# APPLYING THE RELIABILITY ANALYSIS CONCEPT IN ON-BOTTOM STABILITY DESIGN OF SUBMARINE PIPELINES

**M.Daghigh**[1] mdaghigh@gmail.com

**S. Alagheband**[2] alagheband@pogc.ir

**H. Tahmasbi Ashtiani**[3] tahmasbi@live.com

*Pars Oil and Gas Company - No. 133, Parvin Etesami St., Fatemi Ave., Tehran, Iran*

## ABSTRACT
This paper describes a reliability based formulation to determine submarine pipeline on-bottom stability for conventional, concrete coated pipeline system used for transmission of gas and condensate in South Pars Field of Persian Gulf. The instability is defined as a condition where the water will push the pipeline but the movement will not necessarily cause a failure. The Stability conditions are usually checked during installation and operating conditions. The submarine pipeline on-bottom stability design is carried out for different water depth and environmental conditions. Moreover the effect of embedment of the pipeline on the seabed in considered with reduced hydrodynamic coefficients as detailed in the literature [1,3,4,5]. The evaluation of reliability requires an understanding of design methodologies and what design parameters can cause and increase unreliability in a designed system. The on-bottom stability limit state formulation is based on DNV-RP-E305 recommended practice which is widely used in engineering analysis in submarine pipeline industries [2].

**KEYWORDS:** Subsea Pipeline – Reliability Analysis – On-bottom Stability

## INTRODUCTION
This paper is concerned with the application of design standards for subsea pipelines. The subsea pipelines have been constructed for different purposes such as transformation of crude oil and gas transmission. Some countries use their own standards as guideline for design of subsea pipeline. The most popular used standard for on-bottom stability analysis is DNV design standard which will be adopted in this paper.

Pipelines installed on the seabed are subjected to hydrodynamic forces. Waves and steady currents that are characteristics of all offshore areas subject the pipeline on the seabed to drag, inertia and lift forces. For lateral stability, the pipeline resting on the seabed must resist these forces and at a minimum be at equilibrium.

Drag and inertia forces act together laterally on the pipeline, tending to move the pipeline. Lift force acting vertically tends to effectively reduce the submerged weight of the pipeline. The sliding friction between pipeline and seabed soil provide the resistance of the pipeline on the seabed. In general, the larger the submerged weight, the higher the frictional resistance. However, latter methods for

[1] Assistant Professor
[2] MSc, Mech. Eng., Director Engineering and Construction
[3] MSc, Civil Eng., South Pars Project, Phases 6,7&8

determining the stability include the depth of embedment of the pipeline. With considering the pipeline embedment, additional resistance is provided by the soil and this effect reduces the required submerged weight of the pipeline [3,4,5].

## ANALYTICAL MODEL OF RELIABILITY ANALYSIS

In mathematical sense, the determination of the probability of failure comes down to calculating an n-fold integral, n being the number of basic variables. In integration of the limit state function requires the probability knowledge for the resistance and load parameters and also by application of modern computers, it is found that the integration of this differential equation consumes a lot of *CPU* time even for the simple engineering problems. However there are two convenient approximate methods for the evaluation of this integral which are called the *transformation methods* and the *simulation methods*. In the transformation method, the original integral is transformed to the integral in boundary of independent standard normally distributed variables. The first transformation is used for the transformation of correlated basic variables to uncorrelated variables that often requires the determination of *Jacobian* matrix. The second transformation is usually carried out by the *Rosenblatt* transformation that is used to find the independent standard normal varieties from the initial probability density functions (note that they are not necessarily normally distributed). By this formulation, the safety index can be evaluated by the so-called *Hohenbichler* algorithm.

The transformation methods are often split to *FORM* and *SORM* methods. In the next section, we will discuss the linear transformation methods (or a so-called *FORM* procedure). It should be emphasized that the use of *FORM/SORM* methods does require some caution, insight and experience in reliability computation. Among the Level II reliability methods, in the mean-value first order second-moment method (MVFOSM) the linearization of the limit state function takes place at the mean value (MV). The method is formulated on the basis of linear approximation of failure surface and only the first order terms are retained in Taylor series expansion. For application of MVFOSM method, the non-normal distributions of the variables should be converted to equivalent normal distributions.

