Proceedings of the ASME 2009 28th International Conference on Ocean, Offshore and Arctic Engineering
OMAE2009
May 31 - June 5, 2009, Honolulu, Hawaii, USA

OMAE2009-79893

A STABILITY DESIGN RATIONALE - A REVIEW OF PRESENT DESIGN APPROACHES

Knut Tørnes

Hammam Zeitoun

Gary Cumming

John Willcocks

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

ABSTRACT

Pipeline hydrodynamic stability is one of the most fundamental design topics which are addressed by pipeline engineers. In its simplest form, a simple force balance approach may be considered to ensure that the pipeline is not displacing laterally when exposed to the maximum instantaneous hydrodynamic loads associated with extreme metocean conditions. If stability can be ensured in a cost efficient way by applying a minimal amount of concrete weight coating only, this method when applied correctly, can be regarded as a robust and straightforward approach.

However in many cases pipeline stabilisation can be a major cost driver, leading to complex and costly stabilisation solutions. In these circumstances, the designer is likely to consider more refined methods in which the pipeline is allowed to displace under extreme conditions.

This paper discusses various design approaches and acceptance criteria that are typically adopted in pipeline stability design. Both force balance methods and calibrated empirical methods which are typically defined in modern design codes, are discussed in terms of their applicability as well as their limitations.

Both these approaches are based on the assumption, directly or indirectly, that lateral displacement is a Limit State in its own right. It will be argued that this assumption may lead to unnecessary conservative design in many circumstances. It will be demonstrated that even if relatively large displacements are permitted, this may not necessarily affect the structural integrity of the pipeline.

An alternative stability design rationale is presented which is based on a detailed discussion of the Limit States pertaining to pipeline stability. This approach is based on the application of advanced dynamic stability analysis for assessing the pipeline response. Pipeline responses obtained through advanced transient finite element analyses is used to illustrate how a robust design can be achieved without resorting to strict limits on the permissible lateral displacement.

KEY WORDS

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

INTRODUCTION

A standard engineering task when designing subsea pipelines is to ensure that the pipeline is stable on the seabed under the action of hydrodynamic loads induced by waves and steady currents. A comprehensive discussion of the various design approaches for on-bottom pipeline stability that have traditionally been adopted by pipeline engineers was presented by Zeitoun et al. (2008).

Conventionally a subsea pipeline has been considered stable if it has got sufficient submerged weight so the lateral soil resistance is sufficiently high to restrain the pipeline from deflecting sideways.

Since it often will not be cost efficient to increase the steel wall thickness in order to increase the submerged weight, the primary stabilisation method has traditionally been to apply sufficient amount of Concrete Weight Coating (CWC) to achieve the on-bottom stability.

Since there is a practical limit to how much CWC can be applied to a pipeline, e.g. due to pipe lay vessel tension capacity limitation or due to limitation to the practical thickness that can be applied to a pipeline with a given diameter or to the pipe joint weight that can be practically handled in the coating plant, a secondary stabilisation method may have to be adopted such as through lowering the pipeline into the seabed by pre-lay dredging or post lay trenching or by using on-seabed restraints. The latter can involve covering the pipeline by crushed rock. However, where trenching and backfilling or rock dumping is not technically feasible or cost efficient, the designer may have to resort to more costly and technically challenging solutions such as anchoring the pipeline to the seabed for example as discussed by Brown et al. (2002).

An example of an area where pipeline stability is a major challenge is the Australian North West Shelf (NWS) due to the combination of shallow water, the severity of environmental loading during the passing of tropical cyclones and a seabed that over large areas comprises a thin veneer of sand overlaying calcarenite rock. In this environment, the cost of stabilisation is often very significant where Capital Expenditure (CAPEX) cost of stabilisation can represent as much as 30% of the total pipeline CAPEX (Brown et al. (2002)).

Under these conditions, there is a strong motivation for the pipeline designer to adopt the most refined design methods available in order to most importantly reduce the risks associated with on-bottom stability and also where possible reduce any conservatism that may be inherent in the traditional design approaches so to bring the cost down.

This paper discusses how advanced transient finite element analyses may be used to gain a better understanding of the pipeline structural response when exposed to severe hydrodynamic loads and how this can be used to challenge the more traditional approaches adopted in on-bottom stability design.

TRADITIONAL ASSESSMENT METHODS

Force Balance Method

The traditional design approach for submarine pipelines which is expressed in the early design codes, i.e. such as “Rules for Submarine Pipeline Systems”, DNV (1981), was to not to allow for any horizontal movement when a pipeline is exposed to the environmental conditions associated with an extreme return period, i.e. traditionally taken as the metocean conditions with a 100 year Return Period.

The static stability approach is based on a simple force balance calculation:

$$\gamma_s . F_H = \mu . \left(W_s - F_V \right) \qquad \text{Eqn. (1)}$$

where γ_s is a safety factor typically taken as 1.1, e.g. see DNV-OS-F101 (2000), F_H is the horizontal hydrodynamic load, W_S is the pipeline submerged weight, F_V is the vertical lift force and μ is the Coulomb friction factor.

Although this method has been widely replaced by the calibrated or empirical methods described below, the force balance method is still in common use particular for conditions that falls outside the range of validity of the calibrated methods, e.g. which is the case for a pipeline exposed to pure current.

Simplified and Generalised Methods (DNV-RP-E305)

Alternatives to the traditional methods were introduced as a result of extensive research and developments performed in the 1980s in the area of pipeline stability, e.g. refer to Wolfram et al. (1987) and Allen et al. (1989). This work was mainly contained within two Joint Industry Projects (JIPs), namely American Gas Association’s AGA JIP and the PIPESTAB JIP.

As part of these JIPs, special purpose dynamic FE analyses programs were developed in which advanced hydrodynamic loading and pipeline-soil interactions models were introduced, i.e. the AGA stability software (Pipeline Research Council International (PRCI) (2002)) and PONDUS from the PIPESTAB JIP (Holthe et al. (1987)). Both JIPs introduced approaches in which it was accepted that some movement can be allowed during extreme sea states provided that the lateral displacements were kept within defined limits. This is for example reflected in the widely used DNV-RP-E305 (1988) which introduced two simplified or ‘calibrated’ methods that do not require full dynamic FE analyses, i.e. the Simplified Method and the Generalised Methods.

The Simplified Method is as the name suggests a simplification in which the design curves in the Generalised Method (see below) has been replaced by a quasi-static method using a simple equation similar to that presented by Eqn. (1) but in which a calibration factor has been introduced together with non-physical force coefficients. The intention of this was to provide a method that ties the classical static design approach (Eqn. (1)) to the Generalised Method through the calibration of the classical method with the results from dynamic FE simulations. Inherent in the Simplified Method is an expected maximum lateral displacement of 20m.

