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On-Bottom Stability Analysis for a Pipeline on a Mobile Seabed

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ABSTRACT
This paper describes the work performed to assess the stability of an existing pipeline off the North Western coast of Western Australia. Originally, the pipeline was post-trenched (for stability), by ploughing. The plough formed an open v-shaped trench below the pipeline. Several years after the pipeline was installed, a severe cyclone caused significant changes to the seabed along the pipeline. The v-shaped ploughed trench is generally no longer visible with the trench backfilled in places and the pipeline exposed in others. In addition to this, the embedment profile either side of the pipeline was often found to be assymetrical, with higher embedment on one side of the pipeline relative to the other.

Current on-bottom stability guidelines and recommended practices do not account for the effects of assymetrical embedment and seabed mobility (Ref. 5). These two factors are believed to contribute significantly to the overall stability of the pipeline. Consequently, it was decided to use physical model testing to obtain a better understanding of hydrodynamic loading and changes in embedment over the duration of a design storm. The physical model testing was performed for various pipeline embedment profiles. The results of these 2D physical model tests were then applied to the 3D FEA on-bottom stability software, CORUS 3D.

This paper provides a detailed description of the physical model testing program performed as part of the pipeline stability assessment. The method used to combine the results of the physical model testing and apply them to the 3D FEA on-bottom stability software is also presented.

1 INTRODUCTION
This paper presents the work performed to assess the stability of an existing subsea pipeline on a seabed consisting primarily of carbonate sand. The pipeline is located in a region where severe cyclones are common. These cyclonic conditions can generate significant on-bottom flow velocities which in turn induce large hydrodynamic loads on the pipeline. Following the passing of one particular cyclone, the seabed in the vicinity of the pipeline was found to have changed significantly. Consequently, a review of the pipeline's stability was undertaken. This was done to ensure that the current seabed profile would provide adequate resistance to pipeline lateral displacement in the case of a future cyclone.

An ROV survey of the pipeline, undertaken in 2005, was used to perform a pipeline embedment review. This review found that the pipeline was partly embedded with embedment levels varying regularly along the length of the pipeline route. A schematic demonstrating this variation in embedment is presented in Figure 1.

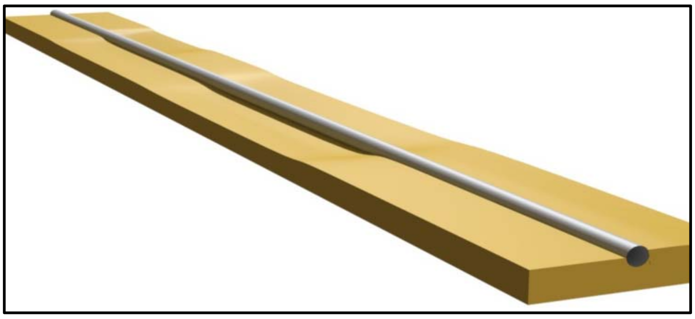

Figure 1: Variation in Embedment Along the Pipeline

It also found that the embedment profiles were generally asymmetric, with greater embedment on one side of the pipeline than the other. Examples of embedment profiles observed along the pipeline route are present in Figure 2.

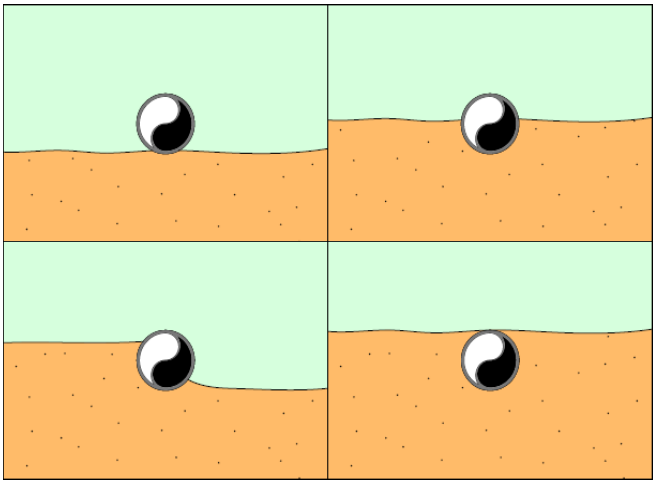

Figure 2: Pipeline Embedment Profile Examples

The subsequent area of study relates to the pipeline stability in its partly embedded state. As an initial step, a 2D stability assessment of the pipeline was performed using the Absolute Stability Methodology outlined in DNV-RP-F109 (Ref. 1).
This methodology was modified to account for the effects of asymmetrical embedments on hydrodynamic loading and soil resistance.
It was recognised however, that this 2D stability analysis could only be used as a preliminary assessment of pipeline stability. The key limitation associated with current stability analysis approaches is the assumption that on-bottom flow velocities do not affect the seabed (except indirectly through the hydrodynamic loads on the pipeline). This limitation is depicted in Figure 3. It is also discussed in detail in Ref. 4 and Ref. 5.