However, for non-linear safety margins, an equivalent and approximate failure function has to be defined by linearization of the safety margin function. The advanced first order second-moment method (AFOSM) has been introduced by Hasofer and Lind (1974) and this approach is implemented by expanding the safety margin in a Taylor series about the linearization point. Such as the MVFOSM method, the point which corresponds to the shortest distance from equivalent linear failure surface is referred to as design point or checking point and the distance $\beta$ (safety index or the corresponding failure probability) can be determined by an iterative procedure. More details about these methods can be found in Ditlevsen and Madsen (1996) and Nowak and Collins (2000) [8,9].

## RELIABILITY ANALYSIS APPROACH AND LIMIT STATE CATEGORIES:

DNV as pioneer Norwegian Company prepared code of practices for submarine pipelines based on Load and Resistance Factor Design (LRFD) method and applying the reliability analysis concept [12,13]. Based on these codes, typical Limit States for a pipeline may be classified in one of the following limit states categories: Serviceability Limit State (SLS), Ultimate Limit Sate (ULS), Fatigue Limit State (FLS) and Accidental Limit State (ALS) Category. The damage due to loss of sufficient weight coating is considered as the SLS limit state and depending on three different safety classes as low, normal and high, the annual probability of failure per pipeline is restricted to $10^{-2}$, $10^{-3}$ and $10^{-3}$ respectively. These safety classes are used for temporary and in-service conditions in different location classes based on DNV design rule.

Determination of appropriate target safety levels is fundamental to the process of developing new design criteria through the application of reliability methods. A target safety level is defined as the maximum acceptable failure probability level for a particular limit state design to be accepted. In this regard, the nominal target failure probability levels have to be calibrated against identical or similar pipeline designs that are known to have adequate safety. The evaluation of the target safety level for pipelines should primarily be based on the implied safety in currently accepted design practice, using uncertainty measures representative at the time when the code was made. Further, the nature of failure and the actual consequence potential in terms of hazard to human health and safety, damage to the environment, economic losses, and the amount of expense and effort required to reduce such hazard potential should be take into account. With no implicit safety level available, the rules provide recommendations on target failure probabilities versus safety class and limit state category. The basis for the values of safety factor relies on a conservative assessment of implied safety in current accepted design practice guided by accident statistics and engineering judgment.

**PIPELINE ON-BOTTOM STABILITY LIMIT CALCULATION METHOD**

The design of the 32” & 4.5” South Pars Phase 11 Sealines will be performed at relevant pipeline life phases and relevant return periods, combining wave and extreme current loading, for different (Serviceability, Ultimate, Fatigue and Accidental) limit states based on DNV-OS-F101. Two sealines, each composed by a 32” and a 4.5” (piggyback) pipelines, will link the onshore plant to each offshore platforms. The 32” pipelines are used for transmitting the gas and condensate from each platform to the onshore plant. The 4.5” piggyback pipelines are used to convey MEG + CI flows from the onshore plant onto each platform.

On-bottom stability assessment of the sealines includes both lateral and vertical stability. The aim of this verification is to check whether the submerged weight of the offshore sealines complies with the stability criteria specified by the design code or not. These criteria consist of :
- Sealines shall have a minimum specific density on soil,
- Sealines shall be laterally stable under the action of wave and current,
- Sealines shall not sink into the soil when laid on soft clay or sediments.

The following design conditions shall be used :
• Pipelines empty, non corroded, during installation (after laying),
• Pipelines containing pressurized fluids at minimum density, assumed 10% corroded, during operation.

The vertical stability and specific density (stability factor) checks have also been performed for the 32” pipeline. None are governing the concrete thickness calculated based on the vertical stability checks. Therefore only the lateral stability of pipelines is addressed in this paper.

In order to comply the above requirements, guidelines have been described in DNV recommended practice for On-Bottom Stability Design [2]. This method is based on a static stability approach, which ties the classical static design approach to the generalized stability method through a calibration of the classical method with the generalized stability results. The calibration factor ($F_W$) included was developed from pipelines designed with a lateral displacement of up to 20 m, assuming that there is no exceeding of the allowable bending strain at anytime with the pipeline. A safety factor of 1.1 is inherent in the calibration factor $F_W$.