The Generalised Method comprises a set of design response curves which have been developed based on a large number of dynamic FE simulations using the PONDUS FE stability software. The background for the methodology used to develop the design curves is presented by Lambrakos et al. (1987) and is based on the assumption that the pipeline lateral displacement is to a large extent a function of a relative small number of non-dimensional parameters. The Generalised Method in E305 (1988) thus comprises a set of design curves for various allowable displacements, δ, ranging from 0 to 40. δ is the displacements normalised by the external diameter of the pipeline, i.e. δ=Y/D where Y is total lateral displacement and D is the external diameter of the pipeline.

The calibrated methods represent simple design methods which are deemed to be relatively conservative as long as they are not used outside their areas of applicability. It is useful to be reminded about the most important limits of the E305 (1988) Simplified and Generalised Methods, i.e.:

- The methods are not applicable for pipelines with external diameter less than 16";
- The methods are not applicable for strong current dominated regimes, i.e. for a current to wave ratio exceeding 0.8;
- It is not applicable outside soil parameters which the pipe-soil model is based on – this is further discussed below.

For design scenarios outside the above applicability, it has been common design practice to revert back to the static stability method represented by Eqn. (1) above.

DNV-RP-F109 CODE REQUIREMENTS

E305 (1988) has recently been replaced by the new Recommended Practice, "On-Bottom Stability of Submarine Pipelines", DNV-RP-F109 (2007). It is not the intention here to discuss the new code requirements in any depth, however it would be useful to outline the main differences to the previous Recommended Practice that it replaces.

Absolute Lateral Static Stability Method (DNV-RP-F109)

It noted that the Force Balance Method defined in DNV-OS-F101 (2000) is not longer included in the revised DNV-OS-F101 (2007) and also that the Simplified Method of E305 (1988) is no longer available in the new stability code F109 (2007).

From this it appears that the intention is that Absolute Lateral Static Stability Method replaces both these methods. The Absolute Lateral Static Stability Method will ensure that no pipe motion will occur even when exposed to the maximum load during a seastate. It is further based on a Load Resistance Factor Design (LRFD) approach with additional partial safety factors which is said to satisfy the target safety level in F101 (2007).

This method appears by inspection to be significantly more conservative than the more traditional Force Balance Method, i.e. considering the relatively high partial safety factors ranging from 1.32 to 1.64 for safety class Normal.

This is also implied in the new Recommended Practice as it is suggested that for wave dominated conditions, the zero displacement requirement is likely to lead to the requirement for a very heavy pipeline and it is stated that the requirement "may be relevant for stability e.g. pipe spools, pipelines on narrow supports, cases dominated by currents and/or on stiff clay".

The authors of this paper can appreciate a more cautious approach with stricter stability requirements for current only or strong current dominated situations or in the cases of narrow supports, however would be more inclined to select a less stringent requirement for pipelines or part of a pipeline system for which lateral displacement is not critical for the pipelines integrity. In most situations some minor pipeline movements (<1m) can safely be allowed, which will significantly reduce the required pipeline submerged weight.

Generalised Lateral Stability Method (DNV-RP-F109)

The Generalised Method first introduced in E305 (1988) has been maintained in the new F109 (2007), however has been significantly revised and updated.

It is noticeable that there is no longer stated limitation on the validity of the method in terms of pipeline diameter or on the current to wave ratios, i.e. the parameter space for which the Generalised Method is applicable appears to have been significantly expanded.

As opposed to E305 (1988) which presented design curves for lateral displacements ranging from 0 to 40 times the external diameter of the pipe, the new design code is based on a lateral displacement limited to 10 pipe diameters during the given seastate.

It is not clear from F109 (2007) why the new code will produce different results to the old E305 (1988) Generalised Methods, However, the authors understanding is that one of the main underlying technical difference between the old and the new RPs is that the that the pipe-soil model which the design RP is based on has been updated to reflect better clay and sand models as prescribed by Verley et al. (1992 and 1995).

DYNAMIC ANALYSIS

In addition to the simplified methods discussed above, both the old E305 (1988) and the new F109 (2007) specifically allows the use of advanced dynamic FE analyses for on-bottom stability.

Although the use of transient dynamic FE analyses to calculate pipeline structural response is the most comprehensive method available to assess pipeline stability, the method has not been widely used by pipeline engineers for several reasons. Firstly, in many locations around the world where stability can be readily mitigated by applying a minimal amount of CWC, there has not been a strong motivation for replacing the simplified force model or the calibrated methods with a more advanced FE based method. Secondly, design tools based on these methods are not easily available; There are only two widely recognised FE based special purpose pipeline stability packages in existence, i.e. the PONDUS software which the calibrated method in F109 (2007) is based on and the AGA stability software. Of these two, only the AGA software has in the past been commercially available with the PONDUS software just recently been made commercially available.

Even though FE analyses have become a prevalent and very powerful tool in pipeline design which is used for a variety of structural design problems, there have been few attempts by pipeline design houses to develop their own FE based stability tools. The main reason for this is essentially down to two main challenges; Firstly a major obstacle is to develop a proper simulation of the time varying load imposed on the pipeline. It has been shown that the traditional approach based on Morison equation is not appropriate and more advanced techniques for example based on Fourier coefficients have been shown to better replicate the actual load history. Although the background for these methods can be found in the public domain, it is quite a challenge to implement this into a FE analyses package. The second challenge is the pipe-soil interaction model (or models). Again the background for the sand and clay model which the AGA and the PONDUS programs are based on can be found in the public domain, however the implementation of these theories into a FE package is far from trivial.

Despite the above and due to the significant challenges that the company is regularly faced with in stability design for pipelines on the Australian North West Shelf (NWS), J P Kenny decided to develop their own transient FE package for stability. Apart from the local design challenges on the NWS, the main motivation has been to address some of the inherent limitations in the existing design analyses packages and to offer the designer more flexibility for addressing non-typical design scenarios not addressed by the commercial packages.

Details of the ABAQUS based FE package which has been named SIMSTAB is presented and discussed by Zeitoun et al. (2009). Some of the main features of the SIMSTAB model are as follows:

- The hydrodynamic force model is based on the Fourier Expansion Method developed by the Danish Hydraulic Institute (DHI) and discussed by Bryndum et al. (1988) and which is the model used in the AGA Level 3 analysis – see PRCI (2002).

- User can choose between Coulomb friction or an energy based sand model similar to the pipe-soil friction model believed to be included in PONDUS – see below. The former may be useful if outside the applicability range of F109 (2007) where no good pipe-soil model presently exists, e.g. for sands with silt content > 20% as commonly found on the NWS.

