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ADVANCED DYNAMIC STABILITY ANALYSIS

Hammam O Zeitoun
J P Kenny Pty Ltd., 221 St. Georges Terrace, Perth, WA, Australia

Knut Tørnes
J P Kenny Pty Ltd.

John Li
Mustang Engineering Pty Ltd.

Simon Wong
J P Kenny Pty Ltd.

Ralph Brevet
J P Kenny Pty Ltd.

John Willcocks
J P Kenny Pty. Ltd.

ABSTRACT

Several design approaches can be used to analyse the stability of subsea pipelines [1]. These design approaches vary in complexity and range between simple force-balance calculations to more comprehensive dynamic finite element simulations. The latter may be used to more accurately simulate the dynamic response of subsea pipelines exposed to waves and steady current kinematics, and can be applied to optimise pipeline stabilisation requirements.

This paper describes the use of state-of-the-art transient dynamic finite elements analysis techniques to analyse pipeline dynamic response. The described techniques cover the various aspects of dynamic stability analysis, including:

- Generation of hydrodynamic forces on subsea pipelines resulting from surface waves or internal waves.
- Modelling of pipe-soil interaction.
- Modelling of pipeline structural response.

The paper discusses the advantages of using dynamic stability analysis for assessing the pipeline response, presents advanced analysis and modelling capabilities which have been applied and compares this to previously published knowledge.

Further potential FE applications are also described which extends the applicability of the described model to analyse the pipeline response to a combined buckling and stability problem or to assess the dynamic response of a pipeline on a rough seabed.

KEY WORDS

Pipeline, subsea, on-bottom stability, finite element, stabilisation, hydrodynamic loading, pipe-soil interaction.

INTRODUCTION

Several approaches can be applied to investigate the hydrodynamic stability of subsea pipelines and to determine the stabilisation requirements necessary to ensure its integrity under extreme waves and steady current action. These approaches can be broadly grouped into:

- Static analyses.
- Calibrated or empirical methods.
- Dynamic analyses.

This paper focuses on the third type of analysis, which is the most comprehensive and complicated of the three. Dynamic stability analysis involves a time domain simulation of the pipeline dynamic response including the modelling of pipe-soil interaction and time variation of hydrodynamic loads.

Dynamic stability analysis has been used for several years [2], [4] to investigate the dynamic response of a pipeline exposed to hydrodynamic loads. An understanding of this response is fundamental to optimally determine the stabilisation requirements for the pipeline.

Pipeline stabilisation requirements can be a major cost driver on subsea pipeline projects and in some locations around the world where the designer is faced with extreme challenges in particular severe metocean conditions. Costly stabilisation requirements such as trenching, anchoring [5], rock dumping, and mattressing have therefore been used in the past to ensure the stability of a pipeline on the seabed.

An understanding of the pipeline structural behaviour requires an accurate formulation of the pipeline structural response, and a realistic description of hydrodynamic loads as well as pipe-soil interaction.

The common tools for modelling pipeline stability response has been to either use special purpose pipeline stability finite element (FE) packages such as AGA PRCI Stability [3] or PONDUS [4] or to use special purpose finite element models [6]. Special purpose FE stability packages often impose several disadvantages including:

- Simplified structural response assumptions including small deflection, two-dimensional motion and no consideration of geometric non-linearity.
- Seabed assumed to be flat.
- Modelling capability inflexible in the sense that non-standard features cannot be modelled, such as irregular seabed, presence of trenches or tie-in spools.
- No capability of modelling additional stabilisation measures such as trenching or anchoring.
- Pipeline stability considered in isolation from other design issues such as free spanning and buckling.

Finite element models developed using general purpose FE packages such as ABAQUS or ANSYS may be associated with:

- Inefficient computer run time.
- Simplified hydrodynamic modelling.
- Simplified pipe-soil interaction modelling.
- Requirement for advanced user FE knowledge.

An example of the above is the model developed by Ose et al. [6] which accurately modelled the pipeline and underlying irregular seabed, but included simplified hydrodynamic loading and pipe-soil interaction models.

The aim of this paper is to describe how an efficient integrated model can be developed to combine the advantages of dedicated stability FE packages together with the flexibility, advanced structural modelling, and advanced stabilisation modelling capabilities of general FE packages.

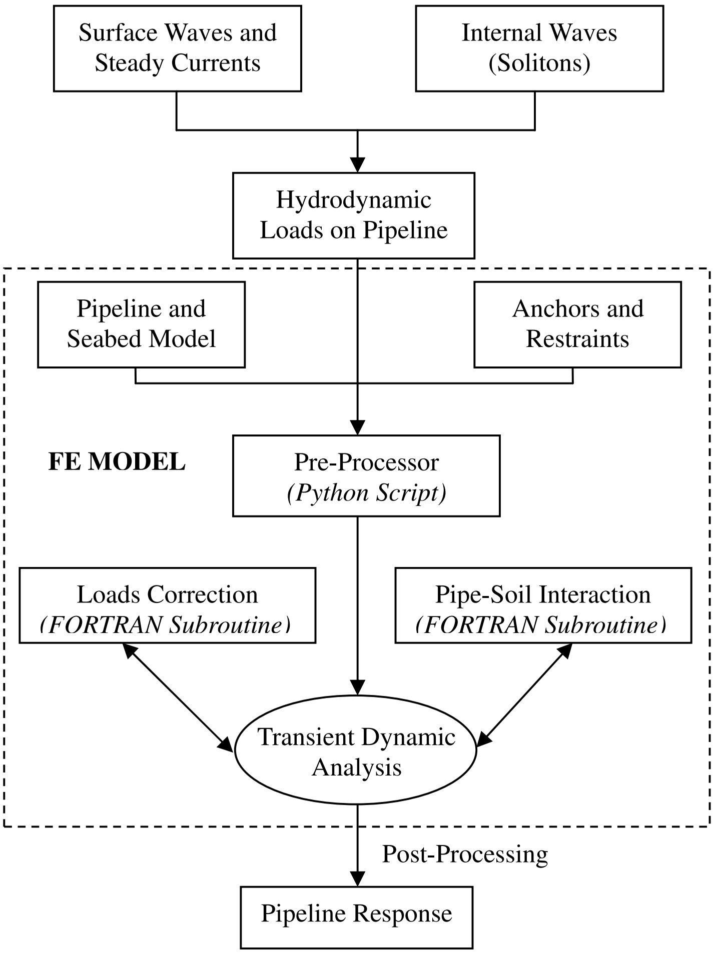
INTEGRATED MODEL STRUCTURE

The building blocks illustrated in FIGURE 1, and described below, form the basis of the integrated model which has been developed:

- A load model to generate the hydrodynamic loading on a pipeline resulting from an irregular sea-state or from internal waves (Solitons and Breaking Solitons) [7].
- A response model which uses ABAQUS for modelling the pipeline, seabed, anchoring and stabilisation methods.
- A pipe soil interaction subroutine which models the increase in embedment and passive soil resistance as a result of pipeline displacement.
- A load correction subroutine which models the effect of pipeline movement on the applied hydrodynamic loading.
- A Fatigue model which can be used (when required) to assess the fatigue damage occurring as a result of direct wave action and pipeline movement.