Based on the reliability analysis method, the lateral stability is given by the following equation;

$$g(R,S) = R - S = \mu(W_S - F_L) - (F_D + F_I) = 0 \qquad (1)$$

Where:

$W_S$ : Submerged weight of pipeline including concrete weight coating

$\mu$ : Lateral soil pipe friction factor as function of soil shear strength in clay seabed and soil internal friction angle in sandy seabed

$F_L$, $F_D$, $F_I$ : Lift force, drag force, inertia forces are defined based on Morison equation

The wave-induced particle velocities and accelerations used in the wave forces calculations are the significant values, as detailed below:

$$F_L = 0.5\rho_w D_1 C_L (U_S.\cos\theta + U_C)^2 \quad (2)$$

$$F_D = 0.5\rho_w D_1 C_D \mid (U_S.\cos\theta + U_C) \mid .(U_S.\cos\theta + U_C) \quad (3)$$

$$F_I = \rho_w C_M .A_S.\sin\theta.(\pi D_2^{\ 2})/4 \quad (4)$$

where

$\rho_w$ : Mass density of water

$D_1$ : Equivalent diameter for drag and lift force calculation,

$D_2$ : Equivalent diameter for inertia force calculation,

$C_L$ : Lift force coefficient,

$C_D$ : Drag force coefficient,

$C_M$ : Inertia force coefficient,

$U_S$ : Significant near-bottom velocity amplitude perpendicular to the pipeline,

$U_C$ : Current velocity perpendicular to the pipeline,

$A_S$ : Significant acceleration perpendicular to the pipeline,

$\theta$ : Phase angle of the hydrodynamic force in the wave cycle.

The DNV RP E305 simplified stability analysis used for the on-bottom stability analysis is based on the following principles:

• Safety factor of 1.1, included in the calibration factor Fw.
• Constant hydrodynamic coefficients (Table 1),
• Lateral displacement of up to 20 m allowed (Table 2),
• Empirical friction factors for clay soil, based on non-dimensional shear strength of the soil, taking into account pipeline penetration (cf. DNV RP E305 Fig. 5.11). Table 1 below provides the basic values of hydrodynamic coefficients to be used in numerical analysis. In some cases, they will be further modified to incorporate some particular effects such as:
• The pipeline embedment that occurs after the pipeline is filled with water (hydrotest and operation) : The amount of embedment in soft soils (in the order of 5%) will be evaluated, and reduced hydrodynamic coefficients as per Ref. [3,4,5] will be used,
• The protective effect of the trench at shore approach area, if significant will also be used if significant,
• The hydrodynamic contribution of the piggy-back line.

**Table 1 – Basic Hydrodynamic Coefficients**

| **Parameter** | **Symbol** | Pipeline resting on seabed |
|---|---|---|
| **Drag Coefficient** | $\mathbf{C_D}$ | 0.70 - 1.20 (1) |
| **Lift Coefficient** | $\mathbf{C_L}$ | 0.90 |
| **Inertia Coefficient** | $\mathbf{C_M}$ | 3.29 |

(1) : $C_D$=1.20 for sub-critical and critical flow regime, i.e. $R_e < 3\times10^5$ and Current to Wave velocity ratio: $M \geq 0.8$.

Based on DNV RP E305, and unless otherwise specified, the allowable maximum lateral displacement of the sealines are:

**Table 2 – Allowable maximum lateral displacement**

| Location | Allowable maximum lateral displacement |
|---|---|
| Zone 1 | 20 m |
| Zone 2 (1) | 0 m |

(1) : Zone 2 criteria is applicable in a 500 m zone from the main facilities, and also in particular pipeline route areas such as crossings, expansion loops and tie-in points.

For pipelines slightly or partly embedded, the soil contributes directly to the lateral resistance. The lateral stability verification will be performed:

• In installation conditions (period of time after installation when the pipeline is resting on the seabed or in an open trench prior to burying or commissioning) : the pipes will be assumed airfilled and non-corroded, and the applicable conditions are the 1-year environmental conditions.

• In operating conditions: the pipes will be assumed full of pressurized fluid at minimum density and 10% corroded. The most severe of the following conditions will be selected, as per DNV RP E305 [2]:

- 100 year wave + 10 year current,
- 10 year wave + 100 year current.

## HYDRODYNAMIC CONTRIBUTION OF THE PIGGYBACK PIPELINE

The stability verification is performed for the bundle composed of the 32” line and the 4.5” line installed on top. The contribution of the 4.5” line to the hydrodynamic forces is calculated as detailed below :

The diameter modeled in hydrodynamic force calculation is the diameter of the 32” alone. The contribution of the 4.5” is included by considering enhanced hydrodynamic coefficients for the 32”. These equivalent hydrodynamic coefficients represent two effects:

• The increased screen area due to the presence of the 4.5”;

• The disturbance of the flow due to the presence of the 4.5”.

