Numerical Analysis of Wave-Induced Liquefaction around Buried Pipeline

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Abstract
This extended abstract presents the numerical solutions from the Finite Element analyses based on a wave flume test in order to investigate the instability of a buried marine pipeline due to wave-induced liquefaction. A soil model based on the Generalised Plasticity Concept namely Pastor-Zienkiewicz Model Mark-III (PZ3) implemented in a Finite element program namely SWANDYNE II is used to predict the behaviour of a seabed subjected by a two-dimensional progressive wave system. The verification of the finite element program and the validation of PZ3 model in simulating the progressive nature of wave induced liquefaction on seabed were performed by Vun *et al.* (2003). The numerical analyses successfully reproduced most of the experimental findings from the wave flume tests conducted by Teh *et al.* (2004) and Sumer et al. (2004). The effects of the pipe and the trench geometries, the wave condition and the soil parameters on the liquefaction potential of the soil around a buried marine pipeline are studied and presented in this paper.

Introduction
The present study aims at reproducing the experimental findings of both Teh *et al*. (2004) and Sumer *et al.* (2004) through a plane strain elasto-plastic analysis. The dynamic finite element program originally developed for earthquake problems namely SWANDYNE II (Chan, 1988) was modified and used in present study. Pastor-Zienkiewicz Model Mark-III (PZ3) (Pastor and Zienkiewicz, 1986) was adopted to simulate the pore-water pressure build up due to wave loading. In the previously study (Vun *et al.*, 2003), the applicability of SWANDYNE II program to the wave-induced soil response problem was verified with the available analytical solution by Jeng and Hsu (1996) and the capability of the PZ3 model in predicting the progressive nature of wave-induced liquefaction on a deposit of silt was validated by the experimental data from Teh *et al.* (2004). A series of parametric studies have been carried out to verify the hypotheses observed from the wave flume experiments conducted by both Teh *et al*. (2004) and Sumer *et al.* (2004) which were used to investigate the instability in pipeline founded in non-cohesive soil bed subjected to varies wave loading.

Experimental Findings
The primary findings from two sets of the experiments were:

Experiments by Sumer et al. (2004): 1) The excess pore pressure built up more rapidly in the vicinity of the pipeline than in the far field (particularly underneath the pipeline); 2) The boundary condition for the pipeline (i.e. roughness, degrees of freedom) has a significant effect on the rate of pore pressure build up; 3) The diameter of the pipeline does not have a large influence on the rate of pore pressure build up; 4) Around the pipeline, liquefaction first begins under the pipeline and then spreads upwards around the perimeter. This is opposite to the far field where liquefaction begins at the surface and moves vertically downwards.

Experiments by Teh et al. (2004): The sinking and the flotation of a pipeline in a liquefied soil depend primarily on the difference in density between the pipeline and the soil.

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Finite Element Analysis

Plane strain analysis was adopted to simulate the instability of a marine pipeline founded in a non-cohesive saturated seabed subjected to sinusoidal wave loading with the wave length of *L* and a wave period of *T*.

Progressive wave loading was facilitated by prescribing the pore pressure *p*, effective vertical stress σ_z, effective horizontal stress σ_x and shear stress τ_{xz} on the soil surface (z = 0) as follows

$$p = p_0 \sin(\lambda x - \omega t); \ \sigma_z = 0; \ \tau_{xz} = 0 \qquad \text{at } z = 0 \qquad \textbf{(1)}$$

where p_0 is the amplitude of the fluid pressure fluctuation imposed on the soil surface (z = 0); $\lambda = 2\pi/T$ is the angular frequency of the wave.