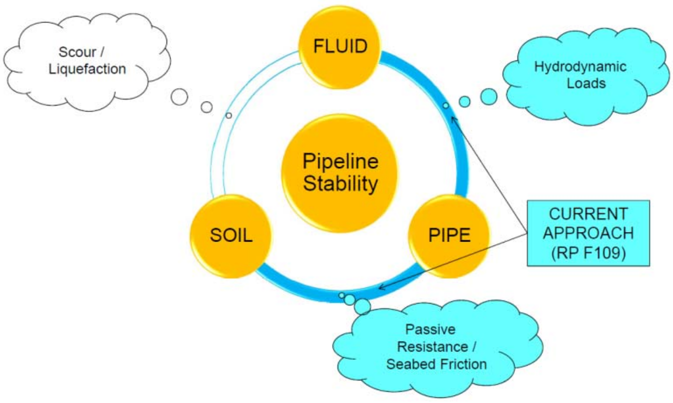

Figure 3: Limitation in Current Stability Design Approach

In addition to the assumption that the seabed is stable, the 2D stability assessment also fails to account for the following:

- The highly embedded sections of the pipeline route. These highly embedded sections are expected to assist in stabilising the pipeline sections with lower embedment levels.
- The 3-dimensionality of wave loading in a typical sea state. This leads to opposing drag forces acting on adjacent sections of the pipeline. These counteracting loads in combination with the pipeline's structural stiffness are expected to significantly reduce pipeline displacements.

In an attempt to address the above mentioned limitations, a new stability analysis method was proposed. This method involved the combination of physical model testing and FEA modelling (Ref. 4). The physical model testing would be used to model the effects of local scour in two dimensions over the duration of a design storm. This testing would be performed for a number of different initial embedment profiles such that all of the embedment configurations observed in the 2005 survey data could be assessed. The results of this 2D physical model testing would then be fed into a 3D FEA model which could account for the stabilising effects of adjacent pipeline sections with higher initial embedment, as well as the effects of three-dimensional wave loading. This paper presents the processes followed in the application of this new method.
The regions that were identified as being the least stable from the 2D stability assessment have been used as the basis for this new stability analysis approach.

2 DATA ACQUISITION

The following section outlines the data required to perform the stability analysis.

2.1. Pipeline Data

The following pipeline properties were required to accurately model the pipeline's stability in a physical model testing facility:

- Pipeline Outside Diameter (OD)
- Pipeline Specific Gravity (SG)
- Pipeline Roughness (k_s)

The prototype pipeline outside diameter and the model pipe diameter were required to define the length scale ($N_L = OD_{prototype}/OD_{model}$). The length scale was then used to accurately scale down the time and flow velocity of the design storm to be applied in the physical model tests. This length scale was also required to scale up the hydrodynamic loads recorded in the physical model test to prototype scale. (See Section 3.5 for more details on scaling)
The pipeline SG was required to accurately replicate the pipeline's self-weight during the physical model testing.
It was also desirable to maintain geometric similitude with regards to pipeline roughness. The model pipe was sand blasted to achieve the required roughness. This allowed the flow regime around the model pipe to remain the same as that observed in the prototype.

2.2. Metocean Conditions

Metocean data relevant to the pipeline section was collated and then used to specify a wave energy spectrum This energy spectrum was in turn applied to generate a number of irregular on bottom velocity time series. The irregular velocity time series combines an irregular oscillatory flow component caused by waves with a steady current component cause by storm surge and tidal forces. The duration of the design storm peak was specified at three hours (as recommended by DNV-RP-F109 (Ref. 1)), with an equally long storm ramp up and ramp down.
Seven realisations of complete on-bottom wave and current velocity data in the time domain were generated for the 100 year RP irregular storm condition. An example of an irregular velocity time series is presented in Figure 4.

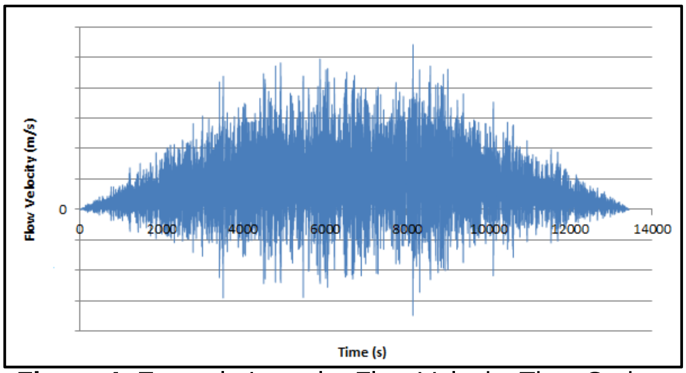

Figure 4: Example Irregular Flow Velocity Time Series

2.3. Geotechnical Data

Soil that was representative of the conditions in the North West Shelf of Australia was required for the physical model testing. Specifically, the soil needed to be representative of the conditions along what was considered to be the most unstable section of the pipeline route. Consequently, soil was sampled from the seabed close to this section of the pipeline.

This soil sample was then assessed to determine its suitability for use in the physical model tests. Ideally, the soil particle size would have been scaled down to match the length scale (N_L). This would have reduced or eliminated any scour distortions due to scaling. However, this is rarely achievable due to various scaling difficulties associated with movable bed model tests.

The following factors will vary when soil properties are modified:

- Soil resistance to lateral pipeline displacement
- Rate of local scour and free field scour
- Onset of local scour and free field scour
- Excess pore pressure build up.

3 PHYSICAL MODEL TESTING

3.1. O-Tube Testing Facility

Physical model testing has been undertaken in a facility called the Large O-Tube (LOT), owned and operated by the University of Western Australia. The LOT comprises of a closed loop channel of water driven by an axial flow pump, with a 1 m wide by 1.4 m high test section in which model testing is conducted.

The LOT is capable of combining steady and oscillatory flow to produce realistic scaled on-bottom current conditions. Images of the LOT facility are presented in Figures 5 and 6.