A nominal gap of 75 mm between the two pipes is considered.

The total drag force is the sum of the drag force on the 32” and the drag force on the 4.5”.

An equivalent drag coefficient is calculated, which includes:

• the intensification of the wave and current particle velocity on the 4.5”, due to the presence of the 32” (factor “$k_1$”)

• the intensification of the wave and current particle velocity close to the 32”, due to the presence of the 4.5” (factor “$k_2$”)

The drag force is proportional to the square of particle velocities, hence the equivalent drag coefficient is expressed as follows :

$$C_{Deq} \times D_{32"} = (C_{D32"} \times k_2^{\,2} \times D_{32"}) + (C_{D4.5"} \times k_1^{\,2} \times D_{4.5"}) \tag{5}$$

with :

$C_{Deq}$ = equivalent drag coefficient

$C_{D32"}$ = drag coefficient of the 32”

$C_{D4.5"}$ = drag coefficient of the 4.5” ($C_{D4.5"}$ = 1.2), as per DNV-RP-305.

$D_{32"}$ = diameter of the 32”

$D_{4.5"}$ = diameter of the 4.5”

$k_1$ = intensification factor of the velocities at 4.5” centre location

$k_2$ = intensification factor of the velocities at 32” centre location

When partial embedment of the 32” is taken into account, $C_{D32"}$ is as specified in Table 3.

**Table 3 – Embedded hydrodynamic coefficients**

| Coefficient | Without burying | With 50 mm burying depth |
|---|---|---|
| Drag Coefficient($C_D$) | 1.2 ($Re < 3\times10^5$)<br>0.7 ($Re > 3\times10^5$) | 1.08 ($Re < 3\times10^5$)<br>0.63 ($Re > 3\times10^5$) |
| Lift Coefficient ($C_L$) | 0.9 | 0.86 |
| Inertia Coefficient ($C_M$) | 3.29 | 2.96 |

The intensification factors are calculated as follows:
For example: intensification factor at 4.5" location due to presence of 32":
$K_1=V_1/V_S$ with: $V_1=V_S(1+r^2/d^2)$
and:
$V_S$ = Undisturbed velocity
r = Radius of the 32" pipe
d = Distance from centerline of 32" to centerline of 4.5".

A symmetrical expression is used to calculate $k_2$. The following values are obtained:
$k_1$ = 1.62 (velocity intensification at the 4.5" location due to the presence of the 32"),
$k_2$ = 1.008 (velocity intensification at the 32" location due to the presence of the 4.5").
The equivalent hydrodynamic coefficients obtained are summarized in Table 5 and Table 6.
For the lift force, the Figure A5 of the code DnV 76 is used [6]:
For the 32" : $C_L$ = 0.9 (non-embedded) or 0.86 (embedded)
For the 4.5": H/D = 0.075 / 0.1143 = 0.656 ⇒ $C_{L4.5"} = 0$
The method is the same as for the drag force, however $C_L = 0$ for the 4.5" line.

$$C_{Leq}\times D_{32"} = (C_{L32"}\times k_2{}^2\times D_{32"}) + 0 \tag{6}$$

For the inertia force, the equivalent inertia coefficient is determined as follows:

$$C_{Meq}\times D_{32"}{}^2 = (C_{M32"}\times D_{32"}{}^2) + (C_{M4.5"}\times D_{4.5"}{}^2) \tag{7}$$

with :
$C_{Meq}$ = equivalent inertia coefficient
$C_{M32"}$ = inertia coefficient of the 32" = 3.29 (non embedded) or 2.96 (embedded)
$C_{m4.5"}$ = inertia coefficient of the 4.5" = 2.2
(as per DnV 76, Fig A3 for H / D = 75 / 114.3 = 0.656)
D32" = diameter of the 32"
D4.5" = diameter of the 4.5"

**Table 5 – Pipeline non-embedded**

| | Basic Coefficient | Equivalent coefficient (effect of 4.5" line) |
|---|---|---|
| Drag Coefficient (CD) | 1.2 ($Re < 3\times10^5$)<br>0.7 ($Re > 3\times10^5$) | 1.65<br>1.14 |
| Lift Coefficient (CL) | 0.9 | 0.92 |
| Inertia Coefficient(CM) | 3.29 | 3.32 |

**Table 6 – Pipeline embedded (50 mm)**

| | Basic Coefficient | Equivalent coefficient (effect of 4.5” line) |
|---|---|---|
| Drag Coefficient(CD) | 1.08 (Re < $3\times10^5$)<br>0.63 (Re > $3\times10^5$) | 1.53<br>1.07 |
| Lift Coefficient(CL) | 0.86 | 0.87 |
| Inertia Coefficient(CM) | 2.96 | 3.00 |

Note: These coefficients are applicable in operation conditions, except at shore approach.
At shore approach, the pipeline is calculated without considering any partial burying, because the soil is firmer (sand).

