Numerical modelling of undrained vertical load-displacement behaviour of offshore pipeline using coupled analysis

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ABSTRACT
Predicting the initial embedment of an offshore pipeline is one of the important design considerations for on bottom stability analysis. This paper reviews the current state of the art to predict the initial embedment, and presents numerical modelling to perform the coupled analysis of pipe-seabed system. Two dimensional plane strain, axis-symmetry model was developed using ABAQUS computer program. Soil was modelled using both Mohr-Coulomb with uniform undrained shear strength and modified Cam-clay obeying Biot consolidation while the pipe was assumed to be rigid. Both undrained and drained loadings were performed in displacement control manner to develop the load-displacement behaviour. The results were benchmarked to the analytical solutions available in literature.

RÉSUMÉ
Prédire l'ancrage initial d'un gazoduc sous-marin est l'un des considérations de conception important pour la stabilité sur l'analyse de fond. Ce document passe en revue l'état actuel de l'art de prédire l'ancrage initial, et présente la modélisation numérique pour effectuer l'analyse couplée de réseau de conduites-sol marin. Deux souche plan bidimensionnel, modèle de l'axe de symétrie a été développé en utilisant le programme informatique ABAQUS. Du sol a été modélisé en utilisant à la fois de Mohr-Coulomb avec la résistance au cisaillement non drainée uniforme et modification de Cam-clay consolidation obéissant à Biot tandis que le tuyau était censée être rigide. Les deux chargements non drainés et drainés ont été effectués de la manière de commande de déplacement à développer le comportement charge-déplacement. Les résultats ont été comparés aux solutions analytiques disponibles dans la littérature.

1 INTRODUCTION

Pipelines are increasingly becoming an important part of the offshore infrastructure as more petroleum resources explored further away from shore. The design measures of the pipelines have given growing attention following the increased usage of extra long pipelines on deep seabed conditions. The pipeline embedment has became an important design parameter to control pipe vulnerability to external interactions such as heat expansion, free spanning, wave oscillation and submarine slides etc (Bruton et al. 2008; Randolph and White 2008; White and Randolph 2007). However, much attention is paid to understand the influence of initial embedment to control the expansion related challenges: (1) axial walking (when expands along the axis of the pipe); (2) lateral buckling (when buckles laterally).

Predicting the initial embedment could be simplified to static loading of a pipe on cohesive soil medium. In the past, undrained pipeline penetration in cohesive soil was simplified to traditional bearing capacity theory for flat surface on a shallow embedment (Ghazzaly and Lim 1975; Karal 1977; Small et al. 1971; Wantland 1979). However, by means of classical plasticity approach, Randolph (1984) estimated the limiting pressure of a laterally moving cylindrical body fully embedded in cohesive soils. Later Murff et al (1989) adopted and extended these plasticity solutions to analyse partially embedded pipe penetration in cohesive soils where the presence of free surface is prominent.

Finite element method has been widely employed to study the nonlinear behaviour of pipe-seabed interaction recently. The vertical load-displacement behaviour of pipe-cohesive soil with linearly increasing shear strength profile and no weight was reported by Aubeny et al (2005). Prior to the analysis, the pipe was placed in a predefined depth called wished in pipe (WIP) as shown in Fig .1.a. Merifield et al (2008) conducted FEM analyses on pipes subjected to combined loading of a WIP in horizontal and vertical directions on weightless uniform soil. Later, Merifield et al (2009) reported that the vertical collapse load is higher in the pushed in pipe (PIP) (Fig. 1. b) case due to the passive influence of heave as additional work is required to displace the soil around the pipe periphery.

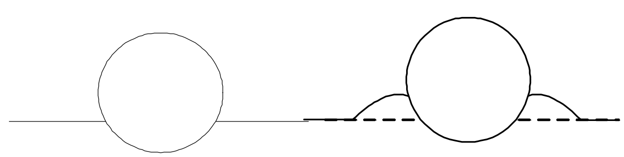


Fig.1.a. Wished in pipe (WIP) b.Pushed in pipe (PIP)

The reliability of the undrained approach to predict the accurate initial embedment is questioned over time. In reality, loading of a pipe on cohesive soil will initially be taken by the soil pore pressure, keeping the load carried by the soil skeleton to minimum. But there is substantial time lag between pipe laying and operational stages resulting dissipation of pore pressures causing a gradual rise in effective stresses at the pipe invert. This time dependent pore pressure governance is critical for on bottom stability analysis in the following ways.

