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On-bottom Stability Design of Submarine Pipelines: The Fundamentals

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ABTSRACT: Pipelines are major cost of items in the oil and gas field development. Poor on-bottom stability design may lead to fatigue, lateral and propagation buckling problems. Consequently, additional cost may be incurred during pipeline design and construction due to critical problems relating to poor design. But cost related to the on-bottom stability problem can be significantly reduced by optimizing design. This paper presents comparative review of submarine pipelines on-bottom stability design methods. Comparing absolute lateral stability, generalized lateral stability and traditional force balance methods show variation in submerged weight and effect of pipe-soil interaction on submerged weight parameters. Overall, most literatures agreed that pipelines lateral stability can be increased by increasing porosity of soil, soil embedment and submerged weight. But steel wall and concrete thicknesses are the major parameters used to establish lateral stability of submarine steel pipelines. Therefore, providing an in depth understanding of on-bottom pipeline stability design is necessary to prevent pipeline movement during operation, its associated risks and optimized design.

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Large deposit of oil and gas discovery under the seabed has led to offshore structure development to support exploration, production and transportation of hydrocarbon. Carbon steel pipelines, which are installed on the seabed from wellheads to tieback installation, have proven to be the most reliable and efficient means of transporting produced hydrocarbon to onshore or offshore location for further processing (Iyalla *et al*., 2010). Over 90000km length of submarine pipelines have been installed since the drilling of first oil well at the seabed in 1947 at the Gulf of Mexico with an average of 5000km pipeline length added every year (Ameh, 2009). In deep water, pipelines are not trenched or buried but laid directly on the seabed since there are no threat of drop object and over trawling (Merifield *et al*., 2009). However, fraction of pipeline diameter laid on the seabed penetrates the seabed because of self-weight and contact stress at torch down during installation (White and Randolph, 2007). There are several subsea pipeline design codes namely BS8010, DNV-RP-F109, ISO-13623, API 1111 and American Gas Association (AGA), but the most widely used in on-bottom design stability are the DNV-RP-F109 and AGA guidelines (Tornes *et al*., 2009; Palmer and Roger, 2008). The DNV-RP-F109 recommends three on-bottom design methods, namely dynamic lateral stability, generalized lateral stability and absolute lateral static stability. Dynamic lateral stability method is a time domain simulation of pipe response that permit displacement not greater than half pipeline diameter as well as estimates lateral displacement which considers time varying force of hydrodynamic forces (DNV, 2010; Bryndum *et al*., 1992). The dynamic analysis method involves full modeling of pipeline resting on seabed, soil resistance, hydrodynamic forces, boundary condition and structural response. The method is mainly used in critical areas of pipelines such as riser tie-in points and pipeline crossings, reanalysis of existing pipelines and where detailed structural responses are required (Gao *et al*., 2006; DNV, 1988). Notwithstanding, the dynamic lateral method is not widely used because of finite element analysis complexity and the comparative advantage of the other two methods to provide detail quantity of concrete coating requirement in the design approach (Tornes *et al*., 2009).

The American Gas Association (AGA) design code that is software based, provide three levels of design philosophies for on-bottom stability criteria. AGA Level 1 analysis adopt the traditional 2D force balance stability method to compute static stability of unburied pipeline against vertical and lateral displacements under current and wave loadings (AGA, 2000). Hence, inertial, lift and drag forces as well as soil restraining effect are considered in Level 1 analysis. Whereas,

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AGA Level 2 adopt quasi-static analysis to compute submerge pipe weight to meet design criteria. The level 2 design Philosophy permits limited pipeline movement and involves computation of environmental loads effects, soil resistance force and pipe lateral displacement for design check (AGA, 2002). On the other hand, the Level 3 is dynamic and time domain based on 2-D finite element model which takes into consideration pipeline motion in horizontal direction and bending deformation effect on axial forces during modeling (AGA, 2002; Allen *et al*., 1989). The aim of this work is to present systematic and analytical design methodologies for concrete and submerge unit weights required to withstand action of combined environmental and functional loads for carbon steel pipeline installation. One of the benefits of the study includes improving in-depth understanding of relevant empirical design models for on-bottom stability analysis of submarine pipelines. The study may further guide pipeline engineers and programmers to develop simple excel and software that could be cheaper than current AGA and PONDUS software that are very expensive.