It should be noted that, data available from Computational Fluid Dynamic (CFD) assessment of piggyback pipelines has generally implied that drag is significantly underestimated using the equivalent diameter (ED) approach, and that lift and inertia are overestimated. The use of equivalent diameter approach might be considered appropriate in a static type analysis given the track record of using the equivalent diameter approach in this type of analysis [1].

## PIPELINE EMBEDMENT

Pipeline embedment is an important parameter to consider when assessing soil resistance and hydrodynamic sheltering. Embedment will provide sheltering from hydrodynamic loads as discussed in the previous section and will also contribute in increasing passive resistance. Pipeline embedment beings during installation and may increase during the design life of the pipeline. During pipelay operation, a pipeline will experience a force at the touch down point much higher than its submerged weight, which will cause the pipeline to embed.

Several approaches have been suggested for evaluating pipeline initial embedment during installation. An empirical approach was proposed by Verley and Lund to evaluate initial embedment on clay soil based on test data in PIPESTAB project [1,2,3]. The total soil resistance consists of a frictional term $F_f$ and an additional resistance due to pipeline embedment, $F_p$ given by:

$$F_R = F_f + F_p \tag{8}$$

In which $F_R$ is the total soil resistance consists of a frictional term $F_f$ and an additional resistance due to pipeline embedment $F_p$ given by the following empirical relations:

$$\frac{F_p}{D.s_u} = 4.13G^{-0.392}\left(\frac{z}{D}\right)^{1.31} \tag{9}$$

And

$$\frac{z}{D} = 0.0071.(S.G^{0.3})^{3.2} + 0.062.(S.G^{0.3})^{0.7} \tag{10}$$

Where:
z= seabed penetration (m)
$S = F_c /(D.s_u)$
$G = s_u /(D.\gamma')$
$F_c$= vertical contact force (kN/m)
D= pipeline external diameter (m)
$s_u$= undrained shear strength (kPa)
$\gamma'$= submerged soil density (kN/m$^3$).
The Verley and Lund formulation is based on curve fitting to data with $(S.G^{0.3})<2.5$. An alternative formulation (linear one), said to be valid for all values of $(S.G^{0.3})$ is given by:

$$\frac{z}{D} = 0.09.(S.G^{0.3})$$

## NUMERICAL ANALYSIS

Data are taken from South Pars field development Projects. The 100 year significant wave height and peak period plus 10 year current are considered for the operating conditions.

The 1-year significant wave height and peak period plus 1-year current are considered for the installation and hydrotest conditions. Pipeline is assumed to be empty during installation and filled with hydrates water (assumed seawater) during hydrotest.

**Table 7 - Data for minimum pipeline submerged weight [10,11]**

| **Parameter** | **Symbol (unit)** | **Case 1.1** | **Case 3.3** | **Case 4.3** |
|---|---|---|---|---|
| **Significant Wave Height** | **Hs(m)** | 2.2 | 2.4 | 2.7 |
| **Spectral Peak Period** | **T(s)** | 7.5 | 7.5 | 7.5 |
| **Mean Water Depth** | **d(m)** | 8 | 18 | 25 |
| **Zero upcrossing period** | **Ts (s)** | 6.6 | 6.6 | 6.6 |
| **Current Velocity at surface** | **Vr (m/s)** | 0.5 | 0.4 | 0.5 |
| **Current Velocity at 1 m above seabed** | **V current** | 0.37 | 0.26 | 0.32 |

The deterministic parameters of geometrical characteristics such as the steel pipe thickness, the corrosion coating density and the density of each section are given in the following Table:

**Table 8- Data for pipeline submerged weight [10,11]**

| | |
|---|---|
| **Diameter of steel, D** | 0.8128 (m) |
| **Pipe wall thickness, t** | 0.0206 (m) |
| **Anti-Corrosion coating thickness, $t_{cor}$** | 0.0042 (m) |
| **Concrete coating thickness, $t_{conc}$** | 50 mm or 90 mm depending to pipe location |