1. Local consolidation below the pipe invert causes the increase in embedment with time. Therefore the predicted undrained embedment, which is soon after the pipe lay is conservative.

2. The time delay between initial lay and 100% consolidation is important for accurate estimation of pipeline stability by considering positive or negative pore pressures at the pipe invert.

3. For axial walking, the degree of excess pore pressure affects whether the soil will behave drained, undrained or in-between manner as the pipe displaces.

Therefore, the initial embedment of a pipe will be better understood by following an effective stress method with concurrent measurement for pore pressure dissipation (i.e. coupled analysis). Recently Gourvenec and White (2010) and Krost et al.(2011) reported a FEM based elastic consolidation analysis for WIP condition. Here the pipe was placed for the predefined embedment and quick loading was applied while the subsequent dissipation was inquired against time. However as they indicated, the main setback of this analysis was the absence of shear generated excessive pore pressure by avoiding continuous loading from the surface (PIP loading). Indicating the importance of more representative soil models (such as Cam clay) with continuous loading (PIP) from surface to account for the shearing effect and the Over Consolidation Ratio (OCR) related pore pressure influence.

The current paper presents a detailed analysis to predict the embedment development of a PIP. The seabed was modelled with both Mohr Coulomb with uniform undrained shear strength and modified Cam-clay with Biot's consolidation. An undrained total stress analysis was carried out with the first thus the time dependency of pore pressure dissipation was avoided. For the latter, an effective stress coupled analyses was employed to model the undrained condition with subsequent pore pressure dissipation. The *ABAQUS* (Dassault Systemes 2009) software was used for this purpose. The capability of coupled modified Cam-clay to predict vertical resistive force for the change in initial embedment is reviewed and the modeling techniques are discussed.

2 METHODOLOGY

2.1 Non Dimensional Analysis

As indicated in the literature the undrained approach provides a useful approach to pipe embedment, particularly for clay soils. The undrained load-displacement relationship of vertical pipe penetration into the seabed in terms of governing parameters can be explained by the following functional form.

$$V = f(E_u, S_u, D, w, \upsilon, \gamma', C_i) \qquad [1]$$

where V = maximum vertical load per unit pipe length, E_u =Young's modulus, S_u =Undrained shear strength, D = Diameter of the pipe, w = Pipe embedment measured from initial seabed surface, υ = Poisson's ratio, C_i = Pipe-soil adhesion, γ' = Submerged unit weight of soil. The dimensions of the variables of equation 1 are $[V] = FL^{-1}$, $[E_u] = FL^{-2}$, $[S_u] = FL^{-2}$, $[D] = L$, $[w] = L$, $[C_i] = FL^{-2}$, $[\gamma'] = FL^{-3}$. Altogether, there are eight variables and two reference dimensions could be used to formulate six non-dimensional groups. Using non-dimensional analysis the functional form could be normalised by using two independent variables, namely S_u and D.

$$\frac{V}{S_u D} = \phi\left(\frac{E_u}{S_u}, \frac{w}{D}, \upsilon, \frac{C_i}{S_u}, \frac{\gamma' D}{S_u}\right) \qquad [2]$$

Both pure undrained and coupled FEM analyses were carried out to investigate the influence of each non-dimensional group identified in Eq 2. The results are presented for the non-dimensional vertical collapse load $\frac{V}{S_u D}$ against non-dimensional embedment $\frac{w}{D}$.

2.2 FEM Undrained

A detailed 2D FE analysis was undertaken to examine the collapse load at varying initial embedment. The mesh and the dimensions used in the analyses are illustrated in Figure 2.

Plane strain conditions were assumed and the advantage of using the symmetry of the vertical axis passing through the centre of the pipe was accounted for. The soil was modelled to behave as elastic perfectly plastic continuum satisfying *Mohr-Coulomb* failure criterion with uniform shear strength over the domain. The pipe was modelled as a rigid body since any deformation in the steel pipe would be negligible in comparison to the soil deformation in the context of this simulation. An Arbitrary Lagrangian-Eulerian (ALE) remeshing algorithm was employed during the large soil deformation to allow for geometric nonlinearities.