Basic Design Data: Basic metocean data for 100-year return condition in Forcados and Escravos fields of Nigeria in Gulf of Guinea region are presented in Table 1. Typical pipeline installation data and parameters in Table 2 are required design basis to estimate minimum concrete wall thickness and submerged weight to establish on-bottom stability design criteria.

Table 1: 100- years return metocean data

Water depth (m)	Wave and current angle to pipe axis (deg)	Maximum wave height (m)	Wave period (sec)	Current velocity (m/s)
120	90	9.2	21.2	0.45
100	90	11.4	21.2	0.51
80	70	13.2	21.2	0.57
60	75	15.7	21.5	0.51
40	90	18.3	21.7	0.62
20	60	20.5	22.2	0.68

Table 2: Pipeline and soil data

Description	Unit
Steel pipe outer diameter	m
Wall thickness	m
Pipe internal diameter	m
Fusion bonding export coating thickness	m
Density of steel	Kg/m^3
Density of sea water	Kg/m^3
Density of pipe content	Kg/m^3
Density of fusion bonding expoxy	Kg/m^3
Density of concrete coating	Kg/m^3
Density of sand soil (medium)	Kg/m^3
Clay soil shear strength	Mpa
2-Layer propethyle coating thickness	m
Density of 2-layer propethyle coating	m

Absolute Static Stability: Absolute static stability is one of the methods in establishing subsea pipeline on-bottom stability design criteria. The absolute lateral static method does not permit pipeline movement (DNV, 1988). Unlike the generalized lateral stability which does not take into consideration soil effect at the seabed, the absolute lateral static considers soil effect associated with load reduction by penetration, passive resistance force and frictional coefficient (Yu *et al*., 2013). Design equations which determine pipeline required submerged weight and concrete thickness that satisfy absolute static stability design criteria can be expressed as (DNV: 2010):

$$\gamma_{sc}\left(\frac{F_y^* + \mu F_z^*}{\mu * W_s + F_R}\right) \le 1.0 \quad (1)$$

$$\gamma_{sc}\left(\frac{F_z^*}{W_s}\right) \le 1.0 \quad (2)$$

Where, γ_{sc} = safety factor (= 1.5) ; μ = frictional coefficient (= 0.5); F_y^* =peak horizontal load; F_z^* = peak vertical load, F_R = passive resistance.
Submerged weight per meter length, W_s can be expressed as (DNV, 1981):

$$W_s = \left(W_{cs} + W_{ep} + W_{pp} + W_c + W_I\right) - B \quad (3)$$

Where, W_{cs} = carbon steel weight per unit length; W_{ep} = epoxy coating weight per unit length; W_{pp} = 2-layer propethyle weight per unit length; W_c = concrete coating weight per unit length; W_I = weigh of content per unit length and B = pipe lift weight (buoyancy). From equation (3), each of the parameter is further expressed as follow:

