Proceedings of OMAE'01
20th International Conference on Offshore Mechanics and Arctic Engineering
June 3–8, 2001, Rio de Janeiro, Brazil

OMAE01-PIPE4030

PIPELINE STABILITY ON A MOBILE AND LIQUEFIED SEABED: A DISCUSSION OF MAGNITUDES AND ENGINEERING IMPLICATIONS

J. S. Damgaard
HR Wallingford Ltd.
Wallingford, United Kingdom
T: +44 1491 822465, F: +44 1491 825743, E: jsd@hrwallingford.co.uk

A.C. Palmer
University of Cambridge
Department of Engineering
Cambridge, United Kingdom

ABSTRACT

The magnitudes of pipeline instability processes are assessed in order to discuss the adequacy of traditional pipeline stability methods. The processes considered are: hydrodynamic loads on pipelines, sediment transport and liquefaction. It is found that significant sediment transport will take place long before the pipeline starts to move horizontally.

NOMENCLATURE

A:	orbital excursion [m]
a:	relative compressibility
$C_{D,L,M}$:	force coefficients
C_v:	coefficient of consolidation [m^2/s]
c:	wave celerity [m/s]
D:	diameter of pipeline [m]
d:	diameter of sediment grains [m]
e:	void ratio
$F_{x,z}$:	horizontal and vertical force [N/m]
f_c:	current friction coefficient
f_w:	wave friction coefficient
g:	acceleration of gravity [m/s^2]
k:	wave number = $2\pi/L$ [m^{-1}]
m_v:	compressibility of soil [m^2/N]
nm_w:	compressibility of water [m^2/N]
S:	non-dimensional wave parameter
s:	relative density $=\rho_s/\rho$
U:	depth-averaged current velocity [m/s]
u_0:	orbital velocity [m/s]
W:	submerged weight of pipeline [N/m]
δ:	'pressure decay length' [m]
θ:	Shields parameter
μ:	soil friction coefficient
ν:	kinematic viscosity [m^2/s]
ρ,ρ_s:	density of water and solids[kg/m^3]
ρ':	bulk density of soil [kg/m^3]
σ_v':	vertical effective stress [N/m^2]
τ':	bed shear stress [N/m^2]
τ_{xz}:	cyclic shear stress [N/m^2]
ω:	wave frequency [s^{-1}]

1. INTRODUCTION

Several practically important and interesting problems in underwater engineering involve structures founded on seabeds that may become mobile under the action of waves and currents. In academic terms, they are close to the triple point where fluid mechanics, geotechnics and structural mechanics meet. Their engineering design implications affect pipelines, mattresses and other pipeline stabilisation devices, and bottom-founded underwater structures such as manifolds.

In this paper we investigate the relative magnitudes of the various potentially important processes in order to assess their engineering relevance. The aim is to derive an appropriate mapping of the parameter space for realistic field conditions and discuss seabed engineering implications.

Stability design of submarine pipelines: example 1

The problem is best illustrated by examples, described in more detail in an earlier paper (Palmer, 1996).

The first example is the 12-inch Harriet pipeline in a shallow sea off NW Australia. of up to 0.8 m of carbonate sand overlying calcareous rock. The design wave occurs during rotating storms, and is limited by breaking to 12m, 0.8 of the water depth: there is evidence that this factor may overestimate the maximum wave height, but that point is irrelevant to the subsequent argument.

The 12-inch pipeline was partially embedded in the sand between kp 21.250 and 29.870, and additional anchoring was thought to be necessary to stabilise the line. As part of an assessment of the proposed stabilisation measures, we determined the wave height at which the seabed itself begins to become unstable, applying the method recommended by Sleath (1984). The coarse sand present along the route has a particle size of about 2 mm. Sand of that diameter moves under 4.7 m waves with no current, and under 3.4m waves with 0.5m/s current. Under a 1.12 m/s current, it moves all the time. The smallest particles that will be stable under 12 m waves are 65 mm in diameter, on the boundary between very coarse gravel (32 to 64 mm) and small cobbles.