Figure 5: O-Tube Testing Facility at UWA

Figure 6: O-Tube Test Section at UWA

The LOT testing facility was selected over other testing facilities including wave flumes and U-tubes, for the following reasons:

- Large relative scale (capable of testing large gas pipelines at scales of 1:6 and smaller pipelines at scales of 1:1) Standard physical model testing facilities involving hydraulic flow are typically only capable of running tests at scales of 1:25.
- Capable of combining both steady flow and oscillatory flow. It can accurately simulate on-bottom conditions representative of irregular sea states.
- High quality recording devices and actuator system. The model pipe can be operated in load control or displacement control. During testing the model pipe is operated in load control mode, meaning that it can displace when subjected to sufficient hydrodynamic loading. This leads to realistic breakout scenarios.

The large relative scale allowed for significant reductions in scaling distortions associated with hydrodynamic loading, onset of local scour, rate of local scour and onset of free field scour.

3.2. Physical Model Testing Scope

A number of tests were performed for various symmetric and assymetric initial embedment profiles as presented in Figure 7. Tests were also repeated using different 100 year RP storm realisations to assess the pipeline's sensitivity to variations in on-bottom velocity time series.

The testing has been performed for an appropriate representation of a 100 year RP irregular wave-induced and steady state current time series. Seven realisations of complete on-bottom wave and current velocity time series were generated for the 100 year RP irregular storm. The storm realisation with the largest peak flow velocity was selected as the base case flow velocity time series for use in the physical model testing program.

The following data was measured from each test for use in the 3D FEA stability model:

- Lateral and vertical hydrodynamic loads as a function of time.
- Change in embedment either side of the pipeline as a function of time.

Pipeline displacement was also monitored for all of the physical model tests.

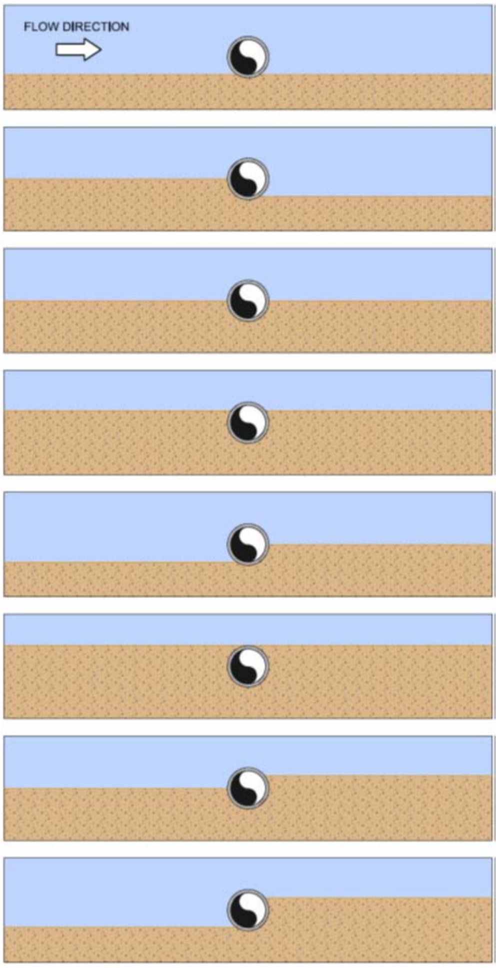

Figure 7: Initial Embedment Profiles Tested in O-Tube

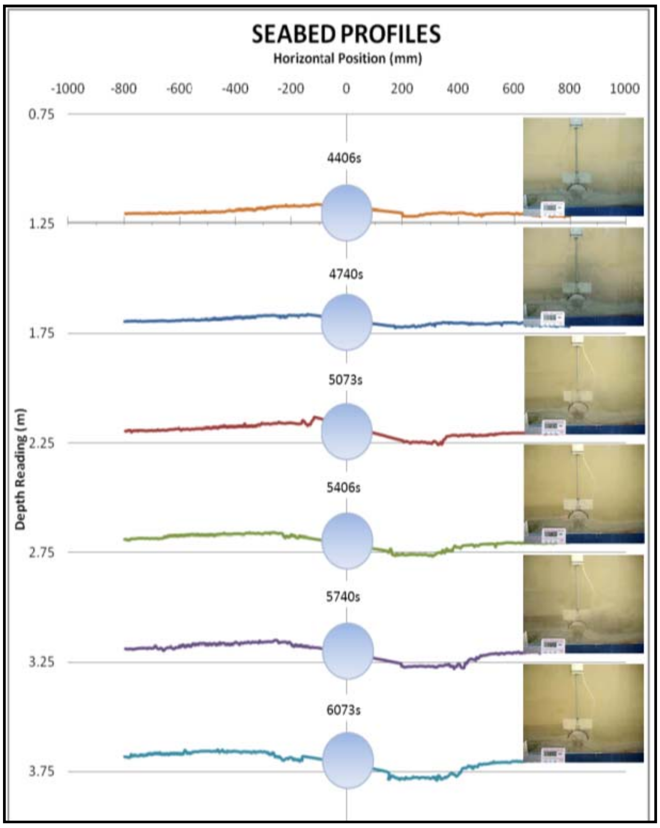

Figure 8: Echo Sounder Profiles

3.4. Model Pipe Embedment

The embedment profile either side of the model pipe was monitored using an echo sounder positioned above the O-tube test section. The echo sounder would pass over the model pipe at regular intervals to produce accurate records of embedment adjacent to the pipeline as a function of time. An example set of echo sounder profiles is presented in Figure 8. These profiles demonstrate the change in embedment level with time.