$$W_{cs} = \pi(D_o - t)t\rho_s \quad (4)$$

$$W_{ep} = \pi\left(D_o + t_{ep}\right)t_{ep}\rho_{ep} \quad (5)$$

$$W_{pp} = \pi\left(D_o + 2t_{ep} + t_{pp}\right)t_{pp}\rho_{pp} \quad (6)$$

$$W_c = \pi\left(D_o + 2t_{ep} + 2t_{pp} + t_c\right)t_c\rho_c \quad (7)$$

$$W_I = \pi\frac{D_i^2}{4}\rho_I \quad (8)$$

$$B = \pi\frac{D_o^2}{4}\rho_{sw} \quad (9)$$

where, D_o = pipe outer diameter; D_i = pipe internal diameter; t = pipe wall thickness; ρ_s = density of steel pipe; ρ_{ep} = density of fusion bonded epoxy coating; t_{ep} = thickness of fusion bonded epoxy coating; ρ_{pp} = density of 2-layer propethyle coating; t_{pp} = thickness of 2-layer propethyle coating; t_c = concrete coating thickness; ρ_c = density of concrete coating; ρ_I = density of content and ρ_{sw} = density of sea water. From equation (1), the peak horizonal load, F_y^* and peak vertical load F_z^* are expressed as follows:

$$F_z^* = r_{tot,z}\frac{1}{2}\rho_w D_o C_z^*(U^* + V^*)^2 \quad (10)$$

$$F_y^* = r_{tot,y}\frac{1}{2}\rho_w D_o C_y^*(U^* + V^*)^2 \qquad (11)$$

$\rho_w =$ density of water; $C_z^* =$ peak vertical load coefficient; $C_y^* =$ peak horizontal load coefficient; $r_{tot,z}$ and $r_{tot,y} =$ total load reduction factor due to pipe soil interaction in vertical and horizontal directions respectively, and are given as:

$$r_{tot,z} = r_{pem,z}\, r_{pen,z} \qquad (12)$$
$$r_{tot,y} = r_{pem,y}\, r_{pen,y} \qquad (13)$$

Where, load reduction factor due to penetration in vertical and horizontal directions ($r_{pen,z}$ and $r_{pen,y}$) are given as:

$$r_{pen,z} = 1.0 - 1.3\left[\frac{z_p}{D_o} - 0.1\right] \quad \text{if} \geq 0.0 \qquad (14)$$
$$r_{pen,y} = 1.0 - 1.4\left[\frac{z_p}{D_o}\right] \quad \text{if} \geq 0.3 \qquad (15)$$

But load reduction factor due to permeable seabed in vertical, $r_{pem,z} = 0.7$ and horizontal, $r_{pem,y} = 0.7$ under the assumption of nonpermeable seabed. While the total penetration of seabed, $z_p = z_{pi} + z_{pm}$. Where, z_{pi} =initial penetration and z_{pm} =penetration due to pipe movement.

The wave induced water particle velocity (velocity from wave of water particle), U^* which is a design single oscillation velocity amplitude is determined from equation (16):

$$U^* = \frac{1}{2}\left[\sqrt{2In\tau} + \frac{0.5772}{\sqrt{2In\tau}}\right]U_s \qquad (16)$$

Where, design spectral velocity amplitude, U_s can be calculated from graph of given Figure 1.

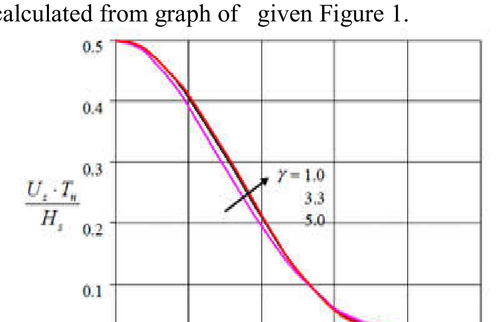

Fig 1: Spectral flow velocity amplitude at seabed (DNV, 2010)

The design spectral velocity amplitude, U_s can equally be obtained analytically with equation (18):

$$U_s = 2\sqrt{\int_o^\infty S_{uu}(\omega)d\omega} = 2\sqrt{M_o} \qquad (18)$$

Where, M_o = spectral movement of order zero. While wave induced velocity spectrum at the seabed, $S_{uu}(\omega)$ may be obtained through a spectral transformation using first order theory (Hassel et al, 1973):

$$S_{uu}(\omega) = G^2(\omega)\, S_{nn}(\omega) \qquad (19)$$

Where, $G =$ transfer function that transforms area surface elevation to wave induced velocity at seabed and is computed from equation (20):