These calculations therefore confirm that the seabed must be frequently in motion, and not just under extreme design conditions. Different formulae for the onset of seabed motion would obviously give slightly different numbers, but the conclusion is robust, in the sense that no reasonable variations in assumptions about particle density, shape, water depth, wave period, etc. will make any difference to the broad conclusion.

The calculation results are consistent with the observation that there is little marine growth on the sand, whereas in some places there is a lot of marine growth on the pipeline: the explanation is that the surface of the bed is often unstable, so that organisms cannot survive long enough to develop rooting systems.

Since the design wave height is 12 m, it must follow that the seabed must become grossly unstable long before the extreme design conditions for the pipeline are attained. The traditional model almost universally applied to pipeline design is to calculate the forces on the

The water is 15m deep. The seabed consists

pipeline from Morison's equation, and to idealise the interaction between the pipeline and the seabed as governed by a limiting ratio between the horizontal and vertical forces transmitted across the contact. That model is irrelevant: it makes no sense to consider the stability of a stationary pipeline on a stationary seabed. In the assessment that motivated this study, the level of embedment seen is a survey after a large storm cannot be a reliable guide to the degree of embedment at the height of the next large storm: it simply represents a ‘snapshot’ of conditions when sediment motion stopped as the last storm was dying down.

Stability design of submarine pipelines: example 2

The second example is the 40-inch Woodside Petroleum North Rankine gas pipeline on the Australian NW Shelf. The seabed is carbonate sand in the area to be considered here, where the water depth is about 80 m. The pipeline was trenched, and as-built surveys showed the trench to be typically half a diameter deep, roughly symmetrical and with gently sloping sides.

A survey after tropical storms Ilona in December 1988 and Orson in April 1989 showed that the cross-section had become completely asymmetric. On the south-west side, the sand was level with the bottom of the pipe. On the north-east side, the sand was level with the top of the pipe. It was uncertain whether the pipe had moved, but it did not move by more than a few m.

The natural conclusion is that during the storms there has been intense sediment transport towards the south-west. The pipe remained stable and acted as a barrier, until the sand reached the top if the pipe and overtopped it. The situation of the pipe is analogous to that of a groyne on a beach subject to intense longshore sediment transport.

In this example, the pipeline is clearly more stable than the seabed

2. SEDIMENT AND PIPELINE STABILITY UNDER STEADY CURRENTS

In the following we consider the stability of a seabed consisting of non-cohesive sediment and the stability of a pipeline resting on the seabed. In both cases the stability is determined by the relative magnitudes of the agitating hydrodynamic forces and the restoring force due to the submerged weight of the grains and the pipe.

The hydrodynamic force on a sediment grain is usually expressed via the Shields parameter

$$\theta = \frac{\tau'}{\rho(s-1)gd} \qquad (1)$$

where τ' is the shear stress exerted on the grains, ρ is the density of water, $s = \rho_s/\rho$ is the relative density of the solid material, g is acceleration of gravity and d is the grain diameter. The skin friction, τ', is due to both viscous forces and pressure gradients caused by separation.

For a steady current the shear stress can be determined from a quadratic friction law

$$\tau' = \frac{1}{2}\rho f_c U^2 \qquad (2)$$

in which f_c is the current friction coefficient and U is the depth-averaged current velocity.

The sediment becomes mobile when the value of θ exceeds the critical Shields parameter, θ_{cr}, and for fully rough turbulent flows θ_{cr} has a constant value of approximately 0.06 (see e.g. Fredsøe and Deigaard, 1992). Hence sediment becomes mobile at the condition

$$U^2_{cr,s} \leq \theta_{cr} \frac{2g(s-1)d}{f_c} = 0.06 \frac{2g(s-1)d}{f_c} \qquad (3)$$

