A NEW FACILITY FOR RESEARCH ON THE STABILITY OF PIPELINES ON UNSTABLE SEABEDS

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Abstract

There is very strong evidence that in most locations the seabed becomes unstable and mobile before the design conditions for a submarine pipeline are reached. The conventional theory of stability takes the seabed as stable, and is therefore fundamentally flawed.

This is an important issue for pipelines to the north-west of Australia, where violent tropical revolving storms occur and the sea is not particularly deep. The paper describes the construction and commissioning of a facility to investigate this question. Previous work has been carried out in wave flumes and U-tubes. The new facility has a different concept, and has a closed O-tube form, in which the oscillating and steady flows are driven by a propeller. A small version was completed in 2008, and the second and much larger facility was completed in late 2009.

Motivation

A seabed pipeline needs to be stable against the hydrodynamic forces generated by currents and waves, so that it is not carried sideways. In the distant past, that aspect of pipeline design was based on specific gravity (SG, relative density referred to water). Brown [1] describes a 12-inch pipeline in a sandy reef area in Tiger Pass in the Mississippi Delta, designed to a SG of 1.3, and trenched to 2 m cover in 2 m of water. Hurricane Flora came through in 1957, and the line rose until some 60 mm were exposed above the mudline. As a result of that experience, it was decided that the SG in that and similar areas ought to be 1.6, and that value was adopted by Tenneco.

In about 1970, it came to be thought that design based on SG was unsound, and that a completely different model ought to be adopted. Currents are measured and extrapolated to a design maximum. Waves are measured or calculated, and then extrapolated to a design wave (or a design seastate). A wave theory determines the wave-induced velocity just above the seabed. The Morison-O'Brien equations determine the hydrodynamic forces that the most severe combination of wave and current induces. The contact between the pipeline and the seabed is idealised as one governed by a Coulomb friction model, so there is some limiting ratio between the horizontal and vertical forces *S* and *R* between the seabed and the pipeline.

That model has been almost universally applied in design, and is applied in recommended practices such as DNV RP F109 [2]. There has been much research, both on the hydrodynamics and on the geotechnics, and some leading references are summarised in [3]. Some refinements have been added, to take account of factors such as partial embedment, which both increases the lateral resistance and reduces the hydrodynamic force, and to allow the pipeline to move, as long as it does not move too far. The refinements modify the model, but do not alter it in any essential way.

There is strong and convincing evidence that the accepted model is fundamentally wrong, because it assumes that the seabed itself is stable and immovable under design conditions. It turns out that under design conditions a sand or silt seabed is almost invariably grossly unstable and mobile. The accepted model may however apply to the unusual cases of pipelines that rest directly on solid rock or stiff clay.

The evidence is discussed at more length in other papers [4-7]. One example is the Woodside operated 1TL 40-inch (1016 mm) gas pipeline from the North Rankin field off the north-west coast of Australia. The line was constructed in 1981-82, and trenched to about 1 diameter depth in a carbonate sand seabed. At the relevant section, the water depth is 80 m. Figure 1(a) is the as-trenched cross-section. Two tropical revolving storms (hurricanes) ensued. Storm Ilona [8] in December 1988 had an estimated maximum significant wave height 8.3 m, significant period 11.5 s, and induced a 1.3 m/s maximum velocity at the seabed (counting all the components together). Storm Orson [9,10] followed four months later: it was close to the 100-year event on the pipeline route, and instrumentation on the North Rankin A platform recorded winds in excess of 249 km/hour, a central pressure of 905 hPa, and waves with a maximum height of 19 m [10]. The maximum significant wave height was 11.5 m, the significant period 13.5 s, and the estimated maximum velocity at the seabed was 2.1 m/s (counting all the components).

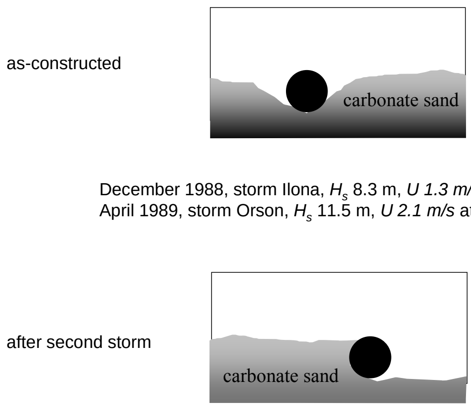

Figure 1 (a) as-constructed (b) after storm Orson

Figure 1(b) is the cross-section revealed by a survey after the second storm. It is incontrovertible that the seabed had moved, and that was confirmed by secondary evidence.