**Table 9 - Density of Pipeline Sections, (kg/m$^3$)**

| | |
|---|---|
| **Steel, $\rho_{st}$** | 7850 |
| **Anti-Corrosion coating (Tar Enamel), , $\rho_{cor}$** | 940 |
| **Concrete Coating, , $\rho_{conc}$** | 2923 |
| **Content, , $\rho_P$** | 840 |
| **Seawater, , $\rho_w$** | 1025 |

Concrete coating thickness is considered as random variable with $t_{conc}$ = 50 mm, 60 mm, 70 mm, 80 mm and 90 mm and standard deviation equal to 0.02 its mean thickness. The submerged weight can be computed as the weight of the product, the steel material, the corrosion weight, the concrete coating minus the buoyancy of the pipe. The objective submerged weight is calculated with the following formula:

$$W_S = 0.25\pi g\{[(D+2t_{cor}+2t_{con})^2-(D+2t_{cor})^2]\rho_{con}+[D^2-(D-2t)^2]\rho_{st}+[(D-2t)^2]\rho_P+ [(D+2t_{cor})^2-D^2]\rho_{cor}-[(D+2t_{cor}+2t_{conc})^2]\rho_w \tag{11}$$

The distribution parameters of hydrodynamic and geometrical variables are given in the following table:

**Table 10: Random Variables**

| **Variable Name** | $t_{concrete}$ | $C_L$ | $C_D$ | $C_M$ | H |
|---|---|---|---|---|---|
| **Variable type** | Normal | Normal | Normal | Normal | Rayleigh |
| **Mean** | 50 mm-90 mm | 0.9 | 0.7 | 3.29 | Conditional on $H_S$ |
| **Standard Deviation** | 0.02 μ | 0.2μ | 0.2μ | 0.2μ | Conditional on $H_S$ |

It is supposed that dependency of hydrodynamic coefficients ($C_D$ , $C_L$ and $C_M$) to water particular velocity and pipe diameter could be neglected and instead we could assume that these coefficients are independent variables with standard deviation about 20% of mean values. To obtain water particular velocity and acceleration process at the seabed from the static design wave method, the Linear Airy wave theory is adopted for the evaluation of statistics. In each case, the phase angle between velocity and the acceleration waves is selected to maximize the required submerged weight (or to minimize the stability factor). The probabilistic treatment of seabed soil have been neglected in the present study.

## RESULTS

### 1- Non-embedded pipeline

In this section, the reliability analysis results are presented for limit state functions based on DNV rule and having five different thicknesses for submarine pipelines. The reliability indices and probability of failure based on DNV standard is given in the following table:

**Table 11: Reliability indices and Failure Probabilities for three critical cases**

| Critical Case No. | | 1.1 | 3.3 | 4.3 |
|---|---|---|---|---|
| Water Depth (m) | | 8 | 18 | 25 |
| Significant Wave Height (m) | | 2.2 | 2.4 | 2.7 |
| Concrete Coating Thickness= 50 mm | Reliability index, $\beta$ | 0.8480 | 1.2851 | 2.1527 |
| | Failure Probability, $P_f$ | $1.982\times10^{-1}$ | $9.940\times10^{-2}$ | $1.570\times10^{-2}$ |
| Concrete Coating Thickness= 60 mm | Reliability index, $\beta$ | 1.8091 | 2.2494 | 3.3631 |
| | Failure Probability, $P_f$ | $3.520\times10^{-2}$ | $1.220\times10^{-2}$ | $3.853\times10^{-4}$ |
| Concrete Coating Thickness= 70 mm | Reliability index, $\beta$ | 2.5913 | 3.0323 | 4.3386 |
| | Failure Probability, $P_f$ | $4.800\times10^{-3}$ | $1.200\times10^{-3}$ | $7.170\times10^{-6}$ |
| Concrete Coating Thickness= 80 mm | Reliability index, $\beta$ | 3.2541 | 3.6943 | 5.1597 |
| | Failure Probability, $P_f$ | $5.688\times10^{-4}$ | $1.102\times10^{-4}$ | $1.236\times10^{-7}$ |
| Concrete Coating Thickness= 90 mm | Reliability index, $\beta$ | 3.830 | 4.2679 | 5.8689 |
| | Failure Probability, $P_f$ | $6.410\times10^{-5}$ | $9.868\times10^{-6}$ | $2.193\times10^{-9}$ |

Five reference concrete coating designs for the above three cases were created first. According to SPP11 survey results, a thin layer of medium silty is present at the surface of the sea bed in the shore approach area (up to 16 m water depth). A lateral coefficient friction of 0.7 is used in accordance with DnV-RP-E305 for this zone. For installation condition, the lateral coefficient of friction in the rest of the sea bed is conservatively assumed equal to 0.3. The reference designs are computed based on a simplified static analysis procedure. The calculated concrete thicknesses are 60 mm for water depth greater than 32 m and 90 mm for water depths below 32 m.