The pipe-soil interaction subroutine and the load correction subroutine have been integrated with the ABAQUS FE model to capture the effect of the pipeline response/movement on the applied loads and resistance.

FIGURE 1 – INTEGRATED MODEL STRUCTURE



DYNAMIC EQUATION OF EQUILIBRIUM

Using finite element, subsea pipeline stability analysis may typically be formulated within the framework of structural dynamics as follows:

$$[M]\{\ddot{x}\}+[C]\{\dot{x}\}+[K]\{x\}=\{F_h\}+\{F_v\}+\{F_r\}$$

Where,

$[M]$	Mass matrix of the structural system
$[K]$	Stiffness matrix of the structural system

$[C]$	Damping matrix of the structural system
$\{x\}$	Displacement vector
$\{F_h\}$	Hydrodynamic loads in in-line direction (drag and inertia loads)
$\{F_v\}$	Hydrodynamic loads in vertical direction (lift loads)
$\{F_r\}$	Lateral seabed resistance (Coulomb friction and/or passive soil resistance)

A direct time integration scheme, Hiber-Hughs-Taylor method, is used in ABAQUS [8] to solve the above equation which is capable of easily capturing the non-linearity inherent in the pipeline structural response. Pipeline structural response is solved at predetermined time steps and results from each time step are used to determine the entire response history of the pipeline.

The high non-linearity in a typical pipeline system stems from several sources including non-linear hydrodynamic loads history, non-linear pipe-soil interaction, plasticity in the material, large pipeline displacement and rotation, and stress stiffening which are all characteristics that ideally need to be considered when analysing the pipeline response in a dynamic stability analysis. This non-linearity render the pipeline response at the end of a particular loading event path dependent and limits the size of the acceptable maximum time step which can be used to ensure that the load path has been captured with acceptable accuracy. During the development of the model it was found that a maximum time step of approximately 1:10-1:30 of the wave zero-crossing is required to ensure the accuracy of the results. Assuming a zero crossing period of 15sec, a maximum time step of 1.5-0.5sec is required. Based on 3-hours storm duration, approximately 21600 time steps would be required in this case. The size of the time step is particularly important since it dictates the computational costs of the analysis. Within each time step, Newton-Raphson method [8] is used to solve the nonlinear equation of equilibrium.

PIPELINE MODEL

The pipeline can be modelled in the ABAQUS based FE model with pipe elements (PIPE31H) or beam elements (B31H).

PIPE31H and B31H are both 2-node linear structural beam element with hybrid formulation available from ABAQUS / Standard [8]. PIPE31H can be used for modelling rigid pipeline and B31H for modelling flexible lines and umbilicals due to the low bending stiffness of the latter which restricts using PIPE31H elements.

Compared to standard beam elements, PIPE31H and B31H are formulated for geometrical nonlinear problems and offer faster convergence which is critical for on-bottom stability runs. Six DOFs are associated with each of the two nodes (3 translational and 3 rotational DOF) as standard 3D beam structural elements.

For rigid pipes, von Mises metal plasticity model with isotropic strain hardening is used when material nonlinearity/plasticity needs to be considered. For flexible lines and umbilicals, the behaviour is assumed to be linear elastic. Bending stiffness, axial stiffness and torsional rigidity are typical design parameters which are required to define the mechanical properties of the beam element B31H used to model flexible lines/umbilicals.

Coatings and marine growth are not implicitly included in the definition of the pipe element PIPE31H. The pipeline stiffness model is only described by the linepipe properties. The contribution of pipe coating to the stiffness is neglected and only accounted for in modelling the pipeline submerged weight and mass. This simplification has been found not to have an impact on the pipeline displacement/response in a stability analysis.

Nonlinear geometry and stress stiffening effects are accounted for through continuously updating the system stiffness matrix according to the current geometry configuration and element state of stress during the solution process.

PIPE-SOIL INTERACTION MODELLING

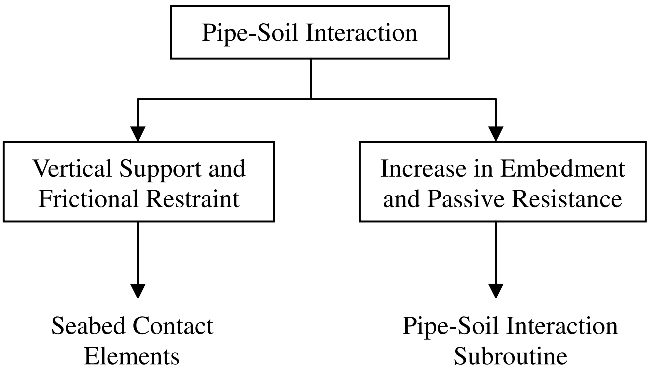
Pipe-soil interaction is an essential element in pipeline stability assessment. The physical processes involved in a pipe-seabed-fluid interacting system are extremely complex and are still a subject of continuing research [1]. The simplest method to model pipe-soil interaction is the basic Coulomb friction model. A Coulomb friction assumes pure constant plastic friction between the pipeline and the seabed and does not consider any loading history or passive resistance due to embedment. In this model, it is considered that the effect of seabed on the pipeline is the main concern while the effect of pipeline on the seabed is neglected. For example the development of berm /embedment as a result of pipeline cyclic movement is not considered in a Coulomb friction model.

Pipe-soil interaction is modelled in the ABAQUS FE model using a seabed model and a pipeline soil interaction subroutine. The seabed model serves two purposes:

- Providing “vertical/normal” support to the pipeline.
- Providing “lateral/tangential” resistance to the pipeline through a frictional mechanism (Coulomb friction).

The increase in embedment and passive resistance as a result of pipeline cyclic movement is not modelled as part of the seabed but using the pipe-soil interaction subroutine. This is illustrated in the FIGURE 2.