The conclusion can be generalised. Figure 2 plots the wave height at which a cohesionless seabed begins to move, against the seabed particle diameter, for two water depths, taking the particle specific gravity as 2.65 and the wave period as $\sqrt{(10H)}$ s, where H is the wave height in m. It can be seen that the threshold wave height for the start of particle movement is much smaller than the design wave height in most locations, unless the water is very deep or the particles are unusually large.

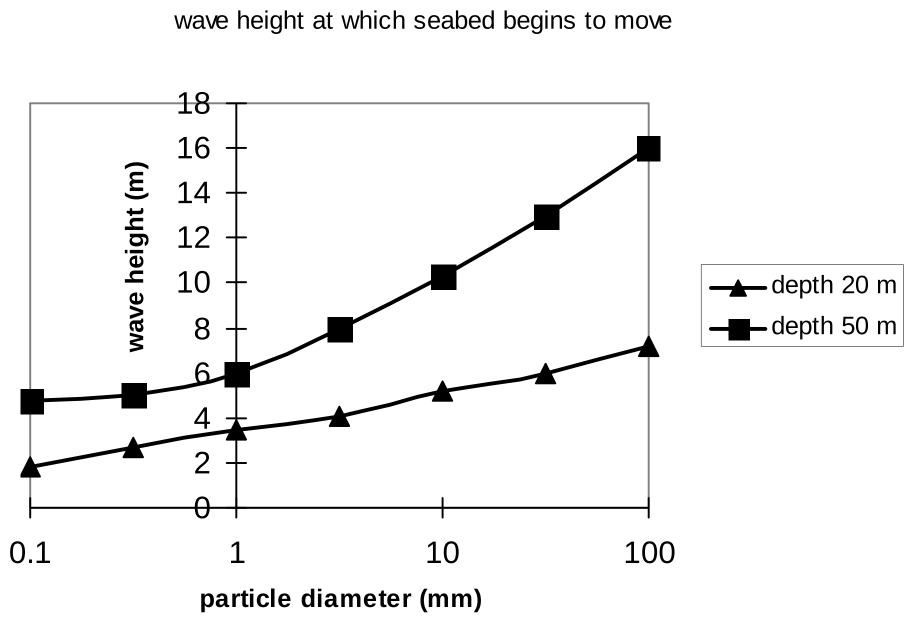

Figure 2

Those conclusions apply to a bare seabed. The presence of a pipeline alters the flow field and complicates the situation. A wake forms on the downstream side, and there the velocities are reduced. The pressure difference between the upstream and downstream sides drives water through a permeable seabed, and on the downstream side water is flowing out of the soil, which may lead to lee-side erosion. The flow-induced changes of effective stress interact with the stresses produced by the loads on the soil induced by the hydrodynamic forces on the pipe, and tend to deform the soil and add to embedment, and piping may occur. Not all these effects have an adverse effect on stability, and some may help the pipeline to embed and become less unstable.

The interactions that govern pipeline on-bottom stability are summarised in Figure 3. There is significant interaction between the pipe (P), the fluid (F) and soil processes (S):

1. Hydrodynamic loading: waves and currents exert cyclic lift and drag forces on the pipeline (F-P).

2. Scour and piping: waves and current cause scour around the pipeline and piping beneath it (F-S-P).
3. Direct liquefaction: waves and current create excess pore pressure in the free field (F-S).
4. Indirect liquefaction: the cyclic loading on the pipeline creates excess pore pressure in the soil near the pipe (P-S).
5. Soil failure: if the load exerted on the soil by the pipeline exceeds the vertical-horizontal (V-H) bearing capacity, soil failure will occur and the pipeline may settle or move laterally. The V-H bearing capacity depends on the excess pore pressure field near the pipe (P-S).