$$G(\omega) = \frac{\omega}{\sinh(kd)} \qquad (20)$$

Where, $d =$ water depth; $\omega =$ circular wave frequency of wave motion $(= 2\pi/T)$; and k is wave number defines as:

$$k = \frac{2\pi}{\lambda} = \frac{\omega^2}{g} \qquad (21)$$

Where, $\lambda =$ wavelength and g =acceleration due to gravity. From equation (19), S_{nn} represent energy spectrum of wind generated sea with Fetch limitation, has been described by Pierson-Moskowitz $(P - M)$ and Joint North Sea Wave project (JONSWAP). The JONSWAP spectrum model takes the form:

$$S_{nn}(\omega) = \frac{\alpha g^2}{\omega^5} e^{\left[-1.25\left(\frac{\omega_p}{\omega}\right)^4\right]} \gamma^{e^{\left[\frac{-0.5(\omega-\omega_p)^2}{2\sigma^2\omega_p^2}\right]}} \qquad (22)$$

Where, $\sigma =$ shape parameter ($\sigma = 0.07$ if $\omega \leq \omega_p$; $\sigma = 0.09$ if $\omega > \omega_p$; $\gamma =$ peakedness$(= 1 - 6)$. But based on wind field velocity and fetch limitation, X_o; average value of $\gamma = 3.3$; $\alpha =$ generalized Philip constant $(= 0.0081)$ when X_o is unknown; $\omega_p =$ peak wave frequency $\left(= 2\pi/T_p\right)$, where T_p =peak wave period.

From equation (17), $\tau =$ number of oscillation in bottom velocity$(= T/T_u)$, where, $T_u =$ Zero up crossing of oscillation period $\left(= 2\pi\sqrt{M_o/M_2}\right)$ and may be obtained from Fig 2. $M_2 =$ Spectral moment of order two$\left(M_2 = \int_o^\infty S_{uu}(\omega)d\omega\right)$. From equations (10 – 11), steady current velocity relative to design oscillation (velocity from current of water particle), V^* may be computed using equation (23) or equation (24):

$$V(z) = V(z_r)\frac{In(z+z_o) - Inz_o}{In(z_r+z_o) - Inz_o}\sin\theta_c \qquad (23)$$

$$V_c = V_c(z_r)\left[\frac{\left(1+{}^{z_o}/_{D_o}\right)In\left({}^{z_o}/_{D_o}+1\right)-1}{In\left({}^{z_r}/_{z_o}+1\right)}\right] \qquad (24)$$

where, $z =$ vertical distance from seabed; $z_o =$ seabed roughness ($= 1.10^{-5}$ for sand); $z_r =$ reference height over seabed entire depth; $V(z_r) =$ velocity of reference height; $V_c(z_r) =$ current velocity at reference height over seabed entire depth; $V(z) =$ current velocity at vertical distance from seabed and $V_c =$ mean current velocity over pipeline diameter.

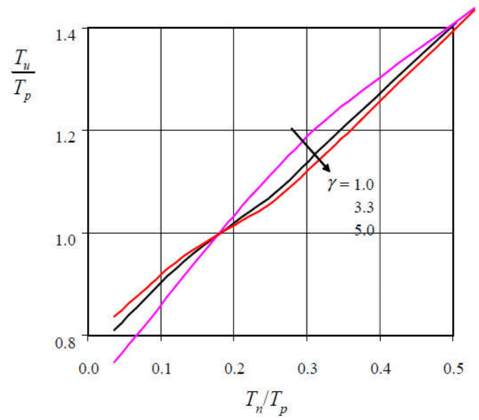

Fig 2: Mean zero crossing period of oscillation period of flow at seabed level (DNV, 2010)

From equation (1), passive resistance, F_R may be obtained from equation (25):

$$\frac{F_R}{F_C} = \frac{4.1 S_u D_o}{G_c^{0.39} F_C}\left[\frac{z_p}{D_o}\right]^{1.3\ 1} \qquad (25)$$