3.3. Hydrodynamic Loading

Pressure transducers were placed around the circumference of the model pipe, as shown in Figure 9. The pressure distribution around the model pipe could be obtained from the pressure readings provided by these pressure transducers. This data was then used to calculate lateral (drag and inertia) and vertical (lift) forces acting on the model pipe as a function of time. These hydrodynamic forces were required as inputs into the 3D FEA stability model. Graphs depicting hydrodynamic loads recorded in the LOT during a storm ramp up are presented in Figure 10.

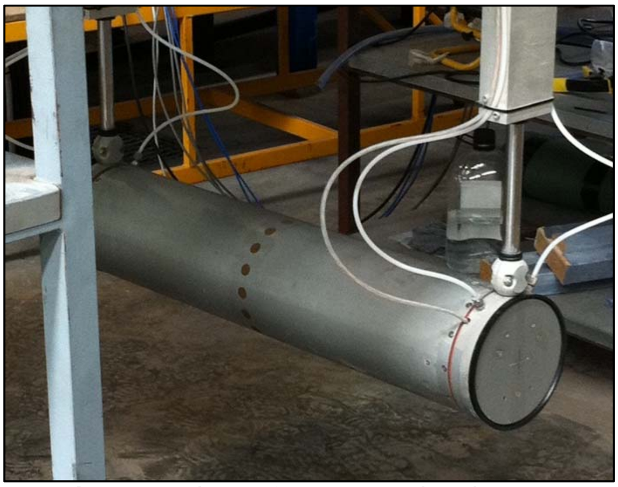

Figure 9: Photo of UWA's Model Pipe

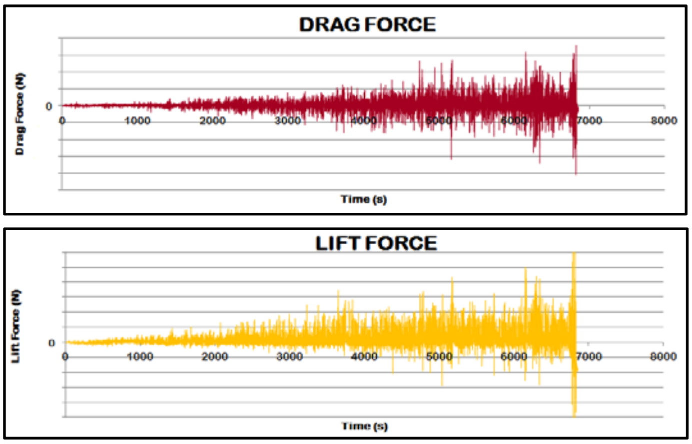

Figure 10: Hydrodynamic Loads Recorded in LOT

3.5. Scaling

The physical model testing was performed at a length scale (N_L) of 1:5.8. Similitude in the Froude criterion was maintained using the scaling ratios presented in Table 1. This ensures that the ratio of inertial forces to gravitational forces remains the same in both the prototype and the model (Ref. 2). As mentioned previously, the soil particle size could not be scaled down to the required length scale due to various scaling difficulties associated with movable bed model tests.

Table 1. Physical Model Testing Scaling Summary

Description	*Symbol*	*Units*	*Scaling*
Pipeline Diameter	D	m	N_L
Soil Particle Size	d	m	1
Time	t	s	$N_L^{1/2}$
Bottom Shear Velocity	U	m/s	$N_L^{1/2}$
Pipeline Roughness	k_s	m	N_L
Fluid Density	ρ_w	kg/m^3	1
Soil Density	ρ_s	kg/m^3	1
Fluid Kinematic Viscosity	ν	m^2/s	1

4 SOIL RESISTANCE

The soil resistance model typically applied to the dynamic stability analyses of pipelines is based on the combination of friction between the pipeline and seabed, and soil passive resistance (resistance caused by pipeline embedment). The friction component is generally based on a coulomb friction approach, with a pre-defined friction coefficient which is dependent on soil type.

The passive soil resistance component of dynamic stability software is often based on the soil resistance model developed by Verley & Sotberg (Ref. 3). This Verley & Sotberg pipe-soil interaction model predicts the development of pipeline embedment into the soil and the associated soil resistance as the pipeline displaces laterally (Ref. 3). The model defines a resistance force that is dependent on:

- the lateral displacement of the pipeline from its original position (y)
- the normal contact force between the pipeline and seabed (F_{ci})
- the embedment level of the pipeline section (z).

The ensuing force-displacement curve is presented in Figure 11, where (F_r) and (y) are the passive resistance and lateral displacement respectively. The F_{r1}, F_{r2} and F_{r3} values are determined using equations that are dependent on:

- Pipeline diameter
- Pipeline embedment
- Pipeline–soil contact force (normal to seabed)
- Submerged unit soil weight.

These values correspond to predefined mobilisation distances (y_1, y_2 and y_3). The F_{r1} value indicates the peak elastic passive resistance, F_{r2} is the maximum passive resistance, also referred to as the breakout force, and F_{r3} is the residual force following breakout.