The effect of concrete coating is obviously observed in Failure Probability of on-bottom stability limit state for three critical cases. The reliability results are compared with a minimum target failure probability of $1\times10^{-3}$ and reliability index as order of 3.09 is selected for the on-bottom stability check. A concrete coating of 60 mm is sufficient for water depths greater than 25 m and the 80 mm concrete thickness would give satisfactory safety factor against sliding for pipelines from shore approach to water depth of 25 m.

### 2- Embedded pipeline

Based on the pipeline embedment and hydrodynamic coefficients alteration, the concrete coating thickness calculated for different critical cases and the reliability indices and failure probabilities are presented in Table 12. The effect of embedment is conservatively neglected for case 1.1 with water depth of 8 m since the pipeline will be located on a thin layer of medium sand.

**Table 12: Reliability indices and Failure Probabilities for two critical cases**

| Critical Case No. | | 3.3 | 4.3 |
|---|---|---|---|
| Water Depth (m) | | 18 | 25 |
| Significant Wave Height (m) | | 2.4 | 2.7 |
| Concrete Coating Thickness= 50 mm | Reliability index, β | 1.6575 | 2.3040 |
| | Failure Probability, $P_f$ | $4.87\times10^{-2}$ | $1.060\times10^{-2}$ |
| Concrete Coating Thickness= 60 mm | Reliability index, β | 2.7587 | 3.7440 |
| | Failure Probability, $P_f$ | $2.900\times10^{-3}$ | $9.055\times10^{-5}$ |
| Concrete Coating Thickness= 70 mm | Reliability index, β | 3.6499 | 4.8892 |
| | Failure Probability, $P_f$ | $1.312\times10^{-4}$ | $5.062\times10^{-7}$ |
| Concrete Coating Thickness= 80 mm | Reliability index, β | 4.4017 | 5.8446 |
| | Failure Probability, $P_f$ | $5.370\times10^{-6}$ | $2.539\times10^{-9}$ |
| Concrete Coating Thickness= 90 mm | Reliability index, β | 5.0523 | 6.6675 |
| | Failure Probability, $P_f$ | $2.182\times10^{-7}$ | $1.301\times10^{-11}$ |

Pipeline embedment effect on the on-bottom stability limit state formulation is considered on the assessing soil resistance and hydrodynamic sheltering phenomena. Although the pipeline embedment has been ignored in the concrete coating thickness designed by DORIS engineering, it has been found by several research studies that during pipelay operation, a pipeline will experience a force at the tough down point much higher than its submerged weight [9,10]. Moreover from the results of Zeepipe 2B pipeline project, the pipeline penetration is found much higher than what it is predicted by Verley model [1]. The penetration depth achieved by a pipeline in real life condition varied between 0.25 m-0.4 m while the calculated embedment was about 0.05 m (as also calculated for the present pipeline in South Pars Field). However using the empirical approach, the failure probability of pipeline with $t_{con.}$=80 mm would be $5.37\times10^{-6}$ about 1/100 of the failure probability of non-embedded pipeline (case 3.3). A concrete coating of 70 mm is sufficient for water depths below 25 m compared to 80 mm concrete thickness in the previous case. However for water depth greater than 25 m, a concrete thickness of 60 mm would be conservative and it would be possible to adopt 50 mm concrete thickness for this region. The on-bottom stability for different concrete coating thickness has been analyzed for two different sea-state and water depth based on Basic engineering documentation. The effect of variation in concrete coating thickness on the failure probability of on-bottom stability is shown in Figs. 1 and 2 for two severe cases 3.3 and 4.3.