The forces acting on a pipeline with diameter D in a steady current environment are:

$$F_x = \frac{1}{2}\rho C_D D U|U| \qquad (4a)$$

$$F_z = \frac{1}{2}\rho C_L D U^2 \qquad (4b)$$

where Fx is the horizontal force, Fz is the vertical force, positive upwards, C_D and C_L are the drag and lift coefficients respectively. The contact between the pipeline and the seabed is idealised as governed by Coulomb friction with a coefficient μ (Palmer et al., 1988). The pipeline becomes mobile when the resultant of the drag and lift forces exceed the resisting force due to the submerged weight of the pipeline:

$$F_x \leq \mu (W - F_z) \qquad (5)$$

where μ is the friction coefficient. The submerged weight is given as

$$W = g\rho(s_p - 1)\frac{\pi}{4}D^2 \qquad (6)$$

and s_p is the specific gravity of the pipeline. Combining Equations 4, 5 and 6 we get:

$$U^2_{cr,p} \geq \frac{\pi}{2} \frac{g(s_p - 1)D}{\left(\frac{C_D}{\mu} + C_L\right)} \qquad (7)$$

Hence the ratio of $U_{cr,s}$ to $U_{cr,p}$ is

$$\frac{U^2_{cr,s}}{U^2_{cr,p}} = \frac{4\theta}{\pi} \frac{(s-1)}{(s_p - 1)} \frac{d}{D} \frac{\left(\frac{C_D}{\mu} + C_L\right)}{f_c} \qquad (8)$$

Typical values of the governing parameters are s = 2.65, s_p = 1.6, C_D = 1.2, C_L = 0.8 (e.g. DnV 1996), μ = 0.6 and f_c = 0.005 (see e.g. Soulsby, 1997). Insertion in Equation 8 yields

$$\frac{U_{cr,s}}{U_{cr,p}} \approx 11\sqrt{\frac{d}{D}} \qquad (9)$$

If the relative pipe diameter, D/d, is set to 5000 we get that

$$U_{cr,s} \approx 0.15\, U_{cr,p} \qquad (10)$$

In other words the velocity required to move the pipeline calculated according to the conventional recipe is about one order of magnitude larger than the velocity required to move the sediment. Consequently the seabed will, for typical field conditions, become mobile long before the critical forcing conditions for pipeline stability are reached.

3. STABILITY CONDITIONS UNDER WAVES

Design conditions usually dictated by extreme storm wave forcing. The presence of gravity waves will, in addition to exerting hydrodynamic loads directly on the sediment and the pipeline, influence the shear strength and the bearing capacity of the seabed soil. Under certain conditions the oscillatory seabed pressure will cause liquefaction of the seabed, in which case the shear strength becomes zero. This has important implications for pipeline stability since liquefaction usually is associated with sinking of an unburied pipeline. If the pipeline sinks the resistance to horizontal movements increases significantly.

It turns out that a useful parameter for quantification of the wave forcing is the S parameter defined by Sleath (1994)

$$S = \frac{u_0\,\omega}{g(s-1)} \tag{11}$$

where u_0 is the amplitude of the orbital velocity and ω is the cyclic wave frequency. The S parameter is essentially the ratio of inertia to gravity forces on sand in the seabed. Sleath found that certain aspects of oscillatory flow sediment transport were related to S. In particular, high values of S resulted in a whole layer of sand being mobilised simultaneously. Sleath called this 'plug' formation.

3a. Sediment mobility

The Shields parameter for wave motion can be expressed as

$$\theta = \frac{\frac{1}{2} f_w u_0^2}{g(s-1)\,d} \tag{12}$$

The wave friction coefficient, f_w, is a function of A/d, where A = u_0/ω is the orbital particle excursion. For rough turbulent flows an approximate relation can be based on Soulsby (1997):

$$f_w = 1.39\left(12\frac{A}{d}\right)^{-0.52} \approx \sqrt{\frac{d}{A}} \tag{13}$$

The last expression is accurate to within approximately 10%, which is sufficient for the present 'broad brush' analysis. Combining Equations 11, 12 and 13 we get

$$S = 2\theta\sqrt{\frac{d}{A}} \tag{14}$$

and it follows that the critical value of S for initiation of sediment movement is

$$S_{cr} = 2\theta_{cr}\sqrt{\frac{d}{A}} \tag{15}$$

The curve for S_{cr} as a function of A/d is shown in Figure 2. It has been assumed that θ_{cr}=0.06.