FIGURE 2 – PIPE-SOIL INTERACTION MODEL



SEABED MODEL

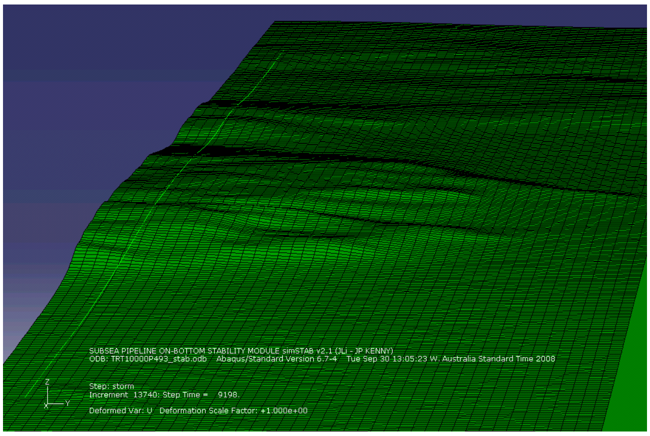
The seabed is considered non-deformable and is included in the model in two forms; a flat seabed or an irregular seabed. A flat seabed is modelled using an "analytical rigid surface" and the irregular seabed using "discrete" rigid surface elements (ABAQUS element type R3D4). A trenched seabed is modelled similar to a flat seabed using analytical rigid surfaces. These two modelling techniques consider that seabed deformations during a pipe to seabed interaction process are neglected. However, the resistance to pipeline movement provided by the seabed is captured through the contact interaction properties defined for the pipe and the seabed.

The main restriction on including a deformable seabed/soil model is imposed by the computational costs involved. With a large area of seabed that needs to be included in a typical dynamic stability analysis, the size of the FE model will increase which could render this approach impractical. Other restrictions include the difficulty of analysis convergence due to the occurrence of excessive localised deformation in the underlying soil elements, and the difficulty of capturing the behaviour properly or identifying an appropriate material constitutive model for it.

Geometrical modelling of both "flat" and "irregular" seabed is simple. Bathymetry data from route survey or direct from Digital Terrain Modelling (DTM) can be used to construct irregular seabed model. FIGURE 3 shows a typical model for a pipeline on an irregular seabed.

Interactions in the normal direction between pipe and underlying seabed (elements) is defined by a "softened contact with linear contact pressure and over-closure relationship" algorithm implemented in ABAQUS. A softened contact algorithm has been used since it is applicable for modelling soft layers on a contact surface (the seabed in this case).

FIGURE 3 – PIPELINE ON IRREGULAR SEABED



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SUBSEA PIPELINE ON-BOTTOM STABILITY MODULE simSTAB v2.1 (JLi - JP KENNY)
ODB: TRT10000P493_stab.odb    Abaqus/Standard Version 6.7-4    Tue Sep 30 13:05:23 W. Australia Standard Time 2008

Step: storm
Increment  13740: Step Time =    9198.

Deformed Var: U   Deformation Scale Factor: +1.000e+00
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In the tangential directions (pipeline axial and lateral directions) the frictional restraint can be defined by either:

- Standard ABAQUS Coulomb friction reinforced by penalty method; or by
- A specifically developed "tri-linear" Coulomb friction model which could capture an initially higher initial resistance prior to breakout [9].

Seabed soil stiffness is used to define the contact stiffness between pipeline and seabed, through the "pressure-over-closure" relationship. The frictional restraint change under the combined pipeline submerged weight and lifting forces, depending on the pipe to seabed contact force and contact over-closure/clearance.

In case of a pipeline separating from seabed due to the high lift forces, the "softened contact" algorithm assumes an elastic contact which does not dissipate any kinetic energy. Since the seabed is modelled with non-deformable rigid surfaces/elements, during pipeline landing and impacting the seabed, kinetic energy is not dissipated through soil plastic deformation and/or other possible mechanisms as it would be expected in reality. Instead it is conserved in the system and causes the pipe to seabed contact to behave essentially as a "mass-spring" system. To simulate the energy dissipation mechanism, and attenuate the possibly excessive contact-impact oscillation, appropriate damping (critical damping definition) has been introduced to dissipate partly or all of the kinetic energy associated with the contact/impact. Damping is only effective in the normal direction for the particular contact definition after contact/impact occurs.

PIPE-SOIL INTERACTION SUBROUTINE

A pipeline subject to oscillatory wave action is likely to develop complex interaction with the soil. Small movement may cause the pipeline to penetrate into the soil, whereas large movement may cause the pipeline to breakout laterally [10], [11] and [12]. For small cyclic lateral movements, the pipe tends to penetrate or embed into the soil. This is caused by the pipe pushing aside the soil, producing a berm or a mound formation, resulting in an increase in passive soil resistance. For large movements, the pipe will instead break out from embedment. When the pipe breaks out, a finite penetration or embedment is still retained. Therefore, in addition to Coulomb friction, there is also a residual lateral resistance due to a mound of soil being pushed ahead of the pipe after break-out.

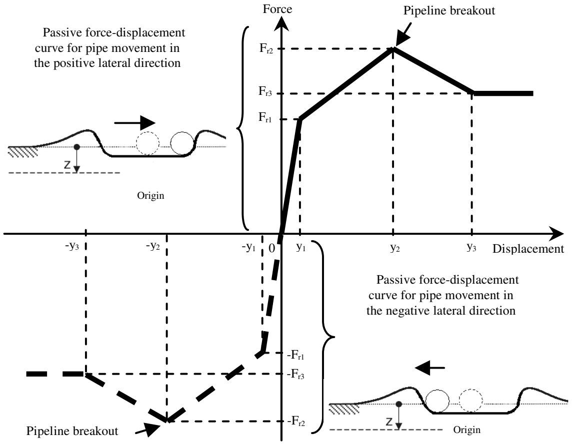
Therefore, the pipe movement, pipe penetration into the soil and soil resistance are interrelated. The pipe movement is dependent on the soil resistance, which in turn is dependent on the penetration caused by pipe movement.

The pipe-soil interaction subroutine implemented in the model predicts the development of pipe penetration into the soil and the associated increase in soil resistance that will result from this penetration. The subroutine is based on the principles described by Verley et al. [13]. The subroutine can simulate pipe-soil interaction for sandy soil, and can be modified to capture pipe-soil interaction for clay soil [14].

Based on the model developed by Brennodden et al. [10] and Verley et al. [13], the passive force-displacement curve consists of four distinct regions (see FIGURE 4):

- Pipe displacements between the origin 0 and y_1 are within the elastic region and no work is done by the pipe on the soil within this region. Therefore, the level of penetration remains unchanged.
- Pipe displacement between y_1 and y_2 cause an increase in penetration. The increase in penetration can be calculated from the work done by the pipe on the soil, as described later.
- Break-out is initiated for pipe displacement greater than y_2. Therefore, the penetration and the passive soil resistance decrease.
- For pipe displacement greater than y_3, the penetration and passive soil resistance remain constant.

FIGURE 4 – TYPICAL PASSIVE RESISTANCE FORCE-DISPLACEMENT CURVE



The above figure shows a typical passive resistance force-displacement curve.

Where,

F_{R1}	Peak elastic passive force
F_{R2}	Peak passive force
F_{R3}	Residual passive force
y_1	Mobilisation distance associated with F_{R1}
y_2	Mobilisation distance associated with F_{R2}
y_3	Mobilisation distance associated with F_{R3}

At the start of every solution time increment, the passive soil resistance on each pipe element is calculated according to the predetermined passive resistance force-displacement curve. The passive resistance is then applied to the element as a distributed external load which in effect opposes the lateral movement of the pipeline.