Where, $G_c =$ soil (clay) strength parameter $(= S_u / D_o\, \gamma_s)$; $\gamma_s =$ dry unit soil weight; S_u = shear strength; $F_C =$ vertical contact force between pipe and soil $(= W_s - F_L)$, and the lift force F_L is given as:

$$F_L = \frac{1}{2} D_o C_L \rho_w (\text{U}\cos\theta + V_c \cos\alpha)^2 \qquad (26)$$

Where, C_L =lift force coefficient $(= 0.9)$ and horizontal water particle velocity, U is defined as :

$$\text{U} = \frac{\pi H}{T}\frac{\cosh\left[\text{k}(z+d)\right]}{\sinh(kd)}\cos\theta \qquad (27)$$

From equation (25), initial penetration depth is given as (Verley and Lund, 1995):

$$\frac{z_p}{D_o} = 0.0071(SG_c^{0.3})^{3.2} + 0.062(SG_c^{0.3})^{0.7} \qquad (28)$$

Where, S =vertical force per unit length $(= F_C / D_o\, S_u)$.

Generalized Lateral Stability: Generalized stability analysis method is based on generalized result from dynamic analysis model using non-dimensional parameters and usually applied to section of pipeline where movement and strain are requirement. However, some assumptions are made during the analysis, namely no initial penetration; pipe is rough; no prior loading; cyclic loading due soil resistance; medium sand soil; use of JONSWAP wave spectrum; no reduction of hydrodynamic forces due to penetration and hydrodynamic forces modified for wave effect (Guo *et al*., 2005; DNV, 1988). Additionally, effect of axial loading due operating pressure and temperature are neglected. The generalized lateral stability method permits pipeline displacement with maximum net movement of forty times pipe diameter in DNV-RP-E305 code but has been suspended by DNV-RP-F109 recommendation of ten times pipe diameter displacement (DNV 2010). Design criteria for the generalized lateral stability method can be defined as follow (DNV, 2010):

$$Y(L, K, M, N, G_s, \tau, G_c) \leq Y_{allowable} \qquad (29)$$

Where, $Y_{allowable} =$ allowable non-dimensional lateral pipe displacement limited to 10 times pipe diameter; $Y =$ non-dimensional lateral pipe displacement $(= y/D_o)$ that is governed by the non-dimensional parameters in equations (29) and each of the non-dimensional parameter is defined as:

$$L = \frac{W_s}{0.5 \rho_w D_O U_s^2} \qquad (30)$$

$$K = U_s T_u / D_o \qquad (31)$$

$$M = V_C / U_s \qquad (32)$$

$$G_s = \frac{\rho_s - \rho_w}{\rho_w} \qquad (33)$$

$$N = V_s / g T_p \qquad (34)$$

Where, $L =$ significant weight parameter; $K =$ load parameter (Keulegan Carpenter number); $M =$ current to wave velocity ratio, $y =$ lateral pipe displacement and $G_s =$ sand soil density parameter, However, the generalized stability method is only valid for the range of parameters: $4 < K < 40$; $0 < M < 0.8$; $0.7 < G_s < 1.0$; $0.05 < G_c < 0.8$ and $D_o \geq 0.4m$.

Alternatively, an intermediate displacement criterion can be computed as follow:

$$\log(L_y) = \log(L_{stable}) + \frac{\log(L_{stable}/L_{10})}{\log(0.5/0.010)}\log(Y/0.5)$$
(35)