3b. Pipeline stability

The conventional model for marine pipeline design is as follows. The horizontal and vertical wave-induced forces per unit length are calculated from the Morison equations:

$$F_x = \frac{1}{2}\rho C_D D u_0 |u_0| + \frac{\pi}{4}\rho C_M D^2 \frac{\partial u_0}{\partial t} \tag{16a}$$

$$F_z = \frac{1}{2}\rho C_L D u_0^2 \tag{16b}$$

where F_y is positive upwards, d is diameter, ρ is water density., and C_D C_L and C_M are coefficients. As before, the contact between the pipeline and the seabed is idealised as governed by Coulomb friction with a coefficient μ. In the equation for F_x, the first 'drag' term dominates when the Keulegan-Carpenter number A/d is large, and the second 'inertia' term when A/d is small.

Suppose that only the inertia term is important. The submerged weight of the pipeline is $(\pi/4)(s_p-1)\rho g d^2$, where s_p is the relative density of the pipeline. We can define a pipeline Sleath number

$$S = \frac{u_0\,\omega}{g(s_p - 1)} \qquad (17)$$

the regular Sleath number with the seabed particle relative density replaced by the pipeline relative density. We assume that the oscillation of u_0 is monochromatic and sinusoidal. In the inertia-dominated regime, the pipeline becomes unstable when

$$S_p = \frac{\mu}{C_M} \qquad (18)$$

Field measurements of pipeline forces (Wilkinson and Palmer, 1988) showed that for values of A/D up to 1.5 the value of C_M was in the range 2 to 5. We then get (using the typical values stated previously)

$$S_p = 0.1 \text{ to } 0.3 \qquad (19)$$

Laboratory experiments show an increase in C_M with increasing values of A/D for wall mounted pipes (see e.g. Sumer and Fredsøe, 1997). However such a trend could not be detected from the field experiments, and furthermore it is not crucial to our analysis.

Similarly, in the drag-dominated regime the pipeline becomes unstable when

$$S_p = \frac{\pi}{2}\frac{1}{\left(\frac{C_D}{\mu} + C_L\right)}\frac{D}{A} \qquad (20)$$

which for the previously used values of the force coefficients reduces to $S_p = 0.6A/D$.

The pipeline Sleath number is related to the regular Sleath number as

$$\frac{S_p}{S} = \frac{(s-1)}{(s_p - 1)} \qquad (21)$$

For typical values of s=2.65 and s_p = 1.6 the pipeline Sleath number is $S_p/S = 2.8$.

Lines representing Equations (19) and (20) are included in Figure 2. A typical value of D/d = 3000 has been assumed. Therefore the limit between the inertia and the drag regime is around A/d = 10^4.

3c. Wave-induced seabed liquefaction

Seabed liquefaction can occur during certain combinations of hydrodynamic load and soil states. The result of full or partial seabed liquefaction is always a decrease in the bearing capacity of the seabed soil. In the extreme case the bed is fluidised and the pipe will tend to sink if its specific gravity is larger than the specific gravity of the fluidised soil. Simultaneously the soil will entirely lose its shear strength in the geotechnical sense. However, if the sediment particles of the fluidised soil start to move a 'residual' shear strength due to grain - grain interactions will be mobilised (cf. Bagnold's experiments, Bagnold, 1954), but this is a pseudo-viscous effect in which the shear stress is a function of shear rate.

There are two different types of liquefaction dependent on the soil drainage characteristics. For a clean medium to fine sand liquefaction can occur instantaneously during each wave cycle. For loosely deposited silty soils, with lower permeabilities, the cyclic loading can cause a residual (in relation to a wave period) increase in pore pressures that can eventually lead to liquefaction, after a number of wave cycles.