The development of the pipe-soil penetration is calculated from the amount of work done by the pipe on the soil through pipe lateral displacement [10]. The work done on the soil can be calculated from:

$$E(t) = \int_0^t F_r \, ds$$

Where:

E(t)	Work done by the pipe on the soil through pipe lateral displacement, from the start of the half-cycle, to a given time “t” after the start of the half-cycle.
F_r	Passive soil resistance.
Ds	Incremental pipe lateral displacement

The embedment developed during a half cycle is calculated based on the work done by the pipe, the pipe diameter, the soil submerged weight and the contact force between the pipe and the seabed.

The development of embedment is captured using the following equation developed by Verley et al. [13] :

$$\left(\frac{z_2 - z_i}{D}\right) = 0.23\left(\frac{E}{\gamma_s' D^3}\left(\frac{\gamma_s' D^2}{F_{ci}}\right)^{-1}\left(\frac{y}{D}\right)^{-1/2}\right)^{0.31}$$

Where:

E	Work done by the pipe (calculated as previously described)
γ_s'	Submerged unit weight of soil
z_2	Final penetration
z_i	Initial penetration
D	Pipeline outside diameter
y	Cycle amplitude
F_{ci}	Vertical pipe-soil contact force “at the instant of maximum break-out force”

The calculated embedment is used to determine passive resistance force-displacement curve (see FIGURE 4) where the peak passive lateral resistance (F_{r2}) is determined by:

$$\frac{F_{r2}}{\gamma_s' D^2} = \left(5.0 - 0.15\frac{\gamma_s' D^2}{F_{ci}}\right) \cdot \left(\frac{z_2}{D}\right)^{1.25}, \quad \frac{\gamma_s' D^2}{F_{ci}} \le 20$$

$$\frac{F_{r2}}{\gamma_s' D^2} = 2.0\left(\frac{z_2}{D}\right)^{1.25}, \qquad \frac{\gamma_s' D^2}{F_{ci}} > 20$$

The peak elastic passive force (F_{r1}) and the residual passive resistance force (F_{r3}) are determined similarly using the formulation described by Verley et al.

For further details regarding the theoretical basis for the implemented pipe-soil interaction subroutine, reference is made to the work published by Brennodden et al. [10] and Verley et al. [13].

SECONDARY STABILISATION MODELLING

Primary stabilisation (concrete coating and pipeline submerged weight) is often not sufficient to stabilise the pipeline. In this case, additional secondary stabilisation methods are used to restrict or limit the pipeline movement. A wide range of secondary stabilisation methods is available for this purpose. Secondary stabilisation method vary between intervention methods such as trenching and rock dumping to pipeline anchoring techniques such as rock bolts, strategic anchors [5], and gravity anchors. Modelling secondary stabilisation as part of the structural model provides a more accurate simulation of the combined pipeline-secondary stabilisation system and offers a better understanding of the combined system ultimate response.

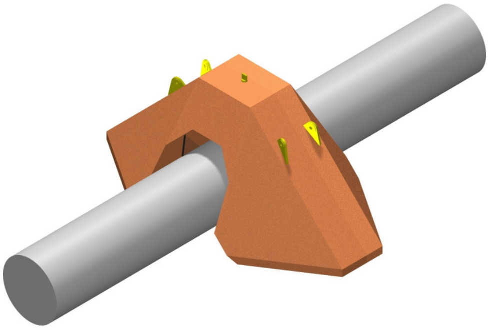
One of the distinct advantages readily offered by general purpose FE structural analysis packages such as ABAQUS is their capability of handling more complex design scenarios involved in analysing pipeline on-bottom stability, compared to other special purpose FE stability tools. This advantage offers the FE user the flexibility of modelling anchors and secondary stabilisation methods used for stabilising a subsea pipeline. The following stabilisation methods can be modelled using the developed integrated model:

- Strategic anchors.
- Rock bolts.
- Gravity anchors.
- Mattresses.
- Rock dumping.

The model can also be easily expanded to model other secondary stabilisation methods if needed. To illustrate how secondary stabilisations can be modelled, the modelling of gravity anchors is discussed next as an example.

Gravity anchors (GA) are large, arch shaped concrete blocks positioned astride the pipeline, as shown in FIGURE 5. If the pipeline moves sideways or upwards under extreme loading, the gravity anchors are engaged, providing additional frictional restraint of the overall system. Gravity anchors can be allowed to move under extreme conditions which impose an additional complex interaction in the design.

FIGURE 5 – GRAVITY ANCHORS



Realistic modelling and simulation of the interaction between the pipeline and gravity anchors under extreme environmental conditions is essential for an accurate assessment of the system response, and for ultimately providing an optimised design. The following aspects of the pipe-GA interaction should therefore be ideally captured:

- Contact/impact between pipeline and seabed;
- Contact/impact between pipeline and gravity anchors;
- Interactions between gravity anchors and seabed; and
- Realistic hydrodynamic load models to both pipeline and gravity anchors such as hydrodynamic load reduction and shielding.

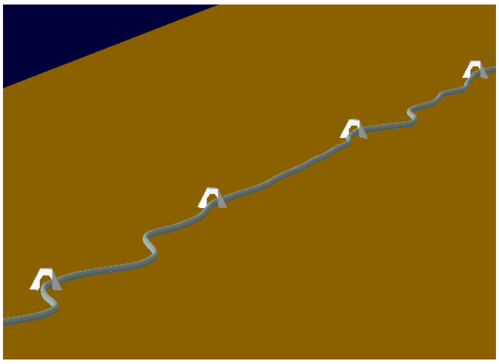
The GA-pipe contact behaviour is modelled using three-dimensional internal tube-to-tube contact elements (ITT31). ITT31 elements are used to model finite-sliding interaction between two “tubes” (pipe and GA) where one tube (GA) lies inside the other (pipe). Deformation of the “tubes” or pipe cross-section is not considered.

The great physical size of the GA suggests that the GA should be modeled as the “outer tube.” However, there is practically no difference in contact interaction and GA-pipe clearance between a contact model with the GAs specified as the “outer tube” and a contact model with the pipeline specified as the “outer tube.” By specifying the pipeline as the “outer tube,” the definition of the GA-pipe contact becomes more straightforward as a single slide line is sufficient and can be defined by the pipe nodes. If the GAs are specified as the “outer tube,” a slide line must be defined for each GA, resulting in many repeated contact definitions for an FE model with many GAs. No tangential friction is defined for the GA-pipe contact. The GAs are capable of sliding along the pipe axial direction

The GA-seabed frictional resistance is calculated based on a pure Coulomb friction model. Due to the lateral GA-pipe clearance, the pipeline will be restrained by the gravity anchor only if they are in contact. Locations where gravity anchors are modelled will therefore offer additional restraint to simulate the resistance offered by gravity anchors as the pipeline moves sideway. Movement of the pipeline inside the gravity anchors as it cycles on the seabed is also captured. Vertical restraint is provided in the event that the pipeline separates from the seabed and hits the gravity anchor. Hydrodynamic loads on the gravity anchors can be modelled explicitly or as a function of the loads on the pipeline.