- The energy based pipe-soil resistance in SIMSTAB is based on the pipe-soil interaction model described by Brennodden et al. (1989) revised with improved sand model proposed by Verley (1992) and which is understood to be the basis for the latest update of PONDUS.

- The model is in full 3D with each pipe node having 6 degrees of freedom. This is as opposed to PONDUS which is understood to be a 1D FE tool with each node only free to move laterally and so cannot capture the effect of axial tension or bending stiffness. AGA is understood to be 2D model in the sense that tension and bending effects can be captured however the pipe nodes are restrained from displacing vertically.

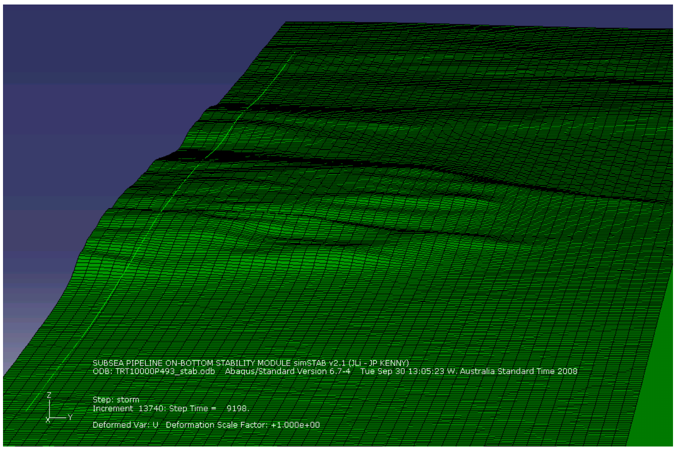

Figure 1: SIMSTAB Modelling Details – Uneven Seabed

- As opposed to PONDUS and AGA, the SIMSTAB tool can readily cope with a full 3D seabed thus can deal with pipelines that instantaneously lifts off the seabed or alternatively used to assess the effects of spanning on seabed stability – See Zeitoun (2009).

- Full non-linear linepipe material can be modelled which can be useful close to fixed points when strain based design are being applied.

- Inclusion of tie-in spools possible.

- Inclusion of other fixed points such as trench transition and burial possible.

Figure 1 presents a typical modelling detail in which an unstable pipeline has traversed an uneven seabed, i.e. from right to left in the picture, during the simulation of a 3hr storm. In this particular case, the analyses showed the pipeline would move in the order of 120m laterally during this exposure to a storm with 10,000yr return period.

LATERAL DISPLACEMENT AS ACCEPTANCE CRITERIA

Traditionally engineers have applied absolute stability criteria to pipeline on-bottom design based on a 'Design' environmental event, i.e. the pipeline has not been allowed to displace or move laterally at all when exposed to extreme wave and current loading although it has been realised for a long time that this is a very conservative approach.

By allowing the pipeline to undergo small cyclic lateral movements this can significantly reduce the requirements for CWC in comparison with a design based on simple force balance method in which no displacement or movement is allowed. Furthermore, small cyclic movements are likely to lead to increase pipeline embedment into the soil which in effect can significantly increase the pipe-soil resistance and thus further stabilise the pipeline. This type of mechanism is for instance addressed in the energy based pipe-soil resistance model (Brennodden (1989)) which is the original basis for the model adopted in the E305 (1988) code.

In addition, by realising that lateral displacement in itself does not necessarily pose any risk to a pipeline, it is now commonly accepted that some displacements can be allowed as long as it is ensured that it is kept within specified limits. For example in E305 (1988) the limit of 20m displacement is inherent in the Simplified method whereas design curves for lateral displacements between 0 and 40 times the pipeline external diameter are presented in the Generalised Method. The background or rationale for these limits is not specifically given, however it is believed that these limits were selected because they were considered reasonable and would not lead to what could be considered excessive displacements. The main risk associated with allowing lateral displacements is associated with possible fixed points that may exist along the pipeline at which the E305 (1988) proposed that no displacements should be allowed, e.g. typically would apply to the 500m zone near a platform where the pipeline would be connected by tie-in spools. Based on this it has been commonly assumed in the industry that these limits are code requirements that should not be allowed to be exceeded, i.e. even if the design is based on the use of advanced transient dynamic analyses rather than the alternative simplified approaches presented in the E305 (1988).

In F109 (2007) this has been further clarified; A maximum accumulated displacement of 10 times the external diameter, which is the basis for the Generalised Lateral Stability Method, is recommended "if other limit states, e.g. maximum bending and fatigue, is not investigated". It is further stated that larger displacements can be accepted but in this case full dynamic analyses are prescribed.

An example of a stability analysis performed using J P Kenny's SIMSTAB is presented in Figures 2 through 5. In this case a 1000m long section of a 36" pipeline installed on a flat seabed which has been analysed through a 3hrs storm with a 100year Return Period.

Figure 2 shows a plan view of the resulting pipeline configuration at the end of the 3hrs storm. In this condition the pipeline section has displaced laterally by an average of between 38-39m from its initial position. with a maximum displacement of approximately 40m at a position 400m from the end. It should be noted that symmetric end condition has been assumed which is strictly not correct because the imposed load is not symmetric, i.e. it varies along the length of the pipeline with time.

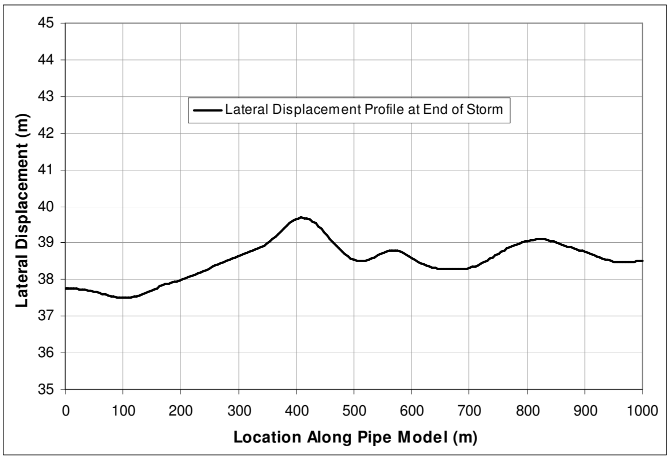

Figure 2: Plan View – Pipeline Deflected Shape

The resulting bending moment envelope at the end of the 3hrs storm is presented in Figure 3, i.e. which is the maximum bending moment imposed on each point along the pipeline throughout the force-time history.

It should be noted that the inaccuracy in the assumption of symmetric end condition manifests itself as a peak loads typically occurring at the model free ends. However, the overall accuracy of the method can easily be demonstrated by increasing the length of the model – a 2km long model will show almost identical overall displacement pattern and load level along the length of the model.