For an infinitely deep seabed the solution for the one-dimensional instantaneous liquefaction problem was given by (for example) Spierenburg (1985):

$$\sigma'_v = (a-1)\,p_0\left(\sin(\omega t) - e^{-z/\delta}\sin(\omega t - \frac{z}{\delta})\right) + g\rho' z \qquad (22)$$

where σ_v' is the effective vertical overburden (we have defined tensile stress as negative in accordance with the soil mechanics sign convention), p_0 is the amplitude of the oscillatory pressure at the seabed surface, ρ' is the submerged bulk density, a is the relative compressibility

$$a = \frac{m_v}{m_v + n m_w} \qquad (23)$$

in which m_v is the volume compressibility of the soil skeleton and nm_w is the compressibility of water. The characteristic length scale, δ, is given as

$$\delta = \sqrt{\frac{2a\,C_v}{\omega}} \tag{24}$$

Note that the solution is similar to the solution for velocity variation through a laminar boundary layer and that δ is analogous to the Stokes length (see e.g. Fredsøe and Deigaard, 1992). Hence the term aC_v has a physical effect similar to that of the kinematic viscosity in the laminar boundary layer.

The present definition of liquefaction is $\sigma_v' = 0$ at a certain depth. Equation (22) is a function of both ωt and z/δ and in order to find the combination of $(\omega t, z/\delta)$ that yields the smallest value of σ_v', Equation (22) is first differentiated with respect to t and we get that

$$(\omega t)_{max} = \arctan\left(\frac{e^{z/\delta} - \cos(z/\delta)}{\sin(z/\delta)}\right) \tag{25}$$

where the subscript 'max' denotes the phase at maximum liquefaction potential. The depth at which σ_v' is closest to zero, and therefore the depth at which liquefaction is most likely to occur, can be found numerically. The value of z_{max} will be a function of the non-dimensional bed pressure amplitude, $p_0/(\gamma'\delta)$ where γ=gρ and the relative compressibility, a. The solution is shown in Figure 1.

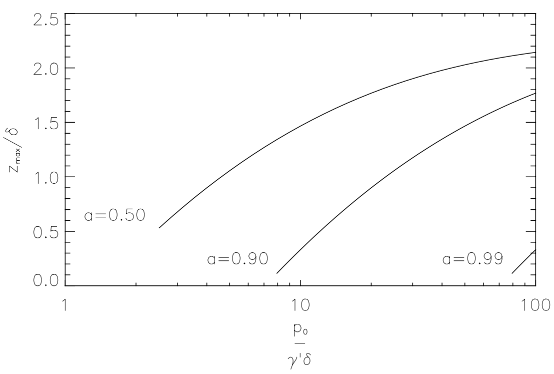

Figure 1 z_{max} as a function of $p_0/(\gamma'\delta)$

It is seen from Figure 1 that for increasing seabed pressure z_{max}/δ tends to constant value in the range of approximately 0.5 to 2 for values of a in the range 0.99 to 0.5. The value of a=0.5 corresponds to $m_v = nm_w$. This is considered to be the smallest realistic value of a. Hence in the 'worst case' (case of maximum liquefaction potential) we have that $z_{max}/\delta \approx 2$ and $(\omega t)_{max} \approx \pi/2$.

Now the conditions of maximum liquefaction potential have been identified. In order for liquefaction to occur under these conditions σ_v' needs to be zero. By inserting the worst case conditions in Equation (22) we get

$$\sigma_v' = 0 = -\frac{1}{2}p_0 + 2g\rho'\delta \tag{26}$$

By applying linear wave theory we get that instantaneous liquefaction at a depth δ will occur if

$$S \geq \frac{4k\delta}{(1+e)} \tag{27}$$

There is a practical lower limit at which the value of δ is too small to be of practical interest. If we require that δ shall be of the same order as the pipeline diameter and if we assume that the void ratio, e, is equal to 2/3, and that $m_v = nm_w$ (i.e. relative large compressibility of the water) then it follows that S = O(10)D/L, where L is the local wave length. D/L will typically be of the order of magnitude $O(10^{-2})$ in which case $S = O(10^{-1})$.