FIGURE 6 shows a typical FE model of the pipeline, gravity anchor and seabed.

FIGURE 6 – GRAVITY ANCHORS AND PIPELINE MODEL



Similar to gravity anchors modelling, FE models have been developed for other secondary stability measures such as rock bolts, mattresses, rock dumping, and strategic anchors [5]. The description of the FE modelling details of each of these methods is difficult within the context of this paper.

IRREGULAR SEA-STATE SIMULATION

The hydrodynamic loads generation (under an irregular sea-state) consists of two main stages for calculating the wave forces on a subsea pipeline.

Stage one consists of generating the water particle velocities near the seabed along a pipeline, based on a specified wave spectrum. These near seabed water particle velocities are then used in the second stage to calculate the hydrodynamic forces on the pipeline. Finally the results are fed into the FE analysis.

The water particle velocities are generated at points (nodes) along the pipeline model. This starts by calculating the wave spectrum $S_{\eta\eta}$ (JONSWAP or Ochi Hubble) for a user specified significant wave height and peak wave period. A typical JONSWAP spectrum can be formulated as [3]:

$$S_{\eta\eta}(\omega) = B.\frac{H_s^{\,2}}{16}.\omega^{-5}.\omega_p^{\,4}.\exp\left(-\frac{5}{4}.\left(\frac{\omega}{\omega_p}\right)^{-4}\right)\ldots$$

$$\ldots\gamma^{\exp\left(-0.5\left(\frac{\omega-\omega_p}{\sigma.\omega_p}\right)^2\right)}$$

Where,

$S_{\eta\eta}(\omega)$	2D Spectrum
ω	Wave frequency
ω_p	Frequency corresponding to peal wave period
H_s	Significant wave height
B	is equal to 3.29 for a JONSWAP spectra
γ	Peakdness parameter

Directionality is included by the application of a wrapped normal spreading function, defined by the mean direction of wave propagation and a standard deviation of wave spreading, resulting in a 3D random sea state:

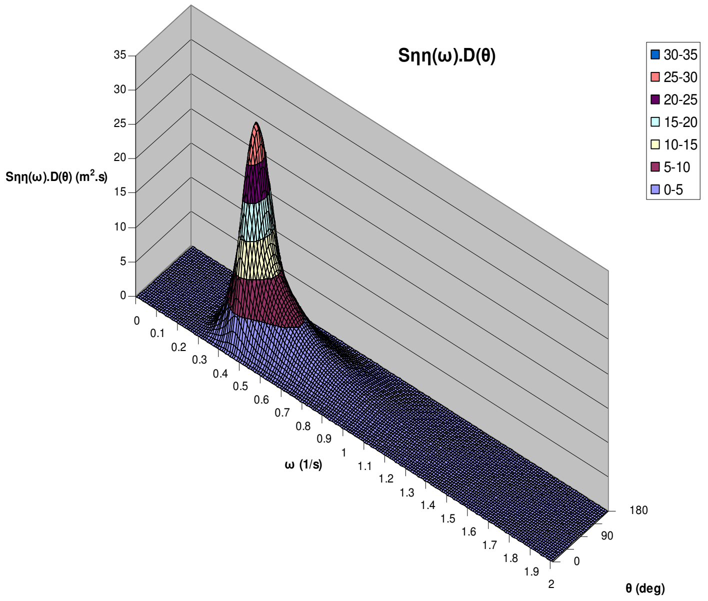
$$S_{\eta\eta}(\omega,\theta) = S_{\eta\eta}(\omega).D(\theta)$$

Where,

$S_{\eta\eta}(\omega,\theta)$	3D Directional spectrum
$D(\theta)$	Spreading function

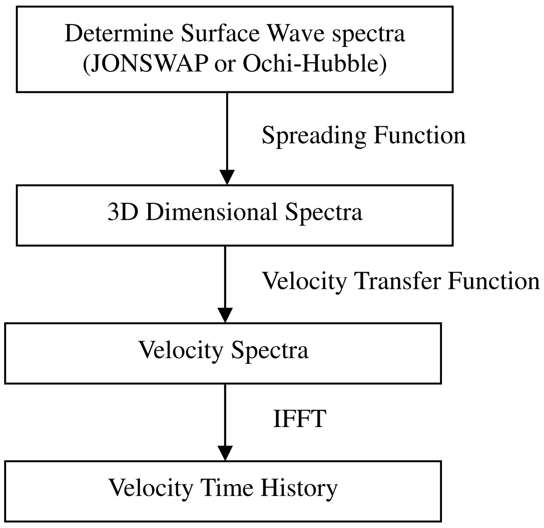
A directional spectrum is depicted in FIGURE 7.

FIGURE 7 –DIRECTIONAL SPECTRUM



Each frequency component of this wave spectrum at the surface is then transferred to the seabed using a frequency transfer function based on linear wave theory, yielding a near seabed velocity spectrum. Finally, the resulting near seabed velocity spectrum is used to generate a time series of (irregular) near seabed velocities by means of an inverse Fast Fourier Transform (IFFT). This process is outlined in FIGURE 8.

FIGURE 8 –VELOCITY GENERATION



The second stage of the load model consists of converting the near seabed water particle velocities to hydrodynamic lift, drag and inertia forces on the pipeline. The conversion is based on the Fourier coefficients model developed by Danish Hydraulic Institute (DHI) for the American Gas Association (AGA) [16]. This model provides a more accurate prediction of the variation of hydrodynamic forces with time compared to Morison equation [1]. A Fourier expansion is applied to obtain an accurate prediction of the time varying hydrodynamic forces on a subsea pipeline. The basis for this approach is that any signal with periodic variation T can be broken down to a number of waves with period equal to T and smaller. So the hydrodynamic loads caused by a wave can be decomposed to a number of sine waves (or harmonics) of period T, T/2, T/3...etc. Based on the work carried by DHI, using nine harmonics is sufficiently accurate to estimate the forces on a pipeline. The following general expression is used to determine the forces on a pipeline:

$$F(t) = \frac{1}{2}.\rho.D.U_w^2 \left[C_o + \sum_{i=1}^{i=N} C_i . \cos i(\omega t - \varphi) \right]$$

Where i is a subscript for the harmonic number, ω is the base wave frequency, and ф is the phase angle related to the base frequency. This expression is used to predict drag and lift forces on the pipeline. Inertia force on the pipeline is determined using the traditional Morison type formulation together with a theoretical inertia coefficient value of 3.29.