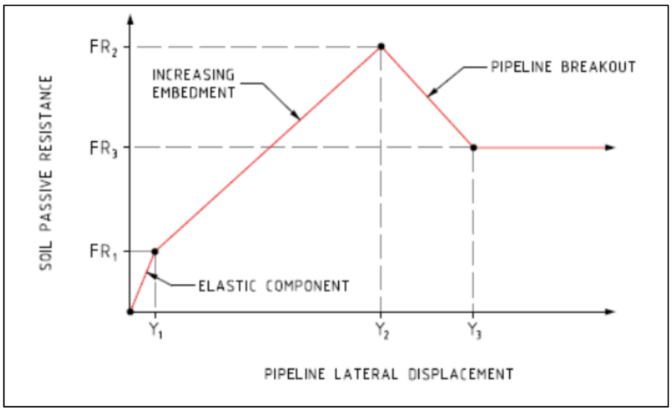

Figure 11: Verley Force-Displacement Curve (Ref. 3)

4.2. LOT Pull Out Tests

To develop a soil lateral resistance model that was representative of the soil conditions present along the pipeline route, six pull out tests were performed at varying levels of embedment (0%, 10%, 25%, 40%, 50% and 60% etc.) using the LOT. All tests were performed using a flat seabed.

The lateral resistance (Coulomb friction and passive soil resistance) was obtained as a function of model pipe displacement by moving the model pipe horizontally, whilst using the load cells on either side of the model pipe to record the horizontal reaction force. Photos of the model pipe at various stages of a pull out test are presented in Figure 12.

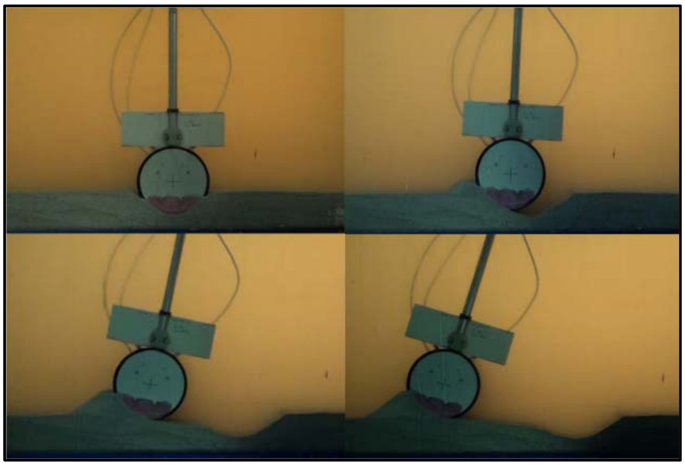

Figure 12. Pull Out Test Photo

The results of these pull out tests were used to develop an adapted version of the Verley model (Ref. 3). This Modified Verley Model (MVM) defined a tri-linear force-displacement soil passive resistance model which most closely represented the soil properties found along the pipeline.

The (MVM) remained dependent on the same factors as the original Verley model; however, the constants in the resistance force equations for F_{R1}, F_{R2} and F_{R3} were modified, as well as the mobilisation distances (y_1, y_2 and y_3). A chart demonstrating the Modified Verley Model fitted against various O-tube pull out test curves is presented in Figure 13.

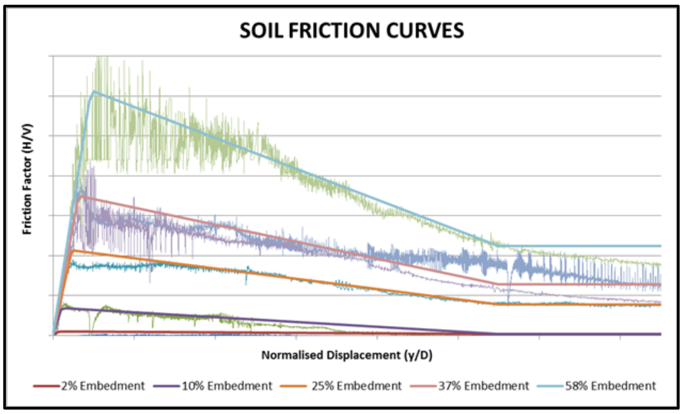

Figure 13. Summary Chart of MVM

5 FEA MODELLING

Following the physical model testing, a finite element analysis (FEA) software package called CORUS-3D was used to perform a 3D dynamic stability assessment of the pipeline. CORUS-3D has been developed using the general purpose FEA package ABAQUS.

Several FEA models have been developed by the pipeline engineering industry in order to analyse the dynamic stability of pipelines in three-dimensions. These software packages typically model the response of the pipeline subjected to irregular waves whilst resting on a rigid seabed with given soil resistance properties. The results of these modelling campaigns are then used to assess the pipeline according to the relevant limit state design criteria. These criteria are typically based on allowable displacement, and allowable stress and strain. At the present time, the models available in the industry allow the designer to vary both the hydrodynamic loading, and soil resistance during the storm series based on changes in the embedment profile of the pipeline. Some models can also simulate the change in pipeline embedment over time due to self burial caused by small oscillatory displacements.

CORUS 3D has been developed with similar capabilities to these software packages. In addition to this, it has been developed with the capability to vary the soil passive resistance based on changes in embedment caused by local scour.

The main features that constitute the CORUS-3D stability model are:

- Model geometry and associated material properties
- Mesh
- Contact interactions
- Loads
- User subroutine
- Boundary conditions.

5.1. Geometry and Material Properties

The finite element model consisted of a pipeline section, represented by a 3-D beam model. The seabed was represented by an analytical rigid surface. Contact definitions in the normal and tangential directions were defined between the two model components.

The cross-sectional (outside diameter and wall thickness) and mechanical material properties of the pipeline were assigned to the beam elements.