The coefficient of consolidation, C_v, is defined as (Terzaghi et al., 1996)

$$C_v = \frac{K}{g\rho m_v} \tag{28}$$

Experiments and dimensional analysis show that the coefficient of permeability, K, is related to the square of the grain diameter (e.g. Terzaghi et al., 1996)

$$K = 0.0011\frac{g}{\nu}d^2 \tag{29}$$

By combining Equations (24), (26), (28) and (29) a relation for S_{cr} as a function of A/d can be derived:

$$S^2 = 0.018 \frac{1}{(1+e)^2} \frac{g}{c^2} \mathrm{Re} \frac{1}{g\rho m_v} \left(\frac{A}{d} \right)^{-2} \tag{30}$$

in which $\mathrm{Re} = u_0 A/\nu$ is the wave Reynolds number and c is the wave celerity. The first term on the r.h.s of Equation (30) is a measure of the relative compressibility of the soil matrix and the water. The second term, g/c^2, tends to ω for deep water and tends to $\sqrt{g/h}$ for shallow water.

In Equation (8) it has been assumed that a=0.5. However, the volume compressibility of the water, nm_w, will approximately be two orders of magnitudes smaller than m_v at a degree of saturation equal to one, i.e. no air content in the water. However, even a small percentage of air in the water will increase nm_w up to two orders of magnitudes (see e.g. Yamamoto et al., 1978 and Sakai et al., 1992), resulting in the situation $m_v = nm_w$. It is usually not clear whether the air content oberved in the field exists as bubbles or in solution. It is known form acoustics that air in solution has a negligible effect on the speed of sound in water, whereas air as bubbles has a profound effect (Urick, 1983).

A typical value of $g\rho m_v$ is 10^{-4} m^{-1} (Terzaghi et al., 1996). The wave Reynolds number is expected to be in the range of, say, $5 \cdot 10^5$ to 10^7. The cyclic frequency will be in the range $2\pi/10$ s^{-1} to $2\pi/30$ s^{-1}. The envelope of S_{cr} – A/d relationships formed by values of a ≥ 0.5 are shown in Figure 2. The lower bound of the S_{cr} curves correspond to high water compressibility and shallow water conditions. It is seen that S_{cr} for a given value A/d can vary by orders of magnitude, mainly depending on the relative compressibility of soil and water.