This Fourier model is sufficiently accurate to predict the time varying hydrodynamic forces on a subsea pipeline. The calculation of these forces for an irregular sea-state starts by decomposing the near seabed velocity time series in half-waves between the zero crossings; the irregular velocity time series at every node is therefore decomposed into equivalent regular waves. For each equivalent wave, the KC-value (Keulegan-Carpenter) and, in case of a steady state current, the current/wave velocity ratio (α) are calculated. The calculated KC and α values are used to determine the Fourier coefficients to be applied in the above expression. The Fourier coefficients are selected from a database established by DHI [16] from flume tests with scale models for different pipe configurations and environments, such as different pipe roughness.

The drag and lift force time series on the pipeline during each equivalent regular half wave are reconstructed using the selected Fourier coefficients. Wake reversal effects (from preceding wave) are incorporated by using the properties of the previous half wave up to a specified position in the current half wave. This effect is controlled by an overlap parameter for which values have been experimentally determined [17].

The force time-history generated by the above procedure is then applied to the pipeline nodes when performing the transient FE analysis. The forces are corrected for pipeline movement using the load correction module as described later.

INTERNAL WAVES SIMULATION

Internal waves, or solitons, are short-duration events which act on a pipeline over relatively short distances, and are driven by temperature and density gradients in the water column. These events occur in some areas around the world such as the

Australian North Western Shelf and are further described in [18].

The main factors that affect the severity of internal waves are the vertical density structure (thermocline depth), local seabed slope and tidal forcing. Other factors include the nature of the background currents and the relative irregularity of the surrounding bathymetry. These forcing mechanisms should be considered when assessing the severity of internal waves.

Due to the typically high current velocities, short duration, and limited crest length, associated with soliton events, adopting static stability analysis methods results in much more conservative estimate of the stabilisation requirements compared to the results achieved by using dynamic analysis. Dynamic stability analysis presents a much less conservative analysis method, by allowing some pipeline movement and by accounting for the short durations and limited length of soliton events.

Similar to hydrodynamic loads generation under an irregular sea-state, the velocity time history due to solitons or breaking solitons is initially determined. The event being simulated is assumed to be a coherent steady current. Based on defined event parameters including crest length, propagation speed, and event shape, a velocity time history is determined which act on the pipeline for a certain duration and along a defined crest length. Event shape can be modelled based on a sine or hyperbolic secant squared profile. The profile of a soliton in the direction of soliton propagation can be described by a sech^2 wave profile as:

$$A = \eta \sec h^2 \left\{ \frac{x - ct}{L} \right\}$$

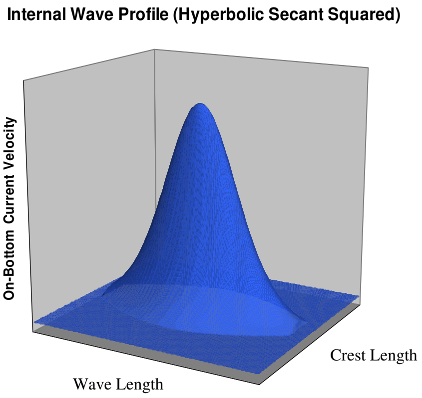
Where

η	Peak amplitude (peak velocities)
C	Phase speed (~ L/T)
L	Wave length
T	Wave period (event duration)
t	Time
x	Distance
A	Amplitude (velocity) along the wave length

A typical soliton profile is shown in FIGURE 9. The direction of propagation and the action of multiple events are considered in the developed model.

Based on the determined velocity time history, the time series of hydrodynamic forces along the pipeline are determined based on Morison equation and user-specified hydrodynamic coefficients, and are subsequently applied in the FE analysis to determine the resulting pipeline response.

FIGURE 9 – TYPICAL SOLITON PROFILE



HYDRODYNAMIC LOAD CORRECTION – MOVING PIPELINE

The hydrodynamic loads calculation process described previously is adequate for a stationary pipeline. A Load-Correction subroutine is used in the FE model to correct the forces for pipeline movement.

Hydrodynamic loading on a pipeline is a function of the flow and relative velocity of the pipeline to the flow [20]. The force-correction subroutine corrects and reduces the force to account for the relative motion between the pipeline and the flow. The correction applied by the subroutine is based on the methodology proposed by Vagner et al. [20].

The drag and lift hydrodynamic forces calculated for a stationary pipeline are modified based on the relative pipe-flow velocity (U_e-U_p), where U_e is the effective (near pipe) velocity and U_p is the pipe velocity. It is emphasised here that the effective velocity (near pipe velocity) is different from the free stream velocity calculated away from the pipe. The use of near pipe velocity was proposed by Jacobsen et al. [19] to predict the loading on a subsea pipeline. The method provides good prediction of the loads when using Morison equation.

To correct the force for pipeline motion, the near pipe velocity needs to be calculated, which can be done using the forces on the stationary pipeline:

$$U_{NP} = \sqrt{\frac{F_D(t)}{\frac{1}{2}.\rho.D.C_D{}''}} . \frac{F_D(t)}{\left|F_D(t)\right|}$$

$F_D(t)$ is the drag force on a stationary (fixed) pipeline, and C_D" is the drag coefficient based on the near pipe velocity determined from [20]. The forces on a moving pipeline are then calculated using the relative velocity between the pipeline and the flow (based on near pipe velocity):

$$F_D(t) = \frac{1}{2}.\rho.C_D''.D.\left|U_{NP} - U_P\right|.\left(U_{NP} - U_P\right)$$

$$F_L(t) = \frac{1}{2}.\rho.C_L''.D.\left(U_{NP} - U_P\right)^2$$

Where U_p is the pipeline velocity. The above correction applies to both drag and lift.

Inertia is updated in the dynamic model by accounting for pipeline added mass. The inertia force on a pipeline moving in oscillatory accelerating water is composed of two terms the Froude-Krylov term and the added mass term:

$$F_I(t) = \frac{\pi.D^2}{4}.\rho.\dot{U}_t + C_a.\frac{\pi.D^2}{4}.\rho.\left(\dot{U}_t - \dot{U}_P\right)$$

Where C_a is the added mass coefficient, and C_M=1+ C_a. The above equation can be written as:

$$F_I(t) = \frac{\pi.D^2}{4}.\rho.C_M.\dot{U}_t - C_a.\frac{\pi.D^2}{4}.\rho.\dot{U}_P$$