It is important to consider the interaction between the pipe and the soil, and this was achieved by defining surface to surface contact, with pipe as the master and soil as the slave. Interaction properties were defined as frictionless (i.e., tangential forces are zero) to simulate the smooth surface conditions. No unloading and reloading was performed. The soil was modelled using CPE8R: 8-node biquadratic plane strain quadrilateral reduced integration elements (Dassault Systemes 2009).

The pipe was modelled using R2D2: a 2-node 2-D linear rigid link elements, since the pipe elements were tied to a single reference point from where the displacement of the pipe was calculated.

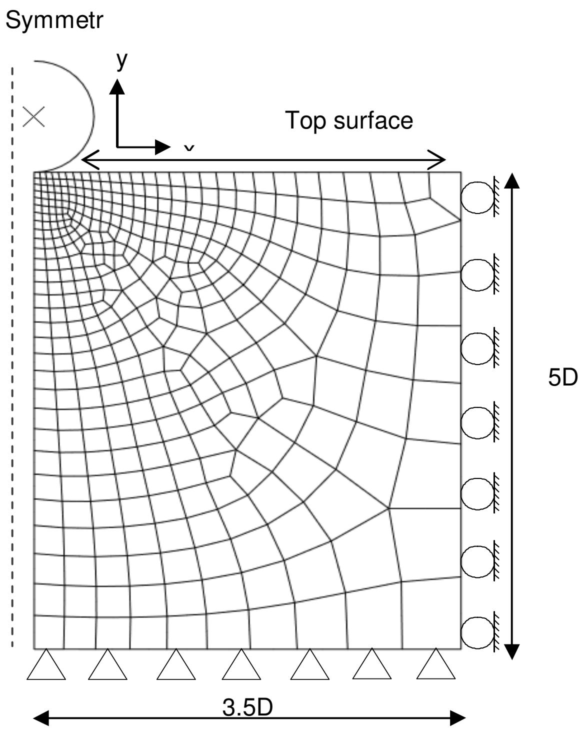


Figure 2. Mesh and boundary conditions

Analyses were carried out only for the PIP conditions. The increments were applied to the pipe as displacement controlled steps for the w/D range of 0.1 to 0.5. In each case, the resultant vertical force was calculated vectorially summing the vertical nodal forces at the contacts.

3.3 Coupled Analysis

Coupled analysis is more rigorous and with a more realistic material constitutive model can provide solutions over a range of parameters. The advantage of using a coupled system over the pure drained/undrained loading is the capability to measure the time depended pore pressure dissipation to study the influence of pore pressure on the response of the system.

The same FEM mesh shown in Fig 2 was adopted for the coupled analysis. Here the soil was selected to be CPE8RP bilinear pore pressure element. The soil was assigned to obey modified Cam clay with pore pressure dissipation with Biot model. Cam clay model properties are given in Table 1. The contact and the other properties were kept ideal to the model performed in the above analysis. However the use of advanced constitutive model like modified Cam clay over Mohr Couloumb has added its own restrictions to be considered, which can be explained as follows.

Table 1. Model properties- Modified Cam clay

Parameter	Value
λ	0.5
κ	0.05
$\sigma'_{v\ top}$	20kPa
K_0	$1-\sin\phi$
M	$\frac{6\sin\phi}{3-\sin\phi}=1$
G	660kPa
K	$10^{-9}ms^{-1}$

1.The inclusion of soil weight to the analysis, which is essential to ensure the initial stress of the soil profile falls within the State Boundary Surface of the Modified Cam clay. Unlike in pure undrained analysis the self standing soil weight (γ') is not an essential parameter to consider unless its effect itself is studied.

2. The coupled pore fluid analysis capable to provide solutions either by "excess" pore pressure or "total" pore pressure measures, Total pore pressure solutions are provided when the gravity distributed load is used to define the gravity load on the model. Excess pore pressure solutions are appropriate in all other cases when gravity loading is defined with body force distributed loads. Since, being an undrained approach to determine the resistive load on an embedding pipe, the total pore pressure was used in this application by invoking gravity load to the soil mass. However, instances such as examining the effects of consolidation the geostatic loading have to be defined as body forces to study the excess pore pressures and their variation.