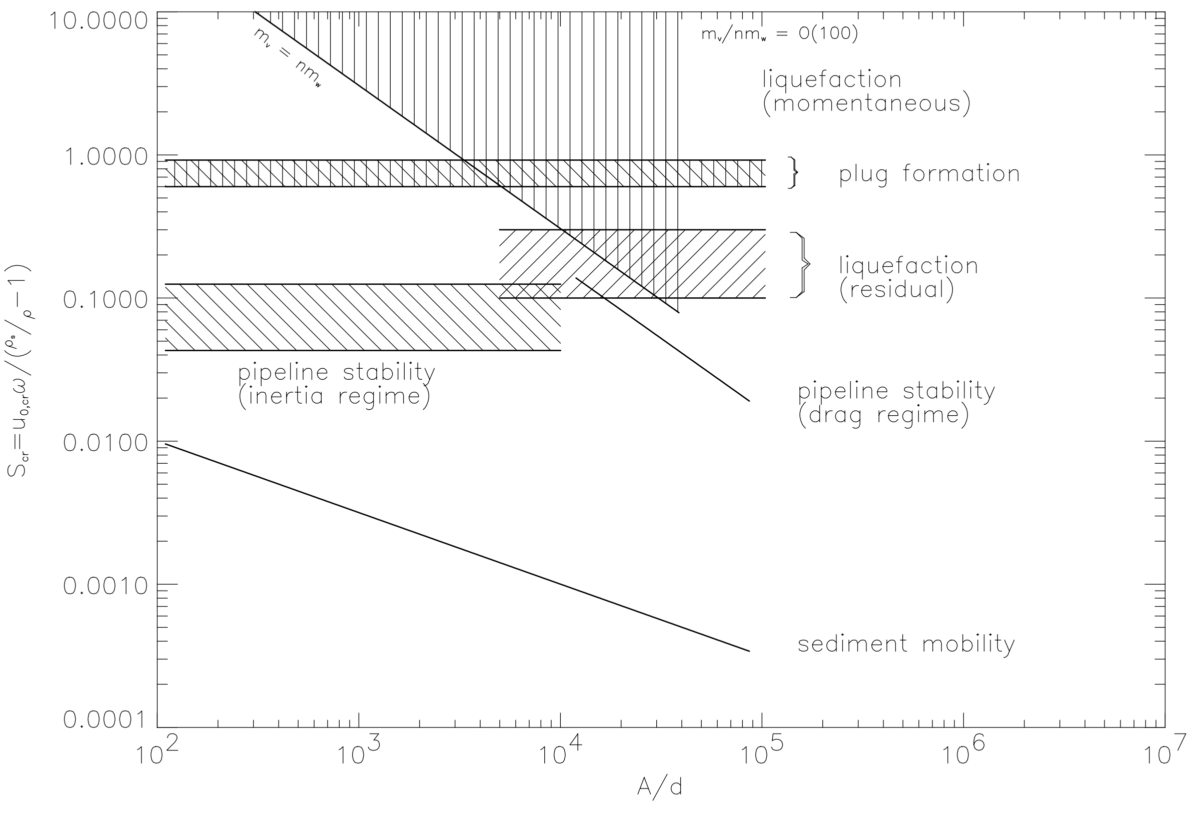

Figure 2: Approximate parameter regions for various seabed stability processes.

The physical processes governing liquefaction due to residual pore pressure build-up are similar to the processes causing earthquake-induced liquefaction. So it is a balance between the rate of pore pressure generation in the soil, which in turn is related to the magnitude of the cyclic shear stress ratio, CSSR = τ_{xz}/σ_v', and the rate of dissipation of pore pressure. The CSSR at the seabed surface is given as (Seed and Rahman, 1978)

$$\text{CSSR}\,(Z=0) = \frac{k}{g\rho'}\,p_0 = S(1+e) \qquad (31)$$

Earthquake induced liquefaction has been observed in the range of CSSR values of 0.1 to 0.3 and since the mechanism of earthquake- and wave-induced residual pore pressure liquefaction are similar, this is also an appropriate range of CSSR values for the present problem. For the one-dimensional situation CSSR decays with depth as exp(-kz).

The expected range of S_{cr} values for initiation of residual liquefaction is shown on Figure 2.

4. PIPELINE ENGINEERING IMPLICATIONS

A number of conclusions can be drawn from the preceding analyses:

1) for all realistic field conditions a sandy seabed will become mobile at forcing levels significantly lower than those required to mobilise a pipeline,

2) marginal pipeline stability can under realistic field conditions be accompanied by seabed liquefaction, which, in turn, is likely to result in sinking of the pipeline, at least for typical values of pipe specific gravity,

3) there are conditions for which the two different types of liquefaction could theoretically coexist.

In terms of subsea pipeline engineering the consequence is that the traditional methods for assessing on-bottom stability are based on an unrealistic physical concept, namely that of a pipeline resting on a hard, or at least immobile, surface. A more realistic situation during extreme conditions is a static pipeline resting on a seabed at which the sediment is mobile. This is also sometimes referred to as a 'live scour'. Consider that the presence of the pipe will cause local amplification of the velocity and the bed shear-stress. It is then clear that the sediment transport near the pipe will be even higher than the transport occurring further away from the pipe. The potentially high sediment transport rates associated with marginal pipe stability will have two contradictory effects: The scour around and under the pipe will result in a certain amount of self-burial and increased stability against horizontal movement. On the other hand the sediment laden near bed flow will effectively increase the 'fluid' density and consequently the hydrodynamic forces on the pipeline.