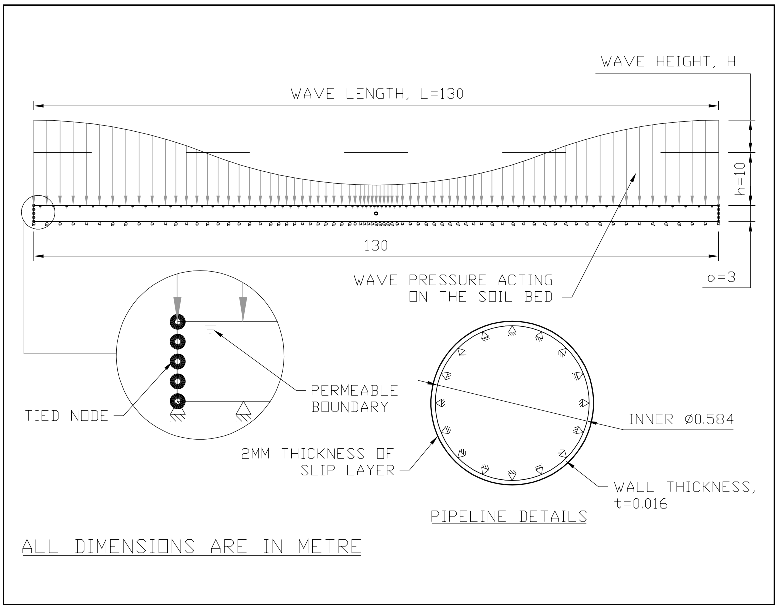

Figure 1: Layout of prototype unmovable buried pipeline analyses

Figure 1 and 2 show the finite element set up for the defined problem. It is assumed that a sand layer overlays an impermeable layer and the buried pipeline is buried in a backfilled trench.

A series of parametric studies were conducted in this study and a list of the various parameters used for each analysis is given in Table 1. Analysis BPA1-0 was set as the reference case of the parametric study therefore the mesh setting and soil properties for other analyses can be referred to the reference case except the parameters needed to be studied. The input parameters of loose sand (base case) and dense sand for the PZ3 model are listed in Table 2.

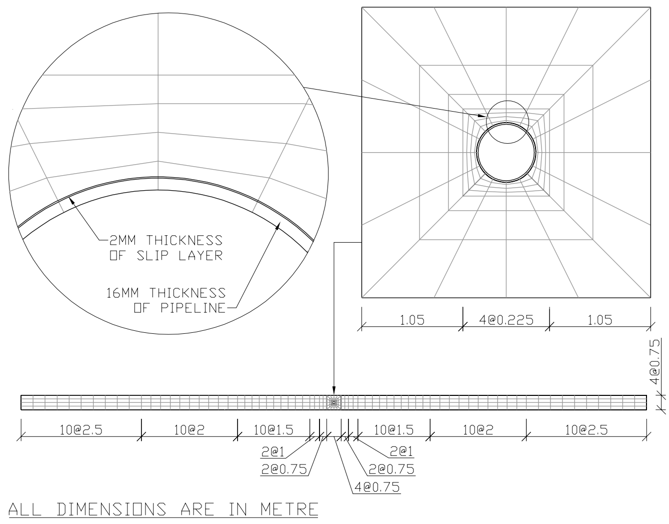

Figure 2: Finite Element mesh

where $|p_0|$ is the amplitude of p_0; e_c is the verital distance from the mudline to the pipeline centre; *Kev0c* is the bulk modulus at the mean effective stress p_0'; Kes0c is the three times shear modulus at p_0'; M_f is the slope of the characteristic line for the yield surface; M_g is the slope of the Critical State line for the determination of the plastic strain vector; α_f and α_g are the relationship of the dilatancy with stress ratio for both loading vector and plastic strain vector respectively; β_0 and β_1 are the constant parameters; H_0 is the plastic modulus for loading; H_{u0} is the plastic modulus for unloading; ν is the Poisson's ratio; k is the permeability; G_s is the specific gravity; e is the void ratio; and S_r is the degree of saturation.

Table 1: Summary of buried pipeline cases

Analysis	G_s *Pipe*	$\lvert p_0 \rvert$ (m)	L (m)	T (s)	d (m)	e_c (m)	D (m)	Fixity	Slip	Trench	Far field
BPA1-0*	2.1	25	130	10	3	1.5	0.6	Fixed	No	Loose	Loose
BPB1-0	2.1	25	130	10	3	1.5	0.3	Fixed	No	Loose	Loose
BPC1-0	2.1	25	130	10	5	1.5	0.6	Fixed	No	Loose	Loose
BPE1-0	2.1	25	130	10	3	1.5	0.6	Fixed	Yes	Loose	Loose
BPF0-0	2.1	25	130	10	3	1.5	0.6	Free	Yes	Loose	Loose
BPF0-1	2.1	40	130	10	3	1.5	0.6	Free	Yes	Loose	Loose
BPF0-2	1.3	40	130	10	3	1.5	0.6	Free	Yes	Loose	Loose
BPA1-1	2.1	25	130	10	3	1.5	0.6	Fixed	No	Dense	Loose
BPG1-0	2.1	25	130	10	3	1.5	1.2	Fixed	No	Loose	Loose

Another constitutive model, namely Slip Element Five, in the SWANDYNE II program was used for modelling the interaction between the structure and the surrounding soil. This model is based on the Coulomb friction criterion and it is capable to simulate the slip effect according to the slip element direction. The friction angle ϕ' of the interface layer was assumed to be 35° and the cohesion was assumed to be zero as non-cohesive silt was used in the test. The interface layer was assumed to have an Young's modulus of 1MPa. The parameters of e and G_s for the interface layer were referred to the loose sand parameters given in Table 2.