The analyses were set up as linear elastic models, incorporating non-linear geometry effects which accounted for changes in stiffness due to large displacements and deflections. The model was based on the following assumptions:

- The seabed is an idealised rigid flat plane with no inclination.
- The pipeline is an idealised straight beam.

The embedment profiles observed in the 2005 pipeline survey were recorded and averaged over 5 m sections of the pipeline. As a consequence, the pipeline model was partitioned every 5 m along its length so that the applied loads, which were defined by the initial embedment profiles, could be applied along matching 5 m intervals.

When defining the geometry of the pipeline to be assigned to the beam model, only the steel section of pipeline was considered given that the concrete weight coating (CWC) was assumed not to contribute significantly to the structural integrity of the pipeline.

The section of pipeline considered to be the most unstable was partitioned into a number of shorter and more manageable sections for modelling in FEA. The sections of pipeline modelled varied in length from 200 metres up to 2,000 metres.

5.2. Mesh

The discretisation of the beam model coincided with the partitioning. Consequently, each beam element was 5 metres in length. The beam element type used for each analysis was a PIPE31H element. This is a 2-node linear pipe in space with a hybrid formulation.

5.3. Contact Interactions

The node-to-surface contact interaction defined between the beam model nodes and rigid surface (seabed) comprised of both normal and tangential contact behaviour. The normal contact behaviour was specified as hard contact (with separation allowed) and the tangential contact behaviour used a classical Coulomb friction approach.

5.4. Boundary Conditions

Boundary conditions were applied to each end of the beam model. The ends of the model were constrained in all six degrees of freedom.

The impact of these constraints on the analysis results was mitigated by ensuring that the end regions of the analysis model corresponded to sections of the pipeline that were fully buried or had been rock dumped.

5.5. Loads

The following loads are applied to the model pipeline:

- Gravity
- Buoyancy
- Hydrodynamic Loads (Lift, Drag and Inertia)
- Soil Passive Resistance to Pipeline Displacement
- Soil-Pipeline Friction.

5.6. Hydrodynamic Loading

The hydrodynamic lift and drag loads were applied orthogonally to every 5 m element, as uniformly distributed vertical and horizontal loads respectively. The beam elements for a section of the pipeline with a particular initial embedment profile were subjected to the force-time history obtained from the corresponding physical model tests.

A simplified schematic detailing the application of hydrodynamic forces in the FEA model is shown in Figure 14.

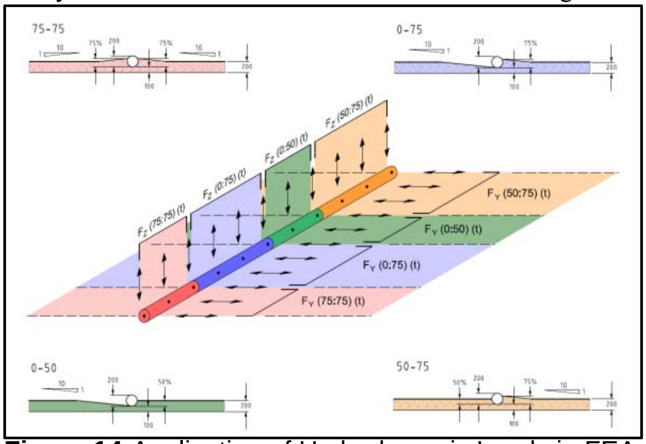

Figure 14: Application of Hydrodynamic Loads in FEA

A time offset was also calculated using Equation 1 to account for the 3-dimensionality of wave loading along the pipeline. The application of hydrodynamic loads acting in phase along the length of the pipeline was considered too conservative given that the wave attack angle specified in the metocean data was not perpendicular to the pipeline.

It is to be noted that this time offset is not an exact reflection of reality. In reality, the time offset will be irregular and variable in both the time and space domain. However, for simplicity a linear time offset was assumed.

$$T(d) = \frac{d}{c} \cdot \cos(\theta) \qquad \text{(Equation 1)}$$

Where:
- T(d) is the time offset as a function of distance (s)
- c is wave celerity (m/s)
- d is the distance from the start of the pipeline section (m)
- θ is the angle between the sea state direction and the pipeline (degs).

The wave celerity (c) has been calculated assuming a linear wave with a significant wave height and associated period which corresponds to the 100 year RP metocean data.

5.7. Soil Resistance

Friction between the pipeline and seabed was modelled using the classical Coulomb friction model. Coulomb friction was established between the nodes of the pipeline section and the rigid seabed surface using a node-to-surface, tangential contact definition.

The Coulomb friction model assumes that the friction force is proportional to the normal contact force (F_{ci}). The friction force F_f is described by Equation 2.

$$F_f \leq \mu F_{ci} \qquad \text{(Equation 2)}$$

Where: μ is the friction coefficient
F_{ci} is the normal contact force.

A small friction mobilisation distance of 0.5% of the pipeline diameter was incorporated into the application of the friction model.

The soil passive resistance was calculated during the analysis by means of an Abaqus specific user subroutine. The user subroutine defined a passive resistance force that was dependent on the:

- Lateral displacement of the beam model (y)
- Normal contact force between the beam model and rigid surface (F_{ci})
- Embedment level of the pipeline section (z)
- Soil resistance mobilisation displacements (y_1, y_2, y_3).

The passive soil resistance was set up in the FEA model as a non-uniform distributed load applied orthogonally to the pipe elements. The magnitude and direction (positive or negative) of the load was calculated using the soil passive resistance subroutine at each time increment.