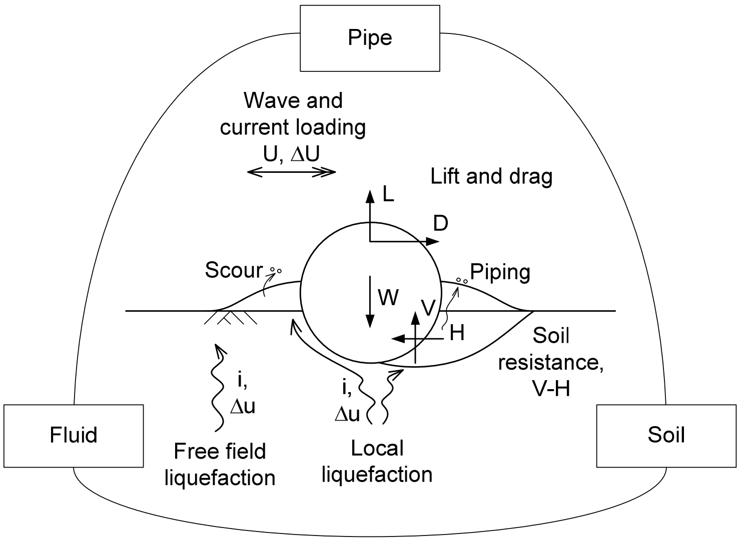

Figure 3

Different combinations of these processes can have a positive or negative effect on the tendency for the pipe to lose stability and displace laterally. If the seabed liquefies locally but the drag load is small, then the pipe will tend to sink, hindering lateral movement and increasing stability. If the seabed liquefies or becomes mobile both locally and in the free field, then the soil resistance will be lower during a storm than in still water, which may cause the pipeline to become unstable.

It may not be correct to treat interactions between two of the elements as independent of the other element when considering the overall stability of the pipe. For example, if the presence of the soil is neglected and the seabed is taken as rigid and impermeable, then the force coefficients linking fluid velocity to lift and drag may differ from the true case, because the permeability of the seabed affects the pressure distribution. Similarly, if the movement of the fluid is neglected in an assessment of the pipe-soil resistance force (or ‘friction factor’), then the possible influence of liquefaction is overlooked.

Of course, it is perfectly valid to perform experiments or develop analyses that ignore the third element. For example, our understanding of free-field liquefaction can be enhanced by performing tests without a pipe present. Model tests of a pipe held in a flow above a rigid floor represent the limiting case of a (rigid) impermeable seabed and can provide the corresponding hydrodynamic force coefficients. However, it must be recognized that this behaviour will be modified when the third element is introduced.

This discussion highlights the need for a multi-disciplinary approach to the assessment of pipeline stability. The hydraulic and geotechnical interactions (i.e. the fluid-pipe and pipe-soil aspects) cannot be performed in isolation, because each has an influence on the other.

Previous research

It is easy to point out the fundamental flaws in the traditional model, but not so easy to know how to replace it with something more rational.

DNV RP F109 reflects this difficulty: it points out that seabed instability can be significant, and talks briefly about the significance of Sleath number, but does not give the designer any definite idea of how to proceed. One might suspect that many designers simply ignore that part of the RP.

Some research was carried out as part of the LIMAS (Liquefaction around Marine Structures) project, supported by the EU and led by Sumer at the Technical University of Denmark [4]. Teh [6,7] placed a pipeline on a bed of carbonate silt in a wave flume, and measured the vertical movement of the pipe, together with changes in pore pressure and in silt density. He found that the behaviour of the pipe was primarily controlled by its SG. Figure 4 plots the initial and final levels of the pipe as a function of its SG. Light pipes remained at the same level or floated upward, and heavy pipes sank into the mobile silt, particularly if their SG exceeded 1.6. It is tempting to suggest a link between that number and the design value of 1.6, arrived at four decades earlier and by a completely different argument. Carbonate silt is fine-grained (D_{50} 33μm) and has a low permeability, and Teh's measurements 80 mm below the mudline showed that substantial pore-pressure buildup occurred, in contrast to Zala Flores' experiments on sand [11], where there were changes in pore pressure in each loading cycle but no longterm buildup.

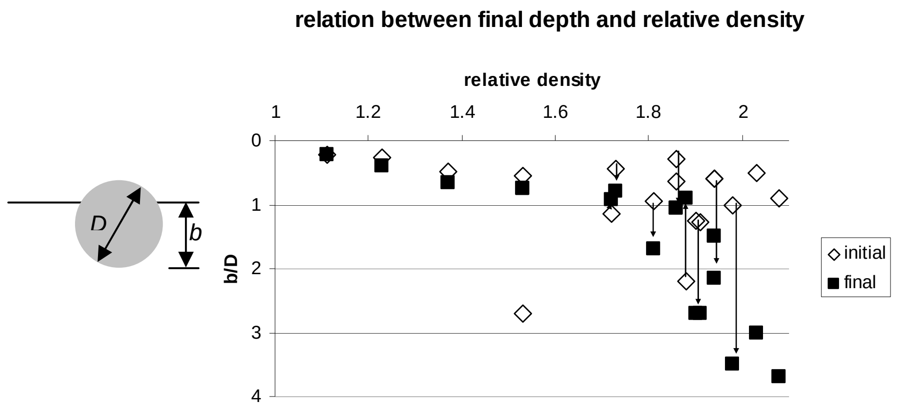

Figure 4

This project

Stability is an important issue. If the pipeline is underdesigned, it is at risk of large movements that might lead to buckling and threaten its integrity. If the pipeline is overdesigned, money may be spent unnecessarily, on measures such as extended rock dumping that might lead to additional problems such as upheaval buckling.