3. The Modified Cam clay model itself is nonlinear and computationally very expensive to model with. Thus it is advantageous to define confining pressures in MPa, instead of the commonly used kPa to prevent the stiffness matrix having very large values. Thus e_1 , e_2 and e_{cs} of the Figure 3 have to be defined with MPa.

4. Unlike in pure undrained analysis, the strain rate is an important parameter in coupled analysis which dictates whether the loading is drained/undrained. Thus the excess pore pressure generation is recorded along with the rate of loading while maintaining the top boundary condition representative of the undrained condition.

5. Being a model controlled in three dimensional stress space the lateral confining K_o condition is essential for Modified Cam clay analysis. Thus Jacky's (Jaky 1944)

relationship was used to establish $K_0 = 1 - \sin\phi$ where ϕ is the soil friction angle.

6. The modified Cam-clay model in *ABAQUS* allows the user to specify either a constant shear modulus or a constant Poisson's ratio. When *G* is defined the model assumes it as constant and the Poisson's ratio changes accordingly. Here in this analysis the constant G value was maintained for $\frac{E_u}{S_u} \approx 400$ and the Poisson's ratio for each was calculated and crosschecked to be within $0.1 \leq \upsilon \leq 0.3$ to keep the consolidation effect due to the compressibility of the soil to be minimum (Krost et al. 2011).

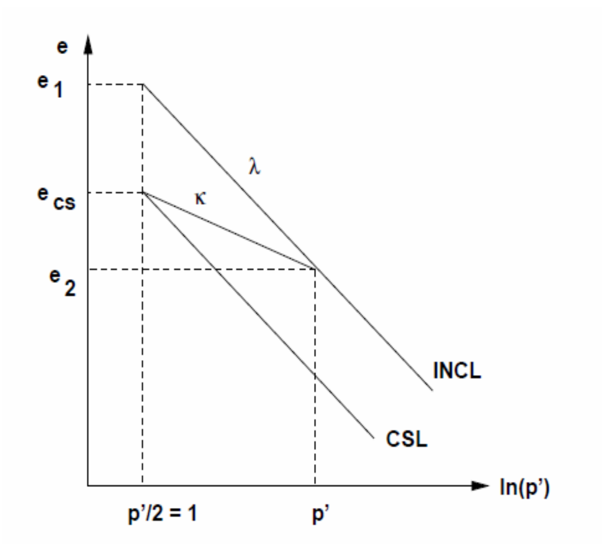


Figure 3. *p'-e* plot modified cam clay

The loading sequence is very important for coupled analysis. Here both load and displacement controlled steps were used. The loading sequence was carried out in the following individual steps.

1. Geostatic step- to ensure the equilibrium is achieved
2. Displacement step- vertical displacement of the pipe from 0 to 0.5D with an impermeable top surface
3. Transition step- change from displacement controlled to load controlled loading
4. Consolidation step- top surface boundary changed to be permeable

Geostatic step- This step has to be the first step in any two phase loading sequence to ensure the converged stresses for the start of the analysis. Here whether the initial geostatic stress field is in equilibrium will be verified with the applied loads and boundary conditions. Iterations will be performed until the equilibrium is achieved with in the required tolerance level. Thus the displacement that occurs during the geostatic step is not due to the external loading, but due to the difference between the user predicted initial stresses and the converged stresses calculated by *ABAQUS*. Therefore the effectiveness of using a geostatic test has been confirmed by keeping the deformations to be less than the minimum tolerance level. As indicated before the soil weight could be applied by gravity or body force distribution corresponding to the total and effective pore pressure measurements. In this undrained analysis, the gravity was defined according to the bulk density of the soil, and the geostatic equilibrium was verified to ensure the stress convergence.
Further defining initial conditions such as top and bottom pressures and the relevant void ratio (Fig 3) are other important considerations of this step. When executed this step computes the geostatic stress at each point and its corresponding void ratio. Later It will be compared with the State Boundary Surface of the modified Cam clay where the OCR of the soil will be estimated.
Displacement step- Similar to the PIP loading mentioned above, here the vertical displacement of the pipe from 0 to 0.5D was performed by displacement controlled loading. Unlike the pure undrained loading performed under Mohr-Coulomb the loading rate determines pore pressure governance around and below the pipeline. Initially the rate of loading was kept fast enough to remain undrained and the top boundary of the soil was kept to be impermeable. The undrained loading was assured by monitoring the pore pressure measurement at the top soil surface. The resultant resistive vertical force was calculated.