The force-displacement curve was updated at specific time stamps for each individual element along the pipeline based on the change in embedment over time observed in the LOT tests. This allowed for the incorporation of changes in embedment during the analysis of a storm series.

5.8. Sensitivity Analyses

A number of sensitivity FEA runs were also performed including:

- Runs using reduced soil passive resistance values
- Runs including conservative changes to the soil model's relationship with contact force (F_{ci})
- Runs including temperature and pressure effects
- Runs using elevated hydrodynamic loads.

These sensitivity runs were performed to address potential areas of unconservatism in the modelling. They were also performed to determine the pipeline's sensitivity to changes in key input parameters.

6 LIMITATIONS

6.1. Modelling Post Breakout Behaviour

The stability assessment methodology proposed in this paper is most effective in assessing the stability of pipelines that have not been designed to move. This is because physical model testing cannot be used to accurately model post breakout fluid-pipe-soil behaviour. In future, it may be possible to develop a better understanding of post breakout behaviour using physical model testing. This study was beyond the scope of this project however. In the absence of this information, a highly idealised and conservative approach can be applied to pipelines that have been designed to displace laterally, whereby the pipeline embedment is assumed to be very small following breakout.

6.2. Scaling of Hydrodynamic Loading

The scaling ratios used for the physical model testing (see Table 1) were based on achieving similitude in the Froude criterion. This criterion was considered to be the most important criterion for the modelling of hydrodynamic loads.

However, achieving similitude in the Froude criterion did not allow the dimensionless Reynold's number (Re) to be scaled correctly. For flow regimes (i.e laminar, critical or turbulent flow) to be accurately modelled, Reynold's number must be the same for both the prototype and the model.

This is a limitation for all scaled down physical model tests involving hydraulic flow and is accepted within industry.

Since the Reynold's number could not be scaled exactly, the hydrodynamic loading recorded in the physical model testing may not have been entirely representative of the prototype conditions. It was noted that the flow in the LOT physical model tests was mostly in the sub-critical regime. However, the peak prototype flow velocities ($Re \sim 3 \times 10^6$) lie in the super-critical regime. The effect of a high Reynold's number is known to lead to a reduction in the drag forces on the pipeline. Consequently, the hydrodynamic loads derived from the LOT are predicted to be slightly higher than those expected in the prototype. This is expected to have led to conservative test results.

6.3. Scaling of Scour

The physical model testing was performed in the LOT using a model pipe that was scaled down at a scale of 1:5.8. The soil particle size was not scaled down accordingly due to various scaling difficulties associated with movable bed model tests (Ref. 2). This resulted in some scour scaling distortions since geometric similitude was not maintained.

Scaling distortions would have affected the following:

- Onset of free field scour
- Onset of local scour
- Rate of local scour (both tunnel scour and lee-wake scour).

These scaling distortions cannot be regarded as either conservative or unconservative since scour can act to both stabilise a pipeline (tunnel scour leading to pipeline embedment) or destabilise a pipeline (lee wake scour reducing pipeline embedment).

The length scale used in the LOT (1:5.8) was large relative to conventional physical model testing facilities. Typically, many physical model tests involving subsea pipelines have to date been performed at scales of 1:20 or smaller. While there is an expected level of inaccuracy in the test results associated with this scaling effect, it is a significant improvement compared to any of the smaller scale tests performed elsewhere.

Based on the equations provided in (Ref. 6), the scaling regime adopted for the tests is not expected to affect the equilibrium scour depth or the initiation of local scour.

6.4. Scour Induced Freespans

Once scour is initiated in a particular location along a pipeline route, it typically propagates along the length of the pipeline in both directions. This scour propagation is caused by flow amplification at the interface between the scour hole and the unscoured sections of the pipeline route (free-span shoulder).

Extensive scour propagation along the pipeline route will lead to the emergence of pipeline freespans. It is expected that over time, as the free span length increases, the pipeline at the midpoint of the free span will lower to the seabed under its own weight. Provided the scour hole has a sufficient depth, sediment will then deposit itself around the pipeline leading to self-burial. This self-burial process will be dependent on the rate and range of scour propagation as well as the pipeline's stiffness.

Scour induced pipeline freespans cannot be predicted using 2D physical model testing methods. An impractically wide test section would be required to model the effects of scour propagation along a pipeline route. In addition to this, there is a significant amount of complexity associated with the numerical modelling of scour propagation, especially when attempting to define when and where the original scour holes initiate.

It is recognised that scour propagation will affect the pipeline's embedment time history during a storm event.

6.5. Variability in Environmental Loading Conditions

Seven storm realisations were generated as part of the data acquisition process. The physical model testing and subsequent FEA analysis was run for the most onerous 100 year RP storm realisation out of these seven cases. It is important to recognise that probabilistically, there will always be a 100 year RP storm realisation with a higher peak velocity or more onerous loading combination than the one chosen from the seven storm realisations. However, its occurrence is less likely relative to the occurrence of a storm with a maximum flow velocity that tends towards the mean maximum flow velocity for the given wave spectrum.

A similar rationale can be applied to the time offset used to account for 3D wave crests. A linear time offset has been applied for the FEA analysis, as described in Section 5.6. In reality, there is always the possibility that peaks in hydrodynamic loading will act in phase over a length of the pipeline. In this case, it would not be possible to rely upon the stabilising effect of counteracting hydrodynamic loads. However, the probability that the pipeline will be subjected to a large hydrodynamic load acting in phase over a significant length of pipeline is relatively low.