The LIMAS work is an example of recent research that has tackled pipeline stability by adopting an approach that is fundamentally different to the current DNV RP F109. A tentative basis for new types of design assessment was guided by new experimental data.

A new project has been initiated at the University of Western Australia (UWA), with support from Woodside and the Australian Research Council. The aim is to explore alternatives to the current DNV RP F109 approach, guided by new experimental results. The academics involved in the project include both hydraulic and geotechnical engineers, all with experience in pipeline design, ensuring a balance of technical expertise. The initial stage of the project has been the development of a novel experimental environment that replicates seabed conditions better than the open-channel flumes used in previous studies.

An open-surface wave flume is not ideal for experiments to throw light on the interaction between the water, the seabed and a pipeline. We are interested in combinations of steady current and large wave-induced velocities, but they imply large waves, and large waves in a flume tend to be steep and asymmetric, and may be close to breaking. A better option is a closed flow loop in which water is driven backwards and forwards by a propeller. A related closed-loop design was constructed many years ago by HR in the UK [12], but there the water was driven by a piston.

The O-tube flumes at UWA

UWA constructed the first loop of this kind in 2007, and calls it the ‘mini O-tube’. Figure 5 is a photograph of the mini O-tube, and Table 1 lists the key parameters. This pilot development showed that an enclosed propeller could produce a reliable superposition of steady and oscillating flow, with stable behaviour through the test section, with feedback being used from the propeller to impose the desired fluid flow characteristics. Experience earned with the operation of the mini O-tube was applied to the design of a much larger version (‘large O-tube’) in 2009, and Table 1 lists its parameters.

These flumes allow the interaction processes between scour, liquefaction and pipeline loading to be investigated under combined wave and current conditions with a relatively large model scale, in a way that has not been done before.

	mini O-tube	large O-tube
length over all (m)	5.0	24.0
breadth over all (m)	1.9	7.8
height of working section (m)	0.3	1.4
breadth of working section (m)	0.2	1.0
length of working section (m)	1.8	17.4
maximum steady velocity (m/s)	1.8	5.0
maximum oscillatory velocity (m/s)	1.0	3.0
minimum period at maximum velocity (s)	3.0	5.0
rated power of drive motor (kW)	4.5	580

Table 1

Figure 5 Mini O-tube

Figure 6 below is a photograph of the large O-tube. Fabrication and assembly were completed in November 2009. At the time of writing, in mid-January 2010, the initial testing, calibration and measurement of turbulence levels are largely complete. The maximum steady current that has been reached so far is 5 m/s, and the maximum oscillatory velocity reached is 3 m/s with a 10 s period.

The first pipe to be tested will be 200 mm OD, but other diameters will be included in the test programme. One of the reasons for this is the exploration of scaling. The linkage and frame that will support the pipeline within the working section are being fabricated. It has been found in other experiments [6] that if a section of pipe in a flume is not constrained, it tends to rotate in plan, which is not possible for a real pipeline. The pipe is supported in an actuator that is designed to prevent that kind of motion, and it can actively control the two-dimensional movement of the pipe, or – via a feedback control system – can leave these motions unconstrained.

Figure 6 Large O-tube

The presence of the pipe partially obstructs the flow in the flume. CFD calculations have investigated this point. Figure 7 below plots the calculated hydrodynamic coefficients as a function of the ratio between the flume depth and the pipe diameter, and indicates that blockage effect is small if the ratio is 8.

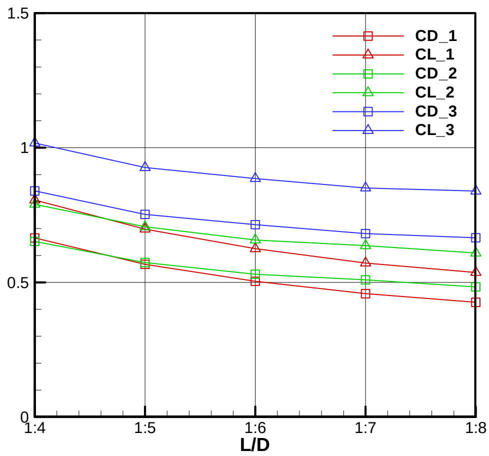

Figure 7
The force coefficients at different D/L values. C_{D_1} and C_{L_1} were normalized by the average of inlet velocity, C_{D_2} and C_{L_2} were normalized by the velocity at the level of the top of the pipe, C_{D_3} and C_{L_3} were normalized by the velocity at the level of the centre of the pipe, for the case where 20% of the pipe diameter is buried in the sand.