The general expression of force in a dynamic analysis (ignoring the stiffness and damping terms) is F = m.a, where m is the structural mass and a is the acceleration. The second term in the inertia expression can be moved from the force side to the mass side and the expression written as F = (m + m_{added}).a, where m_{added} is given by:

$$m_{added} = C_a.\frac{\pi.D^2}{4}.\rho\,.$$

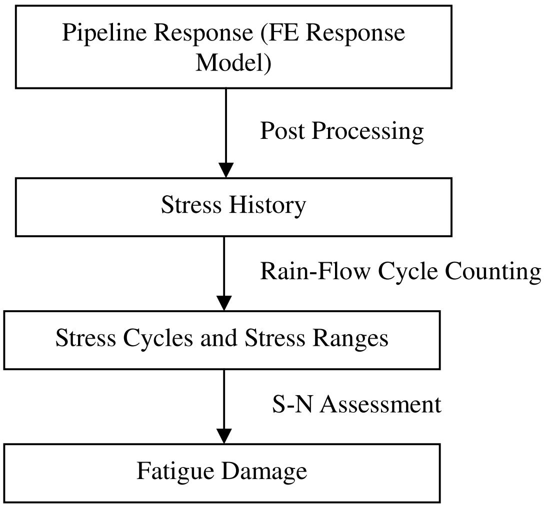
This is accounted for in the FE model by increasing the pipeline mass to include the added mass.

FATIGUE ASSESSMENT

A pipeline moving cyclically under the effect of waves may accumulate fatigue damage particularly at fixation or anchoring points.

A fatigue analysis model has therefore been developed and can be used to assess the fatigue damage of a pipeline during a storm. The fatigue analysis is based on the time history analysis performed for the pipeline. Based on the analysis results a stress time-history is determined for the pipeline throughout the storm. Cycle counting is used to determine the fatigue damage occurring during the storm. Cycle counting is based on the Rain Flow counting method [21]. Cycle counting is used to determine the number, and magnitude of the various cycle stress ranges occurring during the storm. Fatigue assessment is then performed based on defined S-N curves. This process is outlined in FIGURE 10.

FIGURE 10 –FATIGUE ASSESSMENT



ADVANTAGES OF USING DYNAMIC ANALYSES

The adoption of dynamic analysis in stability design typically results in significantly reduced requirement for stability measures compared to a classical absolute stability approach even for the cases where only minor movements are allowed.

When compared to Calibrated methods [1], dynamic analysis offers a much better understanding of the pipeline structural response, and its displacement pattern, and often results in less stabilisation requirements [15].

PIPELINE STABILITY ANALYSIS ON THE SEABED

The previous sections described the theoretical basis for the developed model as well as its structure. The following sections describe some typical applications where the model can be used to assess the pipeline dynamic response. The presented examples serve to demonstrate:

- Benefits of applying dynamic stability analysis.
- Typical model applications.
- Benefits of integrating pipeline stability analysis with other pipeline analyses such as buckling and on-bottom roughness.

A standard design case is to consider the stability of a pipeline section under the combined action of waves and currents during an irregular sea state or under internal waves. A finite element model typically consisting of 1km to 2km long pipeline section can be analysed to determine the stability of the pipeline under the specified design loads. The analysed pipeline model has the following features:

- The model is three dimensional. Dedicated pipeline stability finite element (FE) packages such as AGA PRCI Stability [3] or PONDUS [4] are one or two dimensional.
- The model accounts for geometric non-linearity and stress stiffening.

- Boundary conditions at pipeline ends assume symmetry condition.

The analysed model is used to assess the pipeline response by extracting (i) pipeline displacement (ii) pipeline bending moment (iii) strains, and any other information required to assess pipeline stability. The information can be extracted along the pipeline section and throughout the event duration.

The displacement of the pipeline during typical design events is shown in FIGURE 11 and FIGURE 12.

FIGURE 11 – PIPELINE DISPLACEMENT – IRREGULAR SEA-STAE (THROUGHOUT STORM)

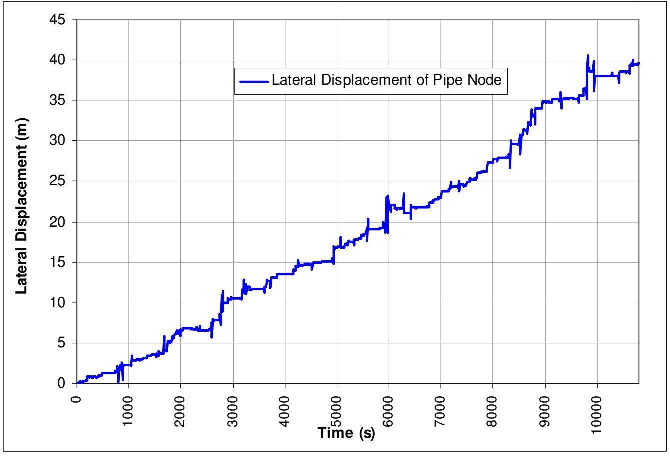
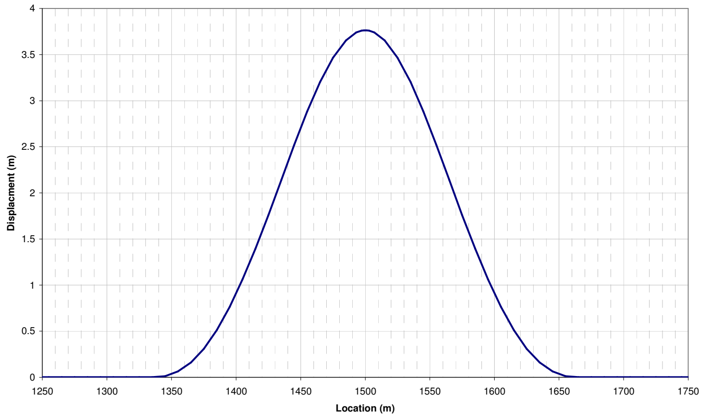


FIGURE 12 – PIPELINE DISPLACEMENT – INTERNAL WAVES (ALONG PIPELINE)



The analysis can be performed using simple Coulomb friction or by considering more complicated pipe-soil interaction models such as the energy based pipe-soil model described earlier.

The analysis can also consider seabed irregularity and assess its effect on the pipeline response. The analysis of pipeline on-bottom stability under extreme hydrodynamic conditions and free spanning resulting from seabed irregularity are often considered in isolation. Using the developed model, the effect of free spanning on the pipeline on-bottom stability may be assessed and an optimal design solution developed based on the results.

PIPELINE STABILITY ANALYSIS IN AN OPEN TRENCH

A trench is a mean of providing secondary stabilisation to a pipeline. The trench can be backfilled following installation or in some cases left open (no backfill) for the entire pipeline design life. Pipeline stability in an open trench is therefore commonly assessed either in the temporary (installation) or permanent (operation) condition.

Pipeline stability in an open trench is typically assessed using static analysis to determine the stabilisation requirements that ensure absolute stability of the pipeline at the bottom of the trench. However, these stabilisation requirements can be significantly reduced through the application of dynamic analysis, where the movement of the pipeline in the trench is allowed as long as the pipeline does not climb to the top of the trench wall and thereafter break out of the trench.