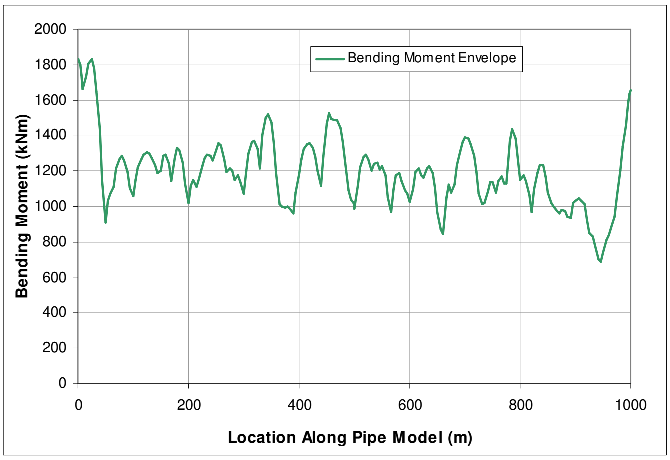

Figure 3: Maximum BM Envelope (absolute values) along Pipeline Section

For this particular pipeline the maximum permissible bending moment to satisfy the F101 (2007) local buckling check was found to be approximately 3800kNm for the Ultimate Limit State (ULS) condition, i.e. with the maximum bending moment of 1550kNm at approximately 350m from the end (discarding the results at the pipeline ends) this would give an approximate utilisation of around 0.4.

Figure 4 represent the nodal lateral deflection for the location of local maximum lateral displacement at approximately 500m from the end shown in Figure 2. It shows how this point on the pipeline cycles back and forth with passing of the higher waves which gradually results in a total deflection of 40m at the end of the 3hrs storm.

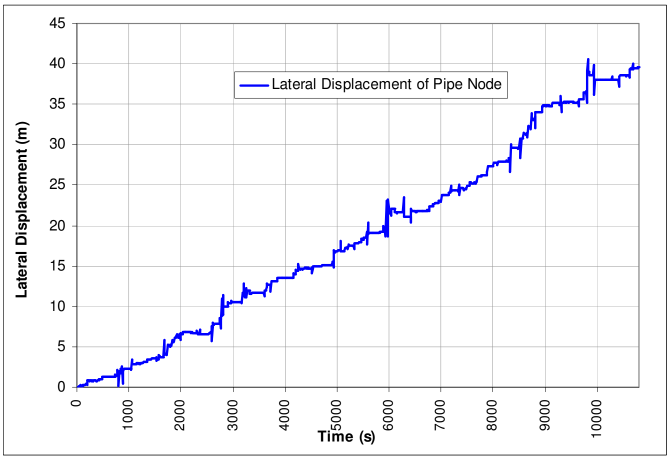

Figure 4: Nodal Lateral Displacement (midline) vs. time

Figure 5 shows how the bending moment at this point varies with time as the pipeline is gradually displacing sideways. Although the bending moments varies significantly over time, this shows that it is kept within a fairly well defined envelope which at no time approaches the maximum acceptable moment of approximately 3800kNm. This demonstrates clearly that there is no increase in bending load with time of exposure to the storm or in proportion to the lateral displacement.

It is apparent from this example that for a pipeline that is not physically restrained at any point along its length, the degree of lateral displacement does not affect the structural integrity in terms of strength.

However, as will be discussed in the following, what need to be addressed in a design that adopts an acceptance of significant lateral displacement, special consideration need to be made at locations where the pipeline is physically constrained and also the Fatigue Limit State need also be addressed.

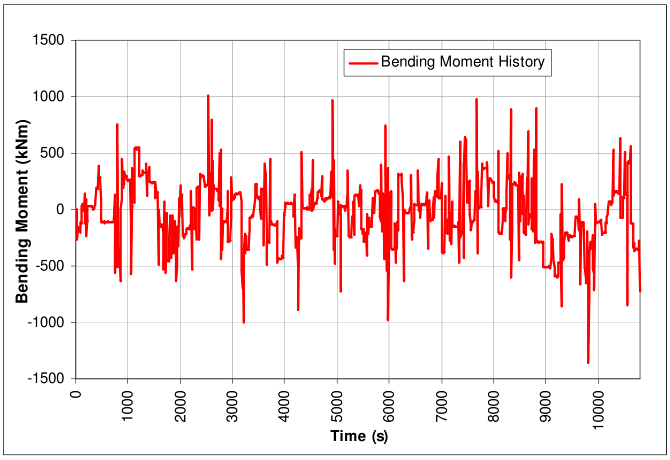

Figure 5: Nodal Bending Moment vs. Time (midnode)

STABILITY DESIGN WHEN CLOSE TO PIPELINE FIXED POINTS

The previous example demonstrates that accumulation of lateral displacement is in itself not likely to induce unacceptably high bending loads in a pipeline as the loads are shown to be fairly limited within a relatively tight envelope. The level of bending moment experienced by a pipeline or a section of a pipeline that is allowed to freely displace laterally will vary depending on both the soil resistance, the hydrodynamic loads as well as the bending stiffness of the pipeline.

However, the above is only true for part of a pipeline which is not close to lateral restraints. Lateral restraints or fixed points will always be present somewhere along all pipelines and can be due to physical obstructions on the seabed such as large boulders, or where pipeline is transiting from a trenched and buried state to a fully exposed condition or wherever the pipelines are being tied in to subsea facilities such as subsea manifolds, valve stations, platforms risers etc.

The risk for a pipeline that is in general allowed to accumulate lateral displacement is that the presence of a restraining point could lead to excessive loads in the pipeline where it is restrained or could impose unacceptable high loads on connecting equipment such as tie-in flanges, valves or structures.

With regards to natural obstructions such as large boulders that could restrain the pipeline from moving laterally these may be removed prior to pipelay.

For other restraining locations such as tie-in spools or trench transitions care must be taken if a design approach is adopted in which relative large lateral displacements are generally allowed.

Example: Tie-In Spools

Where pipelines are connected to other equipment, normally facilitated through the use of flanged tie-in spools, the pipeline often will have a load capacity exceeding the connecting equipment, i.e. such as bolted flanges, valves or spool bends or any structural restraints. The loads imposed on the connecting equipment through the pipeline may thus overload the equipment. This is often the main concern rather than the restraining load imposed on the pipeline itself.

It has therefore been common practice to not allow parts of a pipeline adjacent to tie-in locations to deflect laterally as a result of hydrodynamic instability. This is for instance reflected in the E305 (1988) code where it is proposed that no lateral displacements should be allowed in Zone 2, i.e. typically within a 500m distance from a tie-in location, unless it can be demonstrated that any displacement can be "acceptably accommodated by the pipeline and supporting structure".