Transition step- This is an important step to be performed to permit the pipe to be eligible for consolidation related subsequent settlement. This is carried out comparatively in a very short time period. Here the resistive force experienced by the pipe in the previous step was applied to the pipe. The displacement load of the previous step was deactivated while the load was applied slowly where from the subsequent consolidation effect could be monitoring.

Consolidation step- The actual consolidation process was invoked in this step. The top boundary of the soil which comes in to contact with the pipe was selected and made permeable by assigning the pore pressure boundary condition to be zero. Thus excess pore pressures are permitted to dissipate through the top surface and the time depended settlement due to consolidation has been captured. The choice of the initial time step is an important issue in numerical modelling. Exceedingly small time steps can give model divergence and termination due to numerical instability. The change in excess pore pressure with minimum time step during consolidation was defined by Vermeer and Verruijt (1981). This could be adopted for FEM consolidation analysis, where the Equation 3 introduces a relationship between the minimum usable time increment (t_{min}) and the element size.

$$t_{\min} = h^2 \frac{\gamma_w}{6EK} \quad [3]$$

h - *minimum element size*

E - *Young's modulus of soil*

K - *Permeability*

γ_w - *Specific weight of water*

3 MODEL VALIDATION & RESULTS

3.1 FEM Undrained

The results were benchmarked to the previous theoretical solutions developed for weightless soil with uniform shear strength and E_u/S_u = 400. The closeness of the curves in Figure 5 validates the model dependability. It is clear that for weightless soils, the presence of heave during PIP analysis have no significant influence on the vertical collapse loads as both the PIP FEM and WIP based analytical solutions falls close to each other. The power law was employed to capture the associated relationship as given in Equation 4.

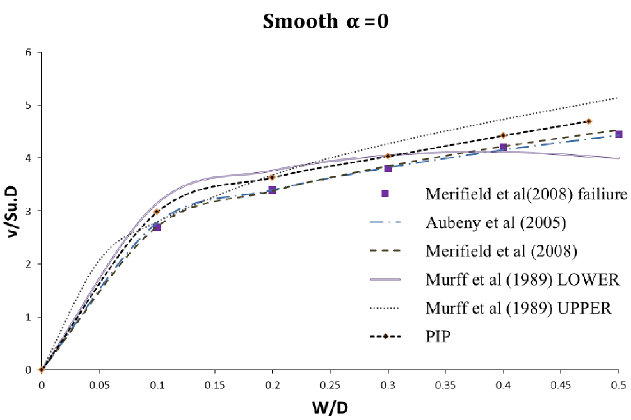
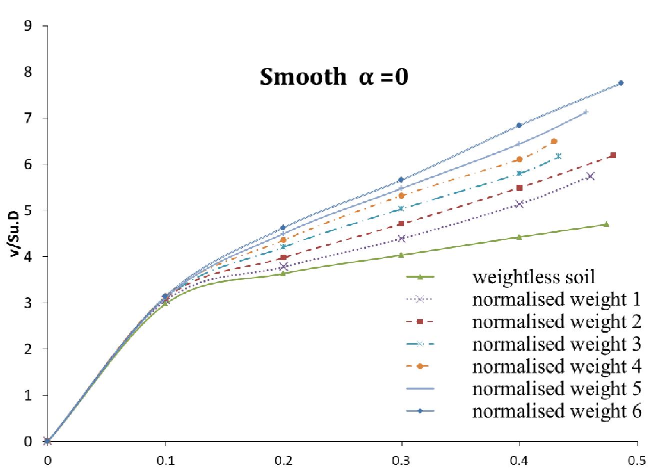


Figure 4. Comparison of collapse load for vertical pipe penetration -Smooth contact conditions, weightless soil

$$\frac{V}{S_u D} = 5.77\left(\frac{w}{D}\right)^{0.29} \quad [4]$$

However the influence of changing soil weight was unclear which is the case in Modified Cam clay analysis.



Therefore a series of FEM analyses were performed for a constant *Eu/Su* ratio of 400 with changing buoyant soil weight. The Figure 5 presents the influence of the non-dimensional soil weight in the range of 0 to 6. As Merifield et al (2009) explained this could be due to the large amount of energy that is needed to displace the soil around the pipe perimeter. The change in trend caused by the normalised unit weight and soil heave was captured by Equation 5.