Where $L_y =$ required weight intermediate displacement; $L_{stable} =$ weight require to obtain virtually stable pipe; $L_{10} =$ weight required to obtain 10 times pipe diameter displacement. The values of L_{stable} and L_{10} may be obtained through empirical expression and design curves (Fig 3- 4) from database of dynamic analysis. Pipeline stability can be verified by comparing the values of weight parameter of $(L_{stable}, L_{10}\ or\ L_y)$ with values of L. Hence, pipeline is said to be stable if computed value of significant weight parameter, L is greater than the weight parameter values $(L_{stable}, L_{10}\ or\ L_y)$. But, the generalized lateral stability design approach is only valid for $N \leq 0.024$ for clay and $N \leq 0.048$ for sand. For all the design methods, vertical stability of offshore pipeline should be checked to prevent flotation of a pipe. Acceptance criteria for vertical stability must meet the submerge weight of pipeline criteria:

$$\gamma_w \frac{B}{W_s+B} = \frac{\gamma_w}{S_g} \leq 1 \quad (36)$$

Where, γ_w =safety factor and S_g =pipe specific gravity.

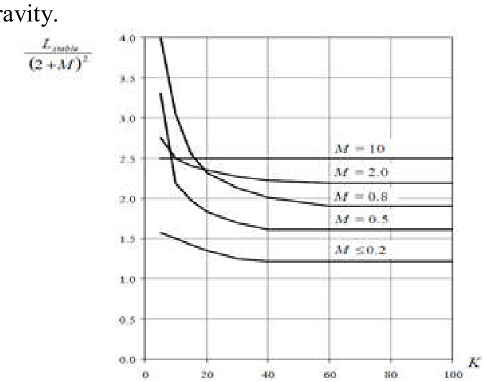

Fig 3: Design curve for L_{stable} weight parameter for pipe on sand

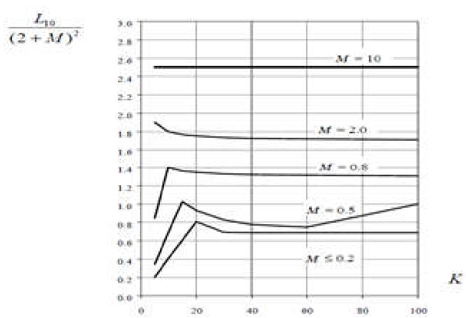

Fig 4: Design curve for L_{10} weight parameter for pipe on sand

Traditional Force Method: Traditional force balance method is governed by Morrison theory which are used to estimate required concrete and steel pipe wall thickness. Conventional model for design balance horizontal and lifts forces against minimum total submerged weight of pipeline include wrap and concrete coating. The static design method is expressed as follow (Bai, 2001):

$$\gamma(F_D - F_I) \leq \mu(W_s - F_L) \quad (37)$$

Where, μ = coefficient of friction; F_D = drag force per unit length; F_I = initial force per unit length; F_L = lift force per unit length (equation 26) and γ = safety factor $(= 1.1)$. The horizontal and vertical wave induced forces per unit length are expressed as follow (Morrison *et al.*, 1950):

$$F_D = \frac{1}{2} D_o C_D \rho_w (\mathrm{U}\cos\theta + V_c \cos\alpha)/(\mathrm{U}\cos\theta + V_c \cos\alpha) \quad (38)$$

$$F_I = \frac{\pi D_o^2}{4} C_I \rho_w (du/dt) \cos\theta \quad (39)$$

Where, C_D =drag coefficient $(= 0.7)$, $C_I =$ inertial drag force$(= 3.29)$; θ =wave flow direction; $\alpha =$ current flow direction, and du/dt is wave induced water particle acceleration (horizontal) defined as:

$$du/dt = a = \frac{2\pi^2 H}{T} \frac{\sinh[\mathrm{k}(z+d)]}{\sinh(kd)} \cos\theta \quad (40)$$

Conclusion: This paper presented different pipeline on-bottom stability design methods. Acceptance criteria requires that computed submerged weight be greater than the lift force for traditional force method. While, estimated sum of peak horizontal and vertical loads divided by sum of submerge weight and resistance force less than unity is acceptance criteria for absolute lateral stability. Whereas, acceptance criteria for the generalized lateral stability defined that significant weight parameter be greater than the weight parameter value.