Although this is in principle a sound approach and even if the pipeline in the 500m zone has sufficient submerged weight to be stable on its own, it may be difficult to demonstrate that it would not displace laterally when imposed by loads from the adjacent section which is not fully hydrodynamically stable. This is particularly a concern if a maximum permissible lateral displacement far exceeding the standard 20m is adopted for the adjacent section.

By using an advanced dynamic FE simulation where the tie-in spool geometry is included, allows a detailed assessment of the interaction between the different sections when exposed to the design storm.

Figure 6 shows an example in which a tie-in spool has been included as part of the FE simulation model. The blue graph shows the initial configuration of the L-shaped spool and the red graph shows the deflected shape at the end of a 3hrs design storm. At a distance of 1.1km from the tie-in spool, the adjacent pipeline has deflected laterally by approximately 30m. Even though the 400m straight section of the pipeline closest to the spool has got a sufficient submerged weight to be hydrodynamically stable on its own, a large portion is still deflecting sideways due the load imposed on it by the adjacent "unstable" section. However, it shows that the first ~60m of the straight section closest to the first spool bend is not displacing sideways which means that in this case no significant bending moment is being imposed on the tie-in flange which is located approximately 5m from the bend.

A zoomed in view of the initial and final configuration is presented in Figure 7 which showed that pipeline has defected axially by approximately 1m due to the axial tension imposed by the "unstable" part of the pipeline as it is displaced laterally.

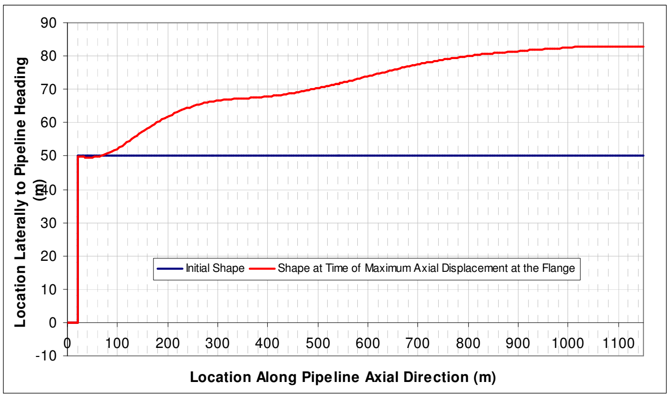

Figure 6; Birds View: Initial & Final Deflection of Tie-In Spool

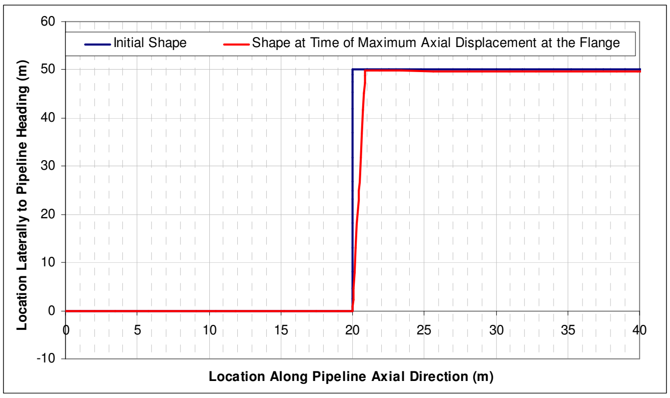

Figure 7: Zoom In:Initial & Final Deflection of Tie-In Spool

Example: Trench Transition

In this example a significant portion of a pipeline has been trenched and buried to provide shelter against the hydrodynamic loads along a section with seabed soil condition lending itself to a trenched solution. However, along a significant length of the route, the seabed mechanical properties are such that trenching is not found to be a technically feasible solution. Thus there is a transition in which the pipeline will change from being trenched and buried to being installed exposed on the seabed. This transition constitutes a fixed point which can attract significantly increases in bending moments as the exposed part of the pipeline displaces laterally.

If it is impractical to achieve sufficient stability by increasing the amount of CWC to render the exposed section stable, e.g. due to installation vessel tension capacity limitation or other reasons, this does not leave the designer with many attractive alternatives. One option could have been to continuously rockdump the pipeline. However it could be prohibitively expensive if a significant length of a pipeline would require a continuous rock cover. Furthermore, it could also be an added design challenge to have to provide a rock berm design that in itself will require to be hydrodynamically stable.

Another option that could be considered would be to anchor the pipeline at discrete points along its length. In addition to being a very expensive solution, this will introduce a number of restrained points along the pipeline. Although introducing a large number of restraints will limit the loads imposed on the pipeline at each restraint, this solution is generally not attractive due to the cost, installation and operational risks from introducing points of restraints.

An alternative that could be considered in this case is to assess the structural integrity at the trench transition by the use the transient dynamic FE analyses model. By assuming initially that the pipeline was completely fixed at the trench transition point, i.e. fully rotationally fixed, analyses showed that this would lead to a 30% exceedance of the allowable bending moment when exposed to a 100yr storm assumed for the ULS condition.

However by taking into account the fact that the pipeline in reality would not be 100% fixed at the trench transition, i.e. by representing the lateral soil resistance by the means of elasto-plastic springs with a finite maximum resistance, it was in this case possible to demonstrate that the integrity would not necessarily be jeopardised at the transition.

An example of analyses results is presented Figure 8. The vertical dashed line represents the location of the trench transition point, i.e. lateral soil-resistance corresponding to a buried line has been applied for the first 300m of the model. The blue graph shows the deflected shape using a 1.3km long model and the orange graph the deflected shape when increasing the model to 2.3km.

The lateral displacement of the pipeline away from the trench is significantly larger for the longer model. The longer 2.3km model shows a lateral end displacement similar to the displacement of an unrestrained model, i.e. which would represent a section of the pipeline far away from the lateral restraints.

Despite the above differences in displacements far away from the restraint at the trench transition, the displacement and deflected shape close to the transition is found to be very similar.

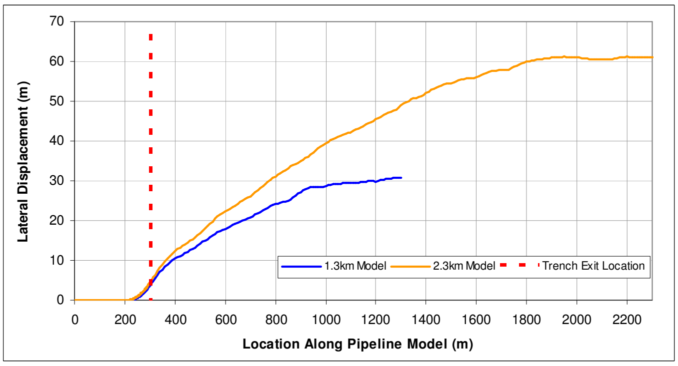

Figure 8: Trench Exit - Lateral Displacement Envelope

The resulting maximum bending moment along the two models are presented in Figure 9 and show that the bending moment distribution is very similar for the two models. There is a peak of approximately 2400kNm at the trench transition itself which reduces to around 1500kNm further away from the transition point. These values are still well below the maximum permissible moment of 3800kNm represented by the horizontal dashed line in Figure 9.