Figure 5. Influence of *γ'D/S*$_u$ on vertical collapse load, PIP loading

$$\frac{V}{S_u D} = \left(5.77 + 0.76\left(\frac{\gamma' D}{S_u}\right)^{1.09}\right)\left(\frac{w}{D}\right)^{(0.29+0.043\frac{\gamma' D}{S_u})} \quad [5]$$

3.2 FEM Coupled Analysis

Benchmarking of a coupled analysis is comparatively difficult. Any possible way to benchmark a coupled analysis could be against a pure drained/undrained loading because any loading in between these extremes cannot be tracked down and classified properly. Here, fast undrained loading of the pipe on a Cam clay elasto plastic coupled medium has been benchmarked to the pure undrained FEM analysis carried out using the *Mohr-Coulomb* soil medium discussed in the previous section. As indicated before the soil weight is a parameter in the Cam clay analysis, thus the vertical penetration resistance for normally consolidated clay with unit normalised weight obeying coupled Cam clay constitutive model and the Mohr coulomb pure undrained loading are illustrated in Figure 6.

The soil with normalised unit weight the Cam clay and Mohr coulomb vertical resistances are close to each other but tend to diverge when the depth of embedment exceeds 0.5D. The closeness of these curves justifies the application of Cam clay coupled analysis to model the vertical resistance for an embedding pipe. However, the failure to accommodate exact matching could be due to the diversity of these constitutive models representing the soil itself. For instance in Cam clay coupled analysis the deviotaric load depended pore pressure generation is a significant advantage while this is not considered in Mohr-Coloumb model. Further the undrained shear strength given in Eq 6 and Young's modulus given in Eq 8 of the Cam clay are stress depended and varies over the soil profile due to the soil weight.

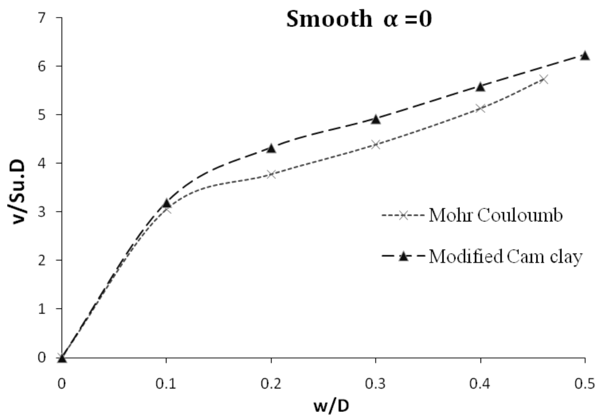


Figure 6. Undrained Vertical collapse load for Mohr Coulomb and Modified Cam clay ($\gamma'D/Su$=1)

3.3 Pore Pressure

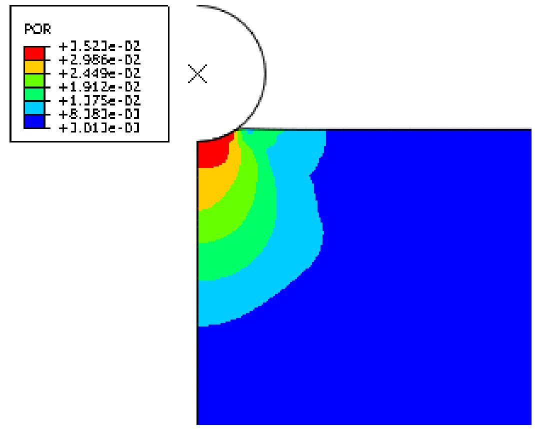
$$S_u = \frac{1}{2} M \left(\frac{e_{cs} - e_0}{\lambda} \right) \qquad [6]$$