The first tests will use seabed soil recovered from dredging in the Pluto field on the North-west Shelf of Western Australia. That soil includes a fine clay fraction, which settles out last and forms a very soft surface layer. The same thing may happen in nature, where after a storm the fine fraction settles out slowly. The fine material also clouds the water in the flume, and obstructs laser measurements of velocity. It may be decided to remove the fine fraction.

Scaling

A flume that could test a large-diameter pipeline at full scale would have to be enormous. The 200 mm pipe tested in the large O-tube has to be thought of as a model. Models can be misleading unless the scaling is thought about carefully [13]. Scaling is discussed in the Appendix to this paper.

Conclusion

This facility will make it practicable to examine the actual behaviour of pipelines on deformable and mobile seabeds, and in due course to arrive at a design method.

Acknowledgement

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Appendix: scaling

The interaction between a pipeline and the seabed involves a number of phenomena, and it is not yet clear which of them are the more important.

One factor is scour and piping. The seabed tends to be destablised by flow through the soil under the pipeline. In the classical analysis, piping occurs when the pressure gradient on the downstream side just balances the effective stress gradient. The maximum pressure difference between the upstream and downstream sides of the pipeline is proportional to ρU^2, where ρ is water density and U is water velocity. If the length of the flow path is a the seepage pressure gradient in the seabed is proportional to $\rho U^2/a$. If there is no flow, the effective stress gradient is $(\rho_s-\rho)g$, where in addition ρ_s is the total weight per unit volume of the sediment. Accordingly

$$\frac{\text{seepage pressure gradient}}{\text{effective stress gradient}} = \frac{\rho}{\rho_s - \rho}\frac{U^2}{ga} \qquad (1)$$

and that ratio is preserved if Froude number U^2/ga is the same in the model as in the prototype, assuming that the model and prototype soils are the same.

Pipe SG is important, and therefore the ratio between the pipeline mean density and the soil mean density ought to be kept the same. Sleath number $\frac{\rho U\omega}{(\rho_s - \rho)g}$ (where in addition ω is the radian frequency of the oscillatory velocity in the water) is important, because it is a measure of the effect on the soil of the pressure gradient in the water (in turn a consequence of the acceleration). The maximum velocity is of course out of phase with the maximum acceleration, and the parameter that represents the effect of the shear stress generated by the velocity is the Shields parameter (densimetric Froude number) $\frac{\rho v^2}{(\rho_s - \rho)gd}$, where v is the friction velocity and d the particle diameter.

Another factor is the possibility of pore pressure increase due to alternating shear stresses. Liquefaction due to earthquake-induced pore pressure build-has been much studied (see, for example, Been and Jefferies [14]), but is rather different, because an earthquake lasts only a minute or so, whereas wave-induced build-up occurs takes hours, and there is time for the increase pore pressure to diffuse away, partially or completely. Pore-pressure diffusion is governed by the diffusion equation

$$\frac{1}{c_{vc}}\frac{\partial u}{\partial t} = \nabla^2 u \qquad (2)$$

where u is excess pore pressure, t is time, and c_{vc} is coefficient of volume consolidation, a combination of the compression modulus of the soil skeleton, the permeability k of the soil, and the viscosity μ of the pore fluid. It follows that if soil and the pore fluid are the same in a model as in the prototype, and the length scale is N_L, the time scale in the model is N_L^2 times smaller than in the prototype. This scaling is often adopted in geotechnical centrifuge modelling, where it is usually thought desirable that the soil should be the same as in the prototype, though the prototype pore water is sometimes replaced by a more viscous fluid to lengthen the timescale. It would not be practicable to fill the whole O-tube with a viscous fluid, but it might be possible to replace the pore fluid in the soil. An alternative is to replace the soil with another soil with a much smaller particle size, taking advantage of the fact that permeability is approximately proportional to (particle diameter d)^2.

It will not be possible to maintain similarity for all the phenomena that might be relevant. Yet another parameter that could be significant is the particle settling velocity, which for small particles is proportional to $(\rho_s-\rho)gd^2/\mu$. If we want to maintain the ratio between the settling velocity and the maximum oscillatory velocity U, then d ought to scale with $\sqrt{U}$, whereas Froude scaling implies that that other lengths scale with U^2.

It must be recognised that these scaling relationships are approximate, and that although scaling of pore pressure diffusion has been considered, pore pressure *generation* has not. These potential sources of discrepancy between the scaled-up model response and the prototype will be considered in the subsequent analysis, and in the stability assessments that will follow.