Note that the pipeline properties in this example was the same as presented in Figure 2 through Figure 5. However, because the pipeline section is located in shallower water the on bottom kinematics are higher resulting in a large total lateral deflection of 60m as opposed to 40m shown in the previous example.

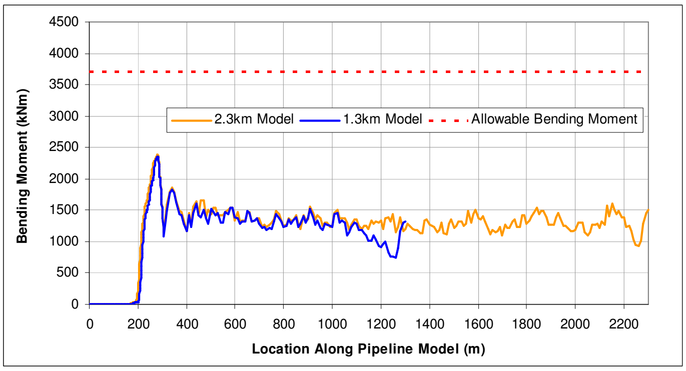

Figure 9: Trench Exit – Bending Moment Envelope

In addition to the check of the local buckling criteria, cyclic loading may also give raise to fatigue damage during the storm which could be a particular concern at a pipeline restrained point. One of the advantages of performing the full time-history analyses is that the resulting stress history can be readily obtained. The SIMSTAB program package includes a module that uses a rainflow counting technique for counting the stress cycles from the stress history, as outlined in ASTM (2005), which is turn used to perform full fatigue analyses. In this particular case, only insignificant fatigue damage was calculated based on the input from the 3hr design storm.

LIMIT STATES AND ACCEPTANCE CRITERIA

On-bottom stability in terms of Limit State Design and corresponding acceptance criteria to apply, often leads to some discussions among pipeline engineers. F109 (2007) is based on the same safety requirements as defined in the pipeline offshore standard F101 (2007) which is based on a Limit State Design approach. However, as it might not always be clear how the two codes relate to each other, the following outlines an approach or rationale that might be adopted. In this respect one should have in mind that F109 (2007) states that for other than the Absolute Lateral Stability Method, "the recommended safety level is based on engineering judgement in order to obtain a safety level equivalent to modern industry practice".

The following Limit States that may be considered and how they pertain to pipeline on-bottom stability is discussed in further details below:

Ultimate Limit State (ULS): Local buckling limit state should be considered for any pipeline that is allowed to displace laterally on the seabed for which high bending moments may be encountered.

Accidental Limit State (ALS): Local buckling limit state considering higher environmental return periods (than ULS) which may be adopted to capture non-linear structural response effects.

Serviceability Limit State (SLS): Consideration of excessive displacements as a limit state.

Fatigue Limit State (FLS): Cyclic loading may lead to fatigue damage for a pipeline that is allowed to move laterally along the seabed.

Ultimate Limit State (ULS)

F101 (2007) defines the ultimate limit state as *"A condition, which if exceeded, compromises the integrity of the pipeline"*, i.e. brings it to a point beyond which the pipeline could experience loss of containment.

With regard to pipeline stability response, the ULS condition can be assumed reached if the bending moment resulting from pipeline lateral displacement in combination with other loads, causes local buckling/collapse of the pipeline wall. For this condition, it is common to assume that the F101 (2007) Local Buckling criteria for load controlled situations would be the most appropriate acceptance criteria.

In order to check this limit state, full dynamic analysis is required, i.e. this is not directly addressed when the simplified or calibrated stability assessment methods are applied which in effect only considers lateral displacements. Furthermore, ULS condition is only likely to ever be reached for a pipeline that is not designed for absolute stability or very limited lateral displacement and as discussed previously then only at locations that are locally restrained.

The environmental load effect factor (γ_E), which is a component of the F101 (2007) local buckling equation, accounts for uncertainties in environmental data. Load combination B is considered to be the relevant load condition to be checked which defines a value of $\gamma_E = 1.3$ to be used in the ULS check.

As per the requirements of F109 (2007) and in line with common industry practice, the ULS condition should be considered in combination with environmental conditions with 100 yr return period.

However care need to be taken with regards to the application of the 100yr return environmental condition when considering ULS conditions for on-bottom stability, particularly with respect to the following:

As in other areas of offshore design, a maximum design return period of 100yr is commonly considered in on-bottom stability design which per definition has an annual probability of occurrence of 10^{-2}. This, which is referred to as the Characteristic Load, represents the most probable extreme load during the design life. Inherent in this approach is an assumption that more extreme events is covered by safety factors which then reduces the annual probability of failure (as opposed to the probability of occurrence of the load) to less or equal to 10^{-4} which is the target safety level for the ULS condition as per the F101 (2007). For example the environmental load effect factor (γ_E) of 1.3 is to be applied to the bending moments (i.e. the Load Effect) resulting from applying the Characteristic Environmental load.

The stability problem for a pipeline may often be very non-linear, particularly if initial pipeline embedment is taken into account or if the pipeline is installed in an open trench. In this case, the pipeline may be absolute stable until the imposed load reaches a certain threshold, beyond which the pipeline may break out and become significantly unstable. In this case, the uncertainty associated with the load can simply not be properly captured by the use of load effect factors applied to the resulting loads, which is a common problem associated with the Load and Resistance Factor Design when applied to a system which has a highly non-linear response. In these cases, it should be considered whether it would be more appropriate to apply the factors to the loads applied in the analyses (i.e. increasing the Characteristic Loads) rather than the resulting load (i.e. the Load Effects).

The second concern is whether the 100yr event in combination with the partial safety factors is likely to represent a sufficiently improbable event regardless of location, considering that the calibration of safety factors is typically based on North Sea conditions. Brown (1999) compared the return period and normalised drag force relationship for the North Sea and the Australian NWS for a typical 42-inch pipeline. This showed that the ratio of drag force associated with a 10,000 yr return period to 100yr return period was significantly higher for the Australian NWS. Such location dependent differences are now reflected in the safety factors in the new F109 (2007) to be applied in the Absolute Lateral Stability Method, however it is not specified how this could be addressed in designs based on dynamic analyses.