Where

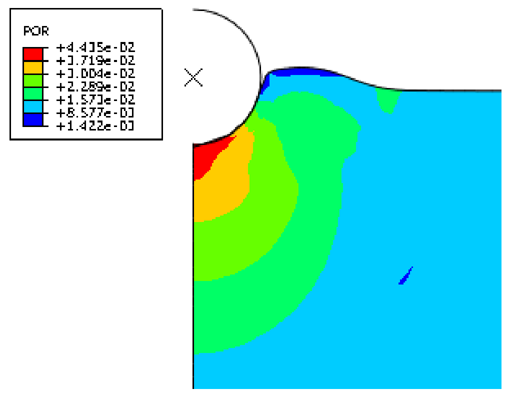
$$e_{cs} = e_1 - (\lambda - \kappa) \ln 2 \qquad [7]$$

$$E = \frac{3(1-2\nu)(1+e_0)p}{\kappa} \qquad [8]$$

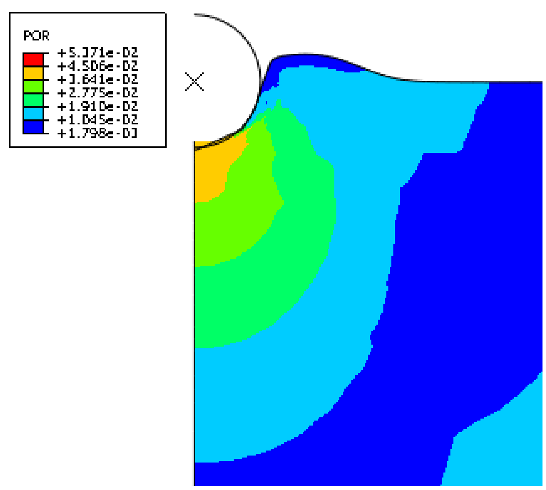
The increase in pore pressures with embedment is illustrated in Figure 7. A concentration of pressure contours was observed around the pipe embedment and was found to extend towards the depth direction with increasing embedment. The influence of the free surface is eminent as the pressure profiles were deep at the invert and they tend to converge at the pipe free surface. Therefore the zone of pore water influence of an embedding pipe is a function of θ as illustrated in Figure 8, unlike the fully embedded cylindrical body in a soil medium.



(a)



(b)



(c)

Figure 7. Pore pressure distribution within the soil domain with changing embedment w/D: (a) 0.1, (b) 0.3 and (c) 0.5

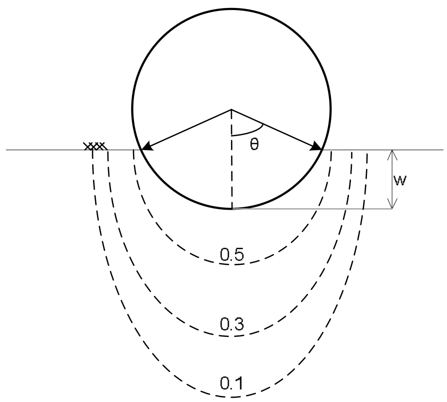


Figure 8. Normalised Pore pressure (Δu/u) contours for w/D=0.3

The excess pore pressure extends deeper at the pipe invert and this effect gets larger with increasing embedment. The Figure 9 illustrates the normalised excess pore pressure (Δu/u) vs. normalised embedment at the pipe invert. The sudden increase in pore pressure at the beginning is mainly due to the initial contact established between the pipe and soil. Initially the resistive load was higher considerably on small contact area, keeping the pore resistive pressure to be high. However as the pipe embed deeper the increase in resistive force tends to fall as well as the change in contact area becomes negligible. Thus the increase in pore pressure with embedment becomes comparatively small.

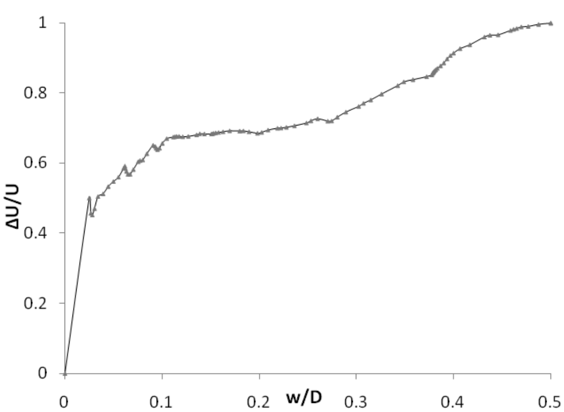


Figure 9. Normalised pore pressure with changing embedment w/D

4 CONCLUSION AND DISCUSSION

The FE analyses were conducted to study the undrained vertical load-deformation behaviour of offshore pipelines using a relatively simple Mohr coulomb constitutive model and more sophisticated Modified cam clay model with pore pressure dissipation. The modelling techniques and sequential steps are elaborated. The pure undrained analyses numerical results were benchmarked against the numerical and analytical studies identified in the literature. The influence of heave was studied with different γ' and an equation was developed for vertical load-displacement behaviour as a function of γ'. The coupled undrained loading was performed and benchmarked to the pure undrained analysis for the presence of γ'. From 0 to 0.1D embedment the resistive loads were identical, but as the embedment increases the loads tends to diverge. This was explained as the diversity of the material model itself and the closeness of the curves could be justified on the basis of a subsequent drained test that is currently under investigation by the authors.