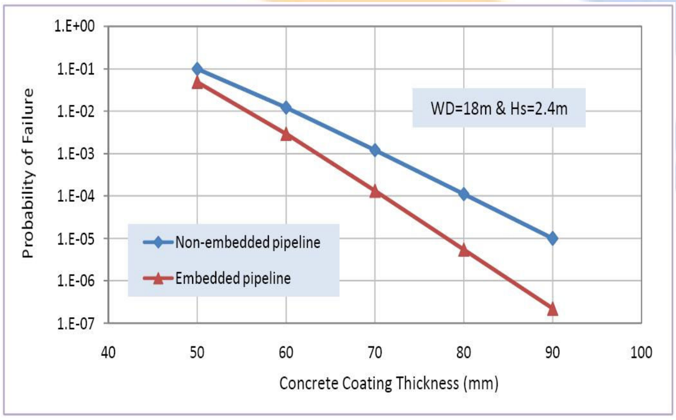


**Fig. 1 - Failure probability vs. concrete coating Thickness, Case No. 3.3**

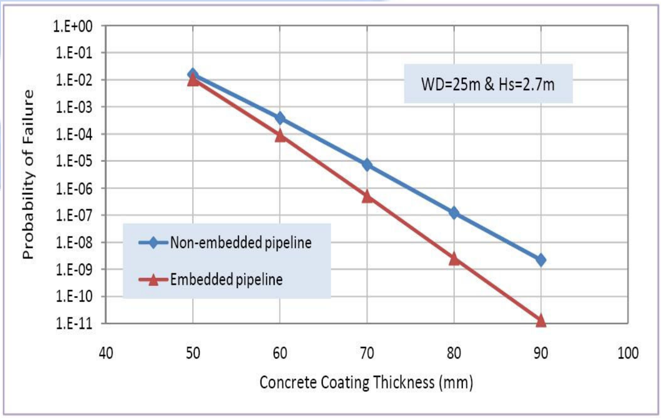


**Fig.2 – Failure probability vs. concrete coating Thickness, Case No. 4.3**

Considering the effect of pipeline embedment in seabed significantly decrease the probability of failure related to on bottom stability design. Furthermore it was found that by increasing the coating thickness, pipe embedment effect would be more notable in failure reduction point of view. In the same probability failure level, the required concrete coating thickness could decrease if designer incorporate the pipeline embedment in on-bottom stability analysis. This matter would be more considerable in less water depths. For instance in probability failure of $1\times10^{-4}$ and design case No. 3.3 (water depth of 18m), the required concrete coating thickness should be reduced by 10 mm if pipe embedment in seabed incorporated in on bottom stability design. The reduction in concrete coating thickness would be in the order of 4 mm for design case No. 4.3 (water depth of 25m).

The reliability analysis shows the relative importance of selected limit state function for failure probability estimation in on-bottom stability analysis. Within the static analysis approach, a simple force balance calculation is formulated requiring no pipeline movement on the seabed under extreme environmental conditions. However it should be kept in mind that adopting the dynamic approach requires time domain or frequency domain simulation of irregular sea-state and the nonlinear hysteric soil resistance based upon a load-displacement history taking into account the hydrodynamic damping which are all time consuming. A similar approach has been developed in recent DNV code presented in DNV-F109 which replaces the old Simplified Design Criteria of DNV RP-E305 [1]. This recent code recommends applying a Load Resistance Factor Design approach to satisfy the absolute stability criteria. In this code, it is assumed that pipeline displacement is a failure criteria.

## CONCLUSIONS

1- Pipeline on-bottom stability force criteria has been addressed with different relevant up to date studies in South Pars Field developments. The current study includes the hydrodynamic loading models on piggy-back lines, pipe embedment for the on-bottom stability criteria and the effect of water depth and sea states on the stability criteria.

2- Failure criteria of pipeline transversal stability has been studied for the optimization of the thickness of concrete coating. The conservative design approach without the effect of pipeline embedment has been investigated for three different sea-states and four different concrete coating thickness. It has been shown that the embedment of pipeline on clay type soils has important stability effect which has to be considered in on-bottom stability designs.

3- Comparing the reliability results based on non-embedded and embedded pipelines, the design concrete thickness approach in DNV-RP-E305 seems conservative and it has to be checked with the recent LRFD method proposed in DNV-F109 code. Unfortunately during this study, the writers did not have access to this recent code but it is proposed to compare the results of static stability analysis with the displacement control limit state criteria in DNV-F109.

4- Based on a model testing research program on the hydrodynamic loads of piggyback lines, significant difference in hydrodynamic loading is observed compared to an equivalent diameter approach. Data available in literature shows that the drag term is significantly underestimated using the equivalent diameter approach, and the lift and inertia forces are overestimated in this approach. This requires further research to reduce the uncertainty associated with hydrodynamic load modeling of combine pipe and piggyback line.

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