The assessment of a load with annual probability of 10^{-4} (ALS condition – see below) in addition to the 10^{-2} normally used in assessment of the ULS condition will indirectly capture some of this uncertainty related to the non-linear behaviour as well as the concern with regards to the appropriate location dependent extreme loading.

Accidental Limit State (ALS)

F101 (2007) defines the accidental limit state as *"An ULS due to accidental (in-frequent) loads"*. Thus as for the ULS condition, the ALS condition is a point beyond which the pipeline could experience loss of containment.

The annual targeted failure probability for ALS condition is exactly the same as for the ULS condition, i.e.10^{-4}, thus the Limit States are very similar except that the probability of occurrence of the load being considered for ALS is much lower. The target safety level is then achieved by setting all partial safety factors to unity and considering loads with an annual probability of 10^{-4}.

For on-bottom stability, the pipeline system should be subjected to an extreme environmental loading with 10,000yr return period in dynamic FE analysis to confirm its survival of the accidental limit state. With regard to pipeline stability response, the ALS is reached if the moment/strain resulting from pipeline lateral displacement causes loss of containment (local buckling) or loss of integrity to occur. Similar to a ULS, an ALS is accordingly reached when the pipeline exceeds its ultimate moment/strain capacity.

The local buckling check provided in F101 (2007) is separated into a check for load-controlled situations (bending moment) and one for displacement controlled situations (strain level). When no usage/safety factors are applied in the buckling check calculations, the two checks in principle should result in the same bending moment capacity. In design however, usage/safety factors are introduced for the ULS condition to account for modelling and input uncertainties. The reduction in allowable utilisation introduced by the usage factors is not the same for load and displacement controlled situations because the latter is considered less critical.

Because of the high utilisation allowed for accidental loads (partial safety factors set to unity), relatively large moment/strains should be acceptable for this limit state. For a pipeline operating well beyond the material proportionality limit, FE analyses based on elastic material properties will tend to overestimate bending moments and stresses and underestimate strains. Non-linear material properties must therefore be utilised in FE analysis in order to obtain more realistic results.

The moment curvature relationship provides information necessary for designing against failure due to bending. For an ALS condition, with the load factors set to unity, the "allowable" moment is equal to the "ultimate" moment which means that the pipeline is allowed to operate at the "flat" portion of the moment-curvature curve. Because the pipeline response in this region is not very sensitive to changes in strains but very sensitive to changes in moments, it can be argued that strain is an appropriate criteria for checking ALS with appropriate consideration of strain concentration effects at the pipeline field joints (this is also the reason why strain criteria is defined for displacement controlled load cases in F101 (2007)).

Based on the above discussion, it is appropriate to use strain as the assessment criteria for checking ALS conditions when subject to 10,000year environmental return period condition.

Serviceability Limit State (SLS)

In F109 (2007) it is suggested that excessive lateral deflection should be considered an SLS, without clearly specifying what is to be considered excessive. In accordance with F101 (2007), a SLS is a condition that renders the pipeline unsuitable for normal operation. For example it would mean that if the state of a pipeline was such that it needed to be de-rated (e.g. due to excessive corrosion), then the pipeline would not longer be considered fit for its intended purpose, i.e. the pipeline will have reached its Serviceability Limit State. It can be argued that on this basis, the lateral displacement in itself can not be considered a Serviceability Limit State unless the lateral displacement leads to a condition that would mean that normal operation of the line is affected. Then it would be this condition that is the actual Limit State, not the deflection that leads to it. For example if a pipeline was routed parallel to a deep scarp, then if a section was deflected so that it was hanging over the edge, it may be considered unsuitable for further service and has thus reached its Serviceability Limit State. This could be because the risk to the integrity from further operation is considered too high, not necessarily because the pipeline in its new condition had exceeded its strength capacity which would have been considered an Ultimate Limit State (ULS – see below).

It is commonly considered in on-bottom stability design that if the pipeline is deflecting outside its original survey corridor (which could typically be in the order of 20m-250m to either side of its centreline), i.e. has moved into “unknown territory”, then it can be said that the SLS condition is exceeded.

If such a criterion is applied, it then remains to decide what environmental condition one should be considered for exceedance of the SLS condition, i.e. in terms of environmental conditions and associated return periods. In accordance with F101 (2007), the main difference between ULS and SLS in terms of target failure probability is that the annual probability of exceedance is to be less than 10^{-4} for ULS and 10^{-3} for SLS for Safety Class Normal. This reflects the fact that it is considered more acceptable (less serious) to reach the SLS (economical impact only) than the ULS (Health and Safety risk).

One approach for the SLS condition could be to assume that the probability of failure is equal to the probability of the occurrence of the load. This would mean that the SLS condition should be considered using 1,000 year return period which has an annual probability of occurrence of 10^{-3}. However, this could be considered overly conservative as this acceptance of excessive displacement would indirectly mean that the pipeline is considered to have become inoperable as soon as it displaces outside the survey corridor which is not necessarily the case. If it is instead conservatively assumed that there is as much as 10% probability that displacement outside the survey corridor would render the pipeline inoperable, then this would in combination with a 100yr environmental condition, give an annual probability of 10^{-3} that the SLS condition will be reached (pipeline becomes inoperable). Using this argumentation, the criteria that may be adopted is that SLS is considered reached if the pipeline displaces outside the survey corridor when exposed to a single 100yr storm event. It is assumed that if “excessive” displacement is discovered following a 100yr event, the survey corridor may be widened so that subsequent 100yr events are not likely to bring the pipeline outside the survey corridor.

It could be argued that the rationale for this approach to deal with the SLS criteria may not always be robust as it does not directly address the problem related to the fact that the stability is a highly non-linear problem, i.e. that the threshold for severe instability may be experienced for a return period slightly higher than the 100year return period. Similarly to the ULS condition it could be argued that it may be un-conservative to not consider higher return period for displacements as a limit state.

On the other hand, should the designer apply the 1,000yr return period (annual probability 10^{-3}) to check the SLS condition for a pipeline which is found to be fully stable when exposed to the 100year return period, and finding that this would result in an unzipping action which causes “excessive displacements” and failure of the SLS acceptance criteria, this might not be appropriate either. What it would mean is that one will be defining stricter requirements to the SLS condition (failed for the 1,000year return period) than the ULS condition (passed the 100year return period) which is not logical considering that the consequence of exceeding the SLS condition is less serious than exceeding the ULS condition.

The above illustrates that the designer should always have the non-linearity of the on-bottom stability problem in mind and that the acceptance criteria for the SLS condition in terms of excessive displacements should be carefully defined on a case-by-case basis.

Fatigue Limit State (FLS)

F101 (2007) defines the accidental limit state as *“A ULS condition accounting for accumulated cyclic load effects*”.