Table 2: FE model parameters for loose and dense sand analyses

Parameter	Unit	Dense sand	Loose sand
Kev0c	MPa	2	0.77
Kes0c	MPa	2.6	1.155
M_g	-	1.32	1.15
M_f	-	1.3	1.035
α_g	-	0.45	0.45
α_f	-	0.45	0.45
β_0	-	4.2	4.2
β_1	-	0.2	0.2
H_0	-	750	600
H_{u0}	MPa	40	40
ν	-	0.31	0.29
k	ms^{-1}	10^{-4}	10^{-4}
p'_o	kPa	10	10
G_s	-	2.65	2.65
e	-	0.6	0.8
S_r	-	1.0	1.0

Results and Discussions

The comparison between the rate of pore pressure build-up of far field and around the pipe for the cases listed in Table 1 are summarised in Table 3.
where N_{far} and N_{pipe} are the numbers of cycles to liquefaction at z=1.5m below mud-line in far field and in the vicinity of pipeline respectively.

The results show that liquefaction takes place more quickly in the vicinity of the pipeline than in the far field for the fixed pipeline. For the free pipeline, liquefaction approximately occurs at the same time at both locations (cases BPF0-0 to 2). This is obviously resulted by the fully fixed boundary condition of pipeline in which additional shear strain is induced when the soil tries to deform around the pipeline and which causes a faster rise in excess pore pressure. Besides, the soil around a fixed pipeline liquefies more slowly if the soil around the trench is denser. Although the pipeline diameter does affect the rate of pore pressure build-up, the influence is not significant. Liquefaction around the pipe was predicted to occur from the top to the bottom in the numerical model and which is in contrast with the experimental findings of Sumer *et al.* (2004). The disagreement could be due to the insufficient compaction to the soil zone just beneath the fixed pipe during the soil bed preparation.

Figure 3 shows the floatation and the sinking of pipeline due to the decrease in the effective stresses of seabed. It exhibits that the pipeline sinks when the G_s of pipe is less that the submersed G_s of soil and floats when the Gs of pipe is greater than the G_s of soil.

Table 3: Summary of results for buried pipeline cases

Test	N_{far}	N_{pipe}	N_{far}/N_{pipe}
BPA1-0	26	21	1.24
BPB1-0	26	23	1.13
BPC1-0	23	20	1.15
BPE1-0	26	21	1.24
BPF0-0	26	24	1.08
BPF0-1	12	11	1.09
BPF0-2	12	11	1.09
BPA1-1	Not Liquefied	30	-
BPG1-0	26	19	1.37

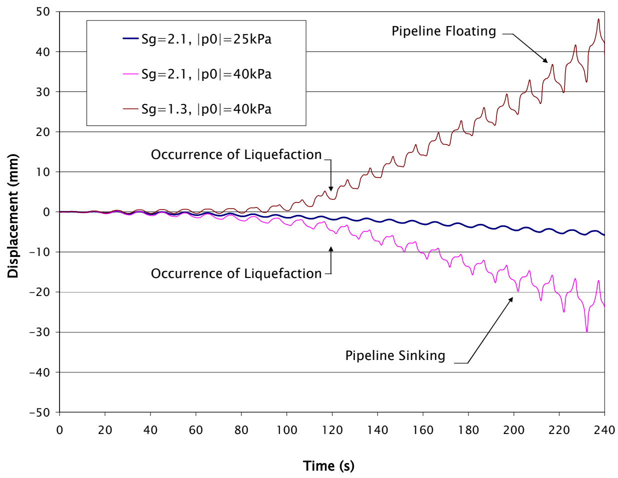

Figure 3: Vertical displacement of two different specific gravities pipelines

Conclusions

The numerical simulations of the buried pipeline largely supported the findings of the physical experiments of Sumer *et al.* (2004) and Teh *et al.* (2004). The numerical prediction also shows that the PZ3 model is capable to simulate the sinking and the floatation of pipeline before and after the liquefaction due to the decrease in effective stresses in seabed.

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