The fact that seabed liquefaction or failure can occur near the design conditions implies that a pipeline with a specific gravity of, say, 1.4 to 1.6 can sink into the soil before horizontal instability is reached. If that happens the resistance to horizontal movement will increase with increasing embedment (see e.g. Damgaard et al., 1999). The continued oscillatory motion of the pipe itself will interact with the mobile seabed and exacerbate the self-burial process. Depending on sub-surface soil conditions, the pipe may bury entirely.

The typical engineering action taken to achieve on-bottom stability is to design the pipe concrete coating in accordance with the traditional stability criteria. The present findings show that conventional methods may lead to over-design. There is a need to investigate these issues further to reach new recommended guidelines that take account of seabed dynamics

ACKNOWLEDGEMENTS

The funding for this work was provided by the Commision of the European Communities Directorate General for Science, Research and Development as a part of the LIMAS collaborative research programme, under the

Fifth Framework Programme. The authors would like to thank John Sleath, University of Cambridge, for fruitful discussions and Scott Dunn, HR Wallingford, for helping with the numerics. Andrew Palmer is supported by the Jafar Foundation. Some of the first part of this paper appeared first in a paper at the Offshore Pipeline Technology Conference in Amsterdam in 1996.

REFERENCES

Bagnold, R.A. (1954) Experiments on a gravity-free dispersion of large solid spheres in a Newtonian fluid under shear. Proc. Royal Soc. London (a) 225:49-63.

Damgaard, J.S., J. White and M. Worsley (1999) Physical modelling of pull-in loads for subsea jumper intsallation. Proc. OMAE '99, ASME (paper OMAE99/PIPE-5006).

Fredsoe J, Deigaard R (1992). Mechanics of Coastal Sediment Transport. World Scientific Publishing Co. Ptw. Ltd. Singapore.

Palmer, A.C., Steenfelt, J.S., Steensen-Bach, J.O. and Jacobsen, V. (1988) Lateral resistance of marine pipelines on sand, Proceedings, Twentieth Annual Offshore Technology Conference, Houston, 4, 399-408, paper OTC 5853.

Palmer, A.C. (1996) A flaw in the convential approach to stability design of pipelines. Offshore Pipeline Technology Conference, Amsterdam.

Sakai, T., H. Mase, D.T. Cox and Y. Ueda (1992) Field observation of wave-induced pore pressures. Proc. 23rd ICCE, ASCE.

Seed, H.B and M.S. Rahman (1978). Wave-Induced Pore Pressure in Relation to Ocean Floor Stability of Cohesionless Soils. *Marine Geotechnology*, Volume 3, Number 2.

Sleath, J.F.A. (1984) Sea bed mechanics. Wiley.

Sleath, J.F.A. (1994) Bed load transport in oscillatory flow. Sediment transport mechanisms in coastal environment and rivers. Eds. M. Belorgey et al. World Scientific, River Edge, New Jersey.

Soulsby R L (1997). Dynamics of Marine Sands. Thomas Telford, London., 249 pages.

Spierenburg, S.E.J. (1985) Geotechnical aspects of submarine pipelines. Proc. BOSS conference, pp. 797-805.

Sumer, B.M. and J. Fredsøe (1997). 'Hydrodynamics around cylindrical structures', World Scientific Publ., 530 pages.

Terzaghi, K., R.B. Peck and G. Mesri (1996) Soil Mechanics in Engineering Practice. Third Edition. Wiley-Interscience.

Urick, R.J. (1983) Principles of underwater Sound, McGraw-Hill

Wilkinson, R.H. and A.C. Palmer (1988). 'Field measurements of wave forces on pipelines', Proc. 20th Annual OTC, Houston.

Yamamoto, T., H.L. Koning, H. Sellmeijer and E. van Hijum (1978) On the response of a poro-elastic bed to water waves. *Journal Fluid Mechanics*, Vol. 87, Part 1.