REFERENCES

AGA (2002). Submarine pipeline on-bottom stability – analysis and design guidelines. Vol. 2. p178.

AGA (2000). Submarine pipeline on-bottom stability analysis software and subsea pipeline design standard. *Pipeline Research Council International*. Version, L52 300, USA.

Allen, DW; Lammert, WF; Hale, JR; Jacobsen, V (1989). Submarine pipeline on-bottom stability: recent AGA research. *Proceeding 21st Annual Offshore Technology Conference*, OTC 6055, Houston, Texas, 121-132.

Ameh, ES (2009). Fracture toughness and evaluation of offshore transmission pipelines, MSc Thesis, Marine Department, Newcastle University, United Kingdom.

Bai, Y (2001). Pipeline and Riser. Elsevier Science Limited, Oxford UK.

Bryndum, MB; Jacobsen, V; Tsahalis, DT (1992). Hydrodynamic forces on pipelines: model tests. *J. Offsh. Mech. Arctic Eng*. 114: 231-241.

Choi, YS; Lee, S K; Do, CH (2013). An optimum design of on-bottom stability of offshore pipelines on soft clay. *Int. J. Naval Archit. Ocean Eng*. 5: 598-613

DNV (1988). On-bottom stability design of submarine pipeline, det norske veritas (DNV). Recommended practice, RP–E305, Oslo Norway.

DNV (1981). Rules for design, construction and inspection of submarine pipelines and rivers, det norske veritas (DNV), Oslo Norway.

DNV(2010). On-bottom stability design of submarine pipelines, det norske veritas (DNV). Recommended practice, RP-F109, Oslo, Norway.

Gao, F; Jeng, DS; Wu, Y (2006). Improved analysis method for wave-induce pipeline stability on sand seabed. *J. Transp. Eng*. 132(7): 590-596.

Gao, F; Yan, S; Yang, B; Wu, Y (2007). Ocean current induced pipeline lateral stability on sand seabed. *J. Eng. Mech.* 133(10): 1086-1097.

Guo, B; Song, S; Chacko, J; Ghalambo, A (2005). Offshore Pipelines. Elsevier, UK.

Hasselmann, K; Barnett, TP; Bouws, E; Carlson, H; Enkeg, K; Ewing, JA; Gienapp, H; Hasselmann, DE; Kouseman, P; Meerburg, A; Muller, P; Olben, DJ; Ritcher, K; Sell, W; Walden, H (1973). Measurement of wind wave growth and swell decay during the joint North Sea wave project. Deutschen Hydrographschen Institute, Hamburg, Germany.

Iyalla, I.; Umah, K; Hossain, M (2010). Computational fluid dynamic modelling of pipe-soil interaction in current. *Proceedings of the World Congress on Engineering*, 2, WCE, London, United Kingdom.

Merifield, RS; White, DJ; Randolph, MF (2009). Effect of surface heave on response of partially embedded pipelines on clay. *J. Geotech. Geoenviron. Eng*. 135(6): 819 - 829.

Morrison, JR; Obrien, MP; Johnson, JW; Schaaf, SA (1950). The forces exerted by surface waves on piles. *J. Petr. Tech.* 189: 149-154.

Palmer, AC; Roger, AK (2008). Subsea pipeline engineering. Pennwell, Oklahoma, USA.

Tornes, K; Zeitou, H; Cumming, G; Willcocks, A (2009). A stability design rational- a review of present design approaches. *Proceeding of the ASME 28th International Conference on Ocean Offshore and Arctic Engineering,* OMAE, Hawaii, USA.

Verley, R; Lund, KM (1995). A soil resistance model for pipelines placed on clay soils, OMAE, Hawaii, USA.

White, DJ; Randolph, MF (2007). Seabed characterization and models for pipeline-soil interaction. *J. Int. Offsh. Polar Eng.* 17(3): 193-204