A pipeline that is designed to be allowed to displace laterally will be exposed to cyclic loading from wave actions. For portions of a pipeline not close to lateral restraints, fatigue will not normally be a concern as the lateral displacement would in general only occur for short periods when exposed to extreme environmental conditions and the associated cyclic stresses are likely to be low. However, fatigue might be a concern at fixation points (i.e. such as trench exits, crossing…etc) where cyclic loading may cause a certain amount of fatigue damage during the pipeline design life. The amount of damage endured by the pipeline as a result of hydrodynamic instability in combination with the damage caused by other design drivers such as installation and free spans has to be below the maximum allowable fatigue damage.

Typically a pipeline that is allowed to displace under extreme environmental conditions will not move under ambient condition and fatigue analysis can therefore be limited to the time history stability analyses performed for these conditions (e.g. 100yr analysis). Based on the analysis results, a stress time history can be determined for the pipeline throughout the storm. Cycle counting can then be used to determine the fatigue damage occurring during the storm, e.g. based on the Rain Flow counting method as defined in ASTM (2005) as a means to summarise the 3-hour load versus time history by providing the associated number and magnitude of the various stress cycles occurring during the storm.

CONCLUSIONS

This paper has discussed various design approaches and acceptance criteria pertaining to on-bottom stability of subsea pipelines.

It has been shown that whereas simplified or calibrated methods may be sufficient for pipelines for which stability is not consider a major concern in terms of cost and risks, full dynamic FE analysis is invaluable for pipelines for which stability is a major design challenge.

Full 3D FE analyses are particularly useful for cases that fall outside the soil parameters for which existing pipe-soil interaction models are based on. This is typically the case for the Australian NWS where the seabed comprises calcareous soils with quite different mechanical properties than silica sand found e.g. in the North Sea. Another example is sand with large silt content in which undrained soil behaviour may be encountered. Even in these cases, dynamic analyses based on simple Coulomb friction is likely to yield significantly less conservative results than reverting back to simple force balance methods which inherently allows no lateral displacement.

In general, pipeline response obtained through advanced transient FE analyses may be used to achieve robust designs without having to resort to strict limits on lateral displacement. Such analyses allows the designer to properly investigate true limit states, e.g. by demonstrate that lateral instability will not lead to the exceedance of local buckling or fatigue limit state.

The dynamic analyses results presented in this paper was performed with the ABAQUS based full non-linear transient dynamic FE package SIMSTAB which was developed by J P Kenny to address the inherent limitations in the commercial available special purpose stability programs.

REFERENCES

Allen, D.W., Hale, J.R. and Jacobsen, V. (1989): "Submarine Pipeline On-Bottom Stability: Recent AGA Research", *Proceedings of the 21th Offshore Technology Conference , OTC6055, Houston, Texas.*

American Society for Testing and Materials (ASTM) (2005): "Standard Practices for Cycle Counting in Fatigue Analysis", ASTM E1049 - 85(2005)

Brennodden, H., Lieng, J. T., Sotberg, T. and Verley, R.L.P., (1989): "An Energy Based Pipe Soil Interaction Model", OTC6057, *Proceedings of the 21th Offshore Technology Conference (OTC), Houston, Texas.*

Brown, N.B. (1999): "A Risk Reliability Based Approach to On-Bottom Stability Design", *Proc. of Sate of the Art Pipeline Risk Management Conference, Perth, Australia.*

Brown, N.B., Fogliani, A.G., Thurstan, B. (2002): "Pipeline Lateral Stabilisation Using Strategic Anchors", *Proc. of the Society of Petroleum Engineers (SPE) Asia Pacific Oil and Gas Conference, SPE 77849, Melbourne, Australia.*

Bryndum, M.B. and Jacobsen, V., 1988: "Hydrodynamic Forces on Pipelines – Model Tests", *Proceedings of the 7th International Conference on Offshore Mechanics and Arctic Engineering,* Houston, Texas.

Det Norske Veritas (1988): "On-Bottom Stability Design of Submarine Pipelines", Recommended Practice DNV RP-E305.

Det Norske Veritas (2007): "On-Bottom Stability of Submarine Pipelines", Recommended Practice DNV RP-F109.

Det Norske Veritas (1981): "Rules for Submarine Pipeline Systems", DNV 1981.

Det Norske Veritas (2007): "Submarine Pipeline Systems", Offshore Standard DNV OS-F101.

Holthe, K., Sotberg, T. and Chao, J.C. (1987): "An Efficient Computer Program for Predicting Submarine Pipeline Response to Waves and Current", *OTC5502, Proceedings of the 19thOffshore Technology Conference, Houston, Texas.*

Lambrakos, K.F., Remseth, S., Sotberg, T., and Verley, R.L.P. (1987): “Generalised Response of Marine Pipeline”, OTC5507, *Proceedings of the 19th Offshore Technology Conference, Houston, Texas*.

Ose, B. A., Bai, Y., Nystrøm, P. R. , Damsleth, P. A. (1999): “A Finite-Element Model For In-Situ Behaviour of Offshore Pipelines on Uneven Seabed and its Application to On-Bottom Stability”, *Proc. of the 9th Int. Conf. on Offshore and Polar Eng. (ISOPE), Brest, France.*

Pipeline Research Council International (PRCI) (2002): “Submarine Pipeline On-Bottom Stability – Analysis and Design Guidelines, Volume 1&2”, *PRCI Project Number PR-178-01132.*

Verley, R.L.P., Sotberg, T. (1992): “A Soil Resistance Model for Pipelines Placed on Sandy Soil”, *Proc. of the 11th Int. Conf. on Offshore Mechanics and Arctic Engineering (OMAE 1992), Alberta, Canada.*

Verley, R.L.P. and Lund, K. M. (1995): “A Soil Resistance Model for Pipelines Placed on Clay Soils”, *Proc. of the 14th Int. Conf. on Offshore Mechanics and Arctic Engineering(OMAE 1995), Copenhagen, Denmark .*

Wolfram Jr., W.R., Gets, J.R., Verley, R.L.P. (1987): “PIPESTAB Project – Improved Design Basis for Submarine Pipeline Stability”, *Proceedings of the 19th Offshore Technology Conference, OTC5501, Houston, Texas.*

Zeitoun, H.O., Tørnes, K., Cumming, G., Branković, M. (2008): “Pipeline Stability – State of the Art”, *Proc. of the 27th Int. Conf. on Offshore Mechanics and Arctic Eng., OMAE2008 – 57284, Estoril, Portugal.*

Zeitoun, H.O., Tørnes, K., Li, J., Wong, S., Brevet, R., Willcocks, J. (2009): “Advanced Dynamic Stability Analysis”, *Proc. of the 28th Int. Conf. on Offshore Mechanics and Arctic Eng., OMAE2009 – 79778, Honolulu, Hawaii.*