Further using a nonlinear soil model such as Cam clay with pore fluid motion is an advanced and computationally demanding numerical modelling exercise. Load controlled steps are the most ideal to replicate the consolidation problem. However, the initial loading was performed with displacement controlled loading since this is considered to be free of rigid body motion and computationally more effective for contact problems.

Nevertheless, the further analysis for drained behaviour demands the displacement constraints to be transformed to load controlled for subsequent consolidation. The most practical recommendation could be back applying the resistive force encountered in the load controlled step to initiate the constant load consolidation process. This provides a more realistic way of modelling embedding pipe, because the buoyant pipe weight itself is the one to cause the initial undrained embedment. There after the subsequent consolidation can be invoked with the constant pipe weight, instead of applying an arbitrary load to a specific embedment to understand the time dependant influence of the pore pressure regime.

5 REFERENCES

Aubeny, C.P., Shi, H., and Murff, J.D. 2005. Collapse loads for a cylinder embedded in trench in cohesive soil. International Journal of Geomechanics, **5**(4): 320-325.

Bruton, D., Carr, M., and White, D.J. 2008. Pipe-Soil Interaction during Lateral Buckling and Pipeline Walking — The SAFEBUCK JIP. Offshore Technology Conference, 5-8 May 2008, Houston, Texas, USA.

Dassault, and Systèmes 2009. Abaqus analysis users manual.

Ghazzaly, O.I., and Lim, S.J. 1975. Experimental investigation of pipeline stability in very soft clay. Offshore Technology Conference, 5-8 May , Houston, Texas, USA: 315-326.

Gourvenec, S.M., and White, D.J. 2010. Elastic solutions for consolidation around seabed pipelines. Offshore Technology Conference, 3-6 May 2010, Houston, Texas, USA

Jaky, J. 1944. The coefficient of earth pressure at rest. Journal for Society of. Hungarian Architects and Engineers(Budapest): 355-358.

Karal, K. 1977. Lateral stability of submarine pipelines Proceedings of the Annual Offshore Technology Conference, **4**: 71-78.

Krost, K., Gourvenec, S.M., and White, D.J. 2011. Consolidation around partially embedded seabed pipelines. Geotechnique, **61**(2): 167-173.

Merifield, R., White, D.J., and Randolph, M.F. 2008. The ultimate undrained resistance of partially embedded pipelines. Geotechnique, **58**(6): 461-470.

Merifield, R.S., White, D.J., and Randolph, M.F. 2009. Effect of surface heave on response of partially embedded pipelines on clay. Journal of Geotechnical and Geoenvironmental Engineering, **135**(6): 819-829.

Murff, J.D., Wagner, D.A., and Randolph, M.F. 1989. Pipe penetration in cohesive soil. Geotechnique, **39**(2): 213-229.

Randolph, M.F. 1984. Limiting pressure on a circular pile loaded laterally in cohesive soil Geotechnique, **34**(4): 613-623.

Randolph, M.F., and White, D.J. 2008. Pipeline Embedment in Deep Water: Processes and Quantitative Assessment. OTC.

Small, S.W., Tambruell, R.D., and Piaseckyj, P.J. 1971. Submarine pipeline support by marine sediments. Proc. 5 Offshore Technology Conference, **1**: 309-318.

Vermeer, P.A., and Verruijt, A. 1981. An accuracy condition for consolidation by finite elements. International Journal for Numerical and Analytical Methods in Geomechanics, **5**(1): 1-14.

Wantland, G.M. 1979. Lateral stabilities of pipelines in clay. Proceedings - Offshore Technology Conference, **v**(2): 1025-1034.

White, D.J., and Randolph, M.F. 2007. Seabed characterisation and models for pipeline-soil interaction. International Journal of Offshore and Polar Engineering, **17**(3): 193-204.