6.6. Variability in Geotechnical Conditions

Comparisons of the model soil and available in situ soil properties indicated that in general, the key properties related to pipe-soil resistance were comparable. As a consequence, the pipe-soil resistance model developed from the pull-out tests was considered appropriate for use in analysing the pipeline's stability. It is noted however that there is the potential for the soil properties along the pipeline route to vary from the in situ samples used for the modelling. This implies a possible limitation in the applicability of the pipe-soil resistance model developed from the pull-out tests to all analysed sections of the pipeline.

However, it is noted that the available in situ data exhibits comparable properties to the model soil and was sampled at four locations along the route. As a consequence, the pipe-soil model developed from the pull-out tests was used as the base case (or best estimate) for the FEA modelling. This approach is in alignment with standard practice in pipeline design where soil properties are determined based on in situ testing and sampling at various spacing (often in the order of 2 km apart), which are then supported by geotechnical data gathering, such as acoustic profiling.

The sensitivity of the dynamic stability results to soil resistance was determined using reduced soil resistance values. This provided an indication of the sensitivity of the base case results to reductions in soil resistance potentially caused by variability in soil conditions (in addition to excess pore pressure as discussed below).

6.7. Excess Pore Pressure Build-up

It is important to note that the excess pore pressure ratios will be different in close proximity to the pipeline due to potential cyclic displacement of the pipeline and the impermeable pipeline surface.

Constraints inherent in the study limited the ability to predict the actual amount of excess pore pressure build-up, free-field or local, which was expected to occur during the design seastate.

In addition to this, a method for quantifying the variation in soil strength caused by excess pore pressure build-up was not available for this project. Given that potential excess pore pressure build-up is expected to lead to decreased soil strength, a reduction in soil passive resistance was used to assess the sensitivity of the base case results to potential excess pore press build-up.

While the sensitivity study did not provide accurate quantitative data on the effects of potential excess pore pressure build-up, it provided an indication of the sensitivity of the base case results to reductions in soil resistance potentially caused by excess pore pressure build-up.

7 CONCLUSION

The results of this stability analysis indicate that seabed mobility has, and most likely will continue, to play a significant role in the stability of pipelines on carbonate sands.

It is important to note that this stability assessment has been performed using one soil type, one specific 100 year RP storm realisation and one pipeline embedment profile (2005 survey data). With these limitations, an "absolute" answer on the stability of a pipeline, accounting for variability and uncertainty in all relevant parameters, cannot be provided using this method. Achieving this level of certainty would be impractical given the level of variability associated with geotechnical properties and metocean conditions, among others.

The following conclusions have been drawn from the stability assessment. The two key contributors to the pipeline's stability are:

- The highly embedded sections of the pipeline route. These highly embedded sections assist in stabilising the pipeline sections with lower embedment levels.
- The 3-dimensional nature of the wave loading. This led to opposing drag forces acting on adjacent sections of the pipeline. These counteracting loads in combination with the pipeline's structural stiffness significantly reduce pipeline displacements.

The effects of seabed instability could not be regarded as either conservative or unconservative since scour can act to both stabilise a pipeline (tunnel scour leading to pipeline embedment) or destabilise a pipeline (lee wake scour reducing pipeline embedment).

In a case where variable embedment is observed along a pipeline route and pipeline stability is a concern, significant cost savings can be made by adopting a similar stability analysis methodology. The cost of performing this study was estimated to have been in the order of fifteen times less than the cost of performing secondary stabilisation along the corresponding section of the pipeline route.

Elements of this approach could also be applied to stability analysis during the pipeline design phase. Less focus on assymetrical embedment and a larger focus on variations in metocean conditions and soil conditions would be expected in this case.

This approach cannot be applied to dynamically stable pipelines without applying unrealistic levels of conservatism. This is because it is not currently possible to model post breakout scour response using physical model testing.

The following topics require additional research to further improve the stability analysis method presented in this paper:

- Effects of excess pore pressure build up in close proximity to the pipeline.
- Scaling of excess pore pressure measured during physical model tests (from model scale to prototype scale).
- Rate of axial scour propagation at free-span shoulders.
- Breadth of excess pore pressure build-up along a pipeline (is it a localised or regional phenomenon)
- Further understanding of soil resistance to lateral pipeline displacement in a variety of carbonate soils, using various embedments and contact forces.

This study can be seen as a step forward in the development of a realistic and representative approach to pipeline stability design and analysis. It demonstrates how innovative approaches to the assessment of on bottom stability can be successfully applied to assess complex scenarios in a cost effective manner.

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9 REFERENCES

[1] Det Norske Veritas, 2007, "*On-Bottom Stability of Submarine Pipelines*", Recommended Practice DNV RP-F109

[2] Hughes. S, 1993, "*Physical Models and Laboratory Techniques in Coastal Engineering*", World Scientific, Singapore

[3] R.L.P Verley and T. Sotberg, 1992, *A Soil Resistance Model for Pipelines Placed on Sandy Soils*, OMAE – Volume 5-A

[4] Ryan. J., 2011, *A Fluid-Pipe-Soil Approach to Stability Design of Submarine Pipelines,* Proceedings Offshore Technology Conference, Houston

[5] Palmer. A.C, 1996, *A Flaw in the Conventional Approach to Stability Design of Pipelines*, Proceedings Offshore Technology Conference, Amsterdam

[6] Mutlu Sumer. B. and Fredsoe. J., 2002, *The Mechanics of Scour in the Marine Environment*, World Scientific, Singapore