \frac{\chi k_{vp}\cdot k_{ve}}{1+\chi k_{vp}/k_{ve}}k_{he}$

* $\chi = \mu\mu_t(1+\beta)^{1+m}\frac{k_{vp}}{k_{he}}\frac{k_{ve}}{k_{ve}-k_{vp}}$

Table 3. Parameters in the theoretical model.

Parameter	Description
k_{vp}	Plastic stiffness of vertical loading
k_{ve}	Elastic stiffness of vertical loading
k_{he}	Elastic stiffness of horizontal loading
β	Intersect of bounding surface on q axis
μ	Shape parameter of bounding surface
μ_0	Shape parameter for surface footings
κ	Gradient of μ increase with embedment
μ_t	Shape parameter in plastic potential equation
m	Exponent in plastic potential equation
ρ	Exponent in the interpolation of plastic modulus
ω	Multiplier of k_{vp} for cyclic loading
K_{max}	Elastic modulus, $K_{max} = \lambda\ k_{vp}$
r	A scale factor for the bubble surface, $r = r_0 R$
r_0	The scale factor for normally loaded pipelines

4.2 *Trenched pipelines*

Figure 6 shows a comparison between model predictions and probe test results for trenched pipelines. Considering that soil disturbance during trenching resulted in a softer response, parameter $\rho = 5$, a constant value $r = 0.05$ and a plastic stiffness $k_{vp} = 200$ kPa/m were used in model simulations. Other parameters are the same as those in the preceding sections.

The comparison indicates that good agreement is achieved between the displacement paths. The peak resistance is relatively well predicted although the initial lateral modulus is not.

4.3 *Probe tests after cyclic loading*

Cyclic loading is a fundamental component of pipeline-soil interaction. The increase in lateral resistance due to accumulated embedment during cycling provides a large contribution to the on-bottom stability of a pipeline.

Measured and predicted pipeline responses in probe events prior to and after cyclic horizontal loading are shown in Figure 7. It can be seen from this figure that the model agrees reasonably well with the experiment. The trend of increasing lateral resistance is well predicted although the exact peak value is not.

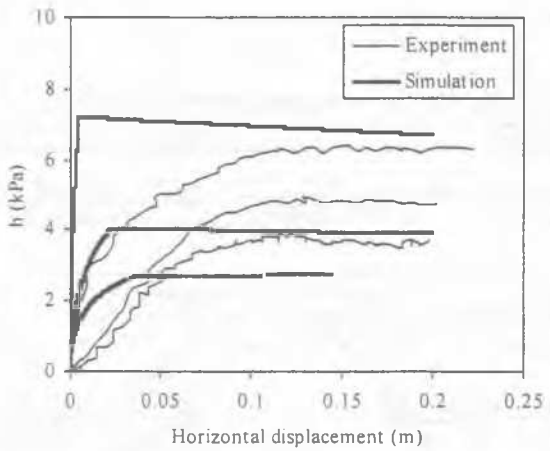

Figure 6. Comparison of the measured and predicted responses for trenched pipelines in calcareous sand.

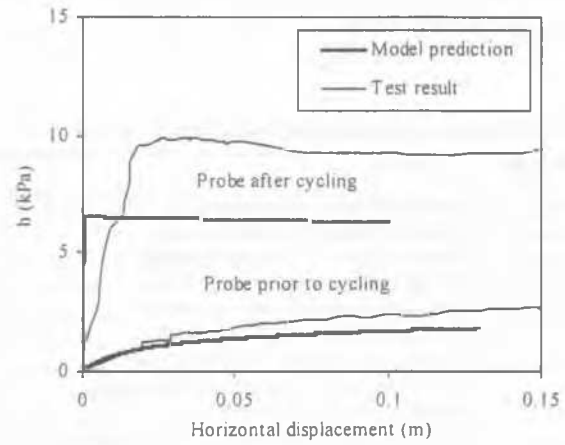

Figure 7. Measured and predicted responses in probes prior to and after cyclic loading, q = 8 kPa for all tests, h = ± 4 kPa in cycling, calcareous sand.

5 CONCLUSIONS

A kinematic hardening two-surface model has been constructed and calibrated for pipe-soil interaction studies. The model provides a suitable basis for simulating the load-displacement response of pipelines of varying embedment under a range of loading conditions.

Attention has been restricted mainly to pipelines on calcareous sand due to space limit although good agreement has also been achieved for pipelines on silica sand (Blinco 2000). The model also has the potential for application to other shapes of foundations, at least in the vertical-horizontal loading plane.

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