An example of the dynamic analysis performed for a pipeline in an open trench is shown schematically in FIGURE 13. The example considers a pre-dredged pipeline section in a trench of 5m bottom width, 3m depth and 2:1 side slopes. Pipeline displacements during an extreme sea-state are shown in FIGURE 14. The displacement pattern demonstrates how the pipeline moves inside the trench without breaking out, and how movement of the pipeline can be allowed inside the trench, hence reducing the stabilisation requirements.

FIGURE 13 – PIPELINE STABILITY IN OPEN TRENCH

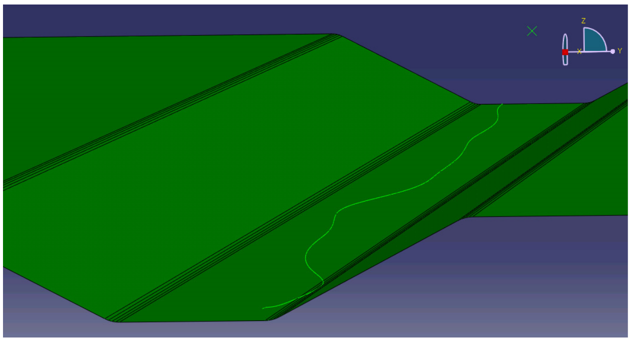
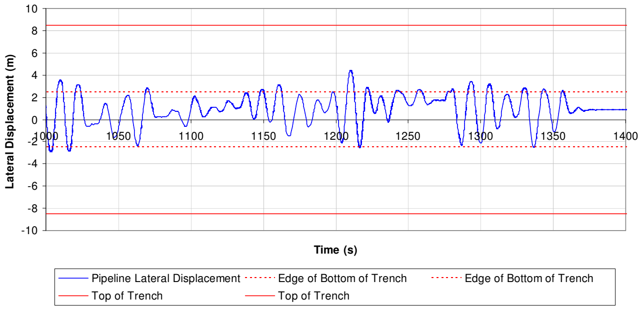


FIGURE 14 – PIPELINE DISPLACEMENT PATTERN IN OPEN TRENCH

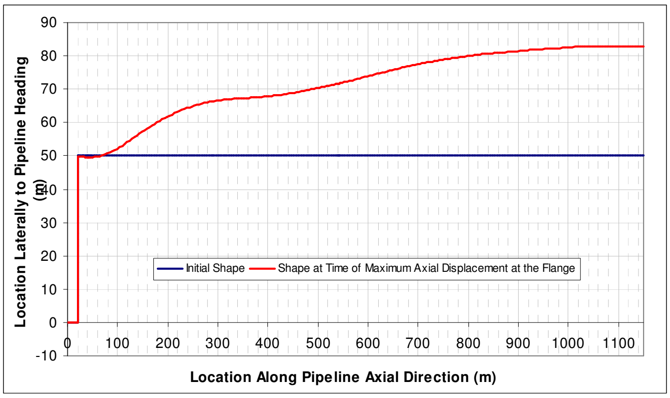


PIPELINE STABILITY AT FIXATION LOCATIONS

The on-bottom stability of the pipeline is commonly assessed at fixation locations such as the stability of the pipeline at a tie-in spool location or at the transition/exit from burial.

A distinct advantage of using general purpose FE packages such as ABAQUS as the FE engine is the ability to model the combined pipeline and spool system at the tie-in location. This system is modelled by varying the weight, mass, stiffness and orientation of the pipe elements according to the properties of the pipeline or spool. The hydrodynamic loads are also varied depending on the diameter and orientation of the pipeline or spool. The spool end attached to the structure/riser is modelled as a location of high restraint. A combined pipeline and spool model which has been analysed is illustrated in FIGURE 15. The on-bottom stability of the pipeline and the spool in-place analysis are often considered in isolation. By analysing the system as a whole, the response of the system to extreme hydrodynamic loading can be more accurately captured.

FIGURE 15 – PIPELINE/SPOOL MODEL



Another critical design case is the analysis of the pipeline at locations where secondary stabilisation terminates, and transitions to only primary stabilisation. An example is the exit of a trench or rock berm, where the pipeline exits from a buried or covered state in the trench or rock berm to an exposed state on the seabed. Under extreme hydrodynamic loads, the exposed pipeline may displace significantly while the buried pipeline is restrained in the trench or berm, resulting in inducing high bending moments at the transition of the buried/covered pipeline. Therefore, it is crucial that the integrity of the pipeline at this location is verified [15].

The assumption of an infinitely high restraint to pipeline movement in the trench or rock berm is considered to be very conservative with respect to the bending moment along the buried/covered pipeline, which is only partially restrained by the trench or rock berm. A less conservative model can be applied by defining finite restraint on the buried/covered pipeline through the implementation of plastic spring or connector elements. The developed model captures tension build-up during the storm (as a result of the end restraint), and the effect of this end tension on limiting the pipeline displacement and stabilising the pipeline. An example of using the model for this application is presented in [15].

INTEGRATION OF PIPELINE ANALYSES

Another advantage offered by general purpose FE structural analysis packages such as ABAQUS is the capability of integrating the stability analysis, with other analyses performed for the pipeline such as buckling or on-bottom roughness.

As an example the model can be used to analyse the pipeline stability at a buckling displacement-initiator location. Displacement initiators have been used in the past to initiate buckles at specific locations along the pipeline route. Displacement initiators such as sleepers (with low friction) or buoyancy modules are used for such purpose. The stability of the pipeline at the displacement initiators can be a concern. The model can be used to restart from the results of a buckling analysis and assess the stability of the pipeline at a buckle location following its buckling. This allows the designer to determine more accurately the effect of the introduced displacements initiators on the pipeline dynamic response as well as the combined stability and buckling response of the pipeline.

The model can also be used to assess the stability of a pipeline on an irregular seabed or at a particular free span location. This can be applied to assess the movement of a pipeline on a free span support and to determine the effect of the pipeline spanning on the resulting displacement.

CONCLUSIONS

The various aspects of a dynamic pipeline stability analysis have been presented. The techniques which can be used to cover these aspects and to develop an advanced pipeline dynamic stability model have been outlined. These techniques have been applied in the development of an integrated FE model which can be used to analyse the dynamic response of subsea pipelines under hydrodynamic loading. This integrated model/tool has been called SimStab. The tool is based on ABAQUS, and combines the efficiency, and advancement of specially developed stability analysis software together with the flexibility of general purpose FE programs.

The various elements of the tool have been described, and the main features for modelling hydrodynamic loads, pipe-soil interaction and the pipeline structural response have been presented.

The paper also presented potential applications for the model and highlighted the main benefits achieved by applying advanced dynamic stability analysis techniques for assessing the pipeline response, and in integrating pipeline stability analysis with other analyses.

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