XXXVI Ibero-Latin American Congress on Computational Methods in Engineering
CILAMCE
Rio de Janeiro, 22-25 Nov 2015

A SEMI-ANALYTICAL METHODOLOGY TO EVALUATE LONG-TERM RESPONSES AT THE TDZ OF CATENARY RISERS

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Abstract. *Response based approaches are not common in the design of risers. Due to the high computational costs associated to these methodologies, it is usual to replace the calculation of extreme long-term responses by the calculation of responses to a few number of artificial sea states, supposed extreme. For instance, designers commonly use combinations of environmental loads such as i) 100y waves and winds plus 10y currents, and ii) 10y waves and winds plus 100y currents. In 2001, however, the DNV standard OS-F201 presented a methodology based on structural reliability, known as long-term response statistics, which is supposed to be more adequate to fit the riser design to a pre-established level of failure probability. The utilization of this methodology is limited, in most cases, by its high computational costs, because time-domain simulations may be computationally very expensive. An alternative to reduce this cost is the utilization of simplified analytical models; in this way, the objective of this work is to detail a semi-analytical model used to obtain short-term bending moments and tensions at the TDZ, based on models proposed by ARANHA et al. (1997). This analytical model can then be used in the evaluation of long term utilization factors for SCRs. The utilization of the semi-analytical model is exemplified by a case study where a rigid riser connected to a spread moored FPSO is analyzed. The obtained results indicate that this methodology can contribute to simplify the way risers are designed, focusing on the response instead of on the occurrence of extreme sea states.*

Keywords: *Riser design, bending moments at the TDZ, long-term responses.*

1 INTRODUCTION

The most used configuration for risers is probably the free hanging catenary, as shown in Figure 1. In this configuration, the riser is supported by the floating unit and extends freely to the sea bottom. This configuration is used not only because of its simplicity, in terms of engineering and construction, but also because of its cost and efficiency.

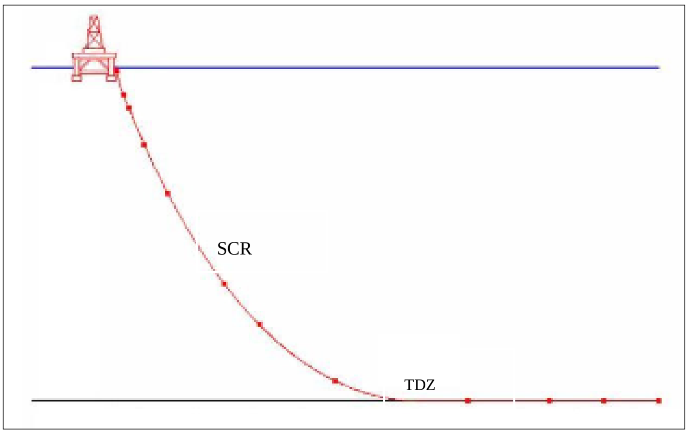

Figure 1 – Standard free hanging catenary configuration.

The evaluation of catenary risers' extreme responses is a very complex task. The top connection motions induced by environmental loads, the drag occasioned by currents and waves and the riser-soil interaction make the risers' behavior nonlinear, mainly at the TDZ (touch down zone). For this reason, risers' analyses are usually performed on time domain, using specific finite elements software, e.g. ANFLEX (2010). These analyses need very high computational resources, requiring long simulation times to achieve the stability of the statistical parameters of the response for each loading case.

To obtain risers' extreme responses, standards such as API RP 2RD (1998) and API RP 17B (2002) suggest that analyses should be performed subjecting the risers to pre-established combinations of extreme environmental loads. In this way, to evaluate 100y responses, riser designers commonly use combinations such as 100y waves and winds plus 10y currents, and 10y waves and winds plus 100y currents. This assumption allows the evaluation of responses based on a relatively few number of loading cases, but it does not evaluate the real response associated to a given recurrence period (for instance, 100y) – it evaluates the riser response to loading cases that are supposed centenary.

In 2001, the DNV OS-F201 standard (DNV, 2001) suggested new procedures for the design of risers. The standard defines an alternative methodology based on the long-term statistics of risers' responses; also, for steel risers, it introduced a new structural assessment criterion, represented by the LRFD utilization factor. DNV explicitly recognizes, however, that this approach has a main challenge, which is to determine the short-term load effect distribution – the methodology may be computationally very expensive.

To reduce this computational cost, researchers at LACEO/COPPE/UFRJ have been working on the development and implementation of analytical or semi-analytical methods to obtain short-term responses. For instance, PAPALEO (2009) proposed an integrated analytical procedure to evaluate extreme motions at platforms, and SOUSA (2011) used an analytical model proposed by ARANHA and PINTO (2001a) to evaluate long term tension at the top of risers.

The objective of this work is to extend the evaluation of risers' responses to the TDZ. To achieve this goal, an analytical model developed by ARANHA et al. (1997) to evaluate dynamic bending moments was used. Due to the obtained results, the focus of this work was limited to moments and utilization factors for steel catenary risers (in this work, riser responses mean LRFD utilization factors).

The work is still under development, and the authors are starting the evaluation of long term responses. For this reason, the text was organized aiming to just outline the methodology to evaluate long-term responses, and to detail the semi-analytical model to determine bending moments and utilization factors at the TDZ.

2 SUMMARY OF LONG TERM RESPONSES CALCULATION

The main steps of the long-term response evaluation methodology are summarized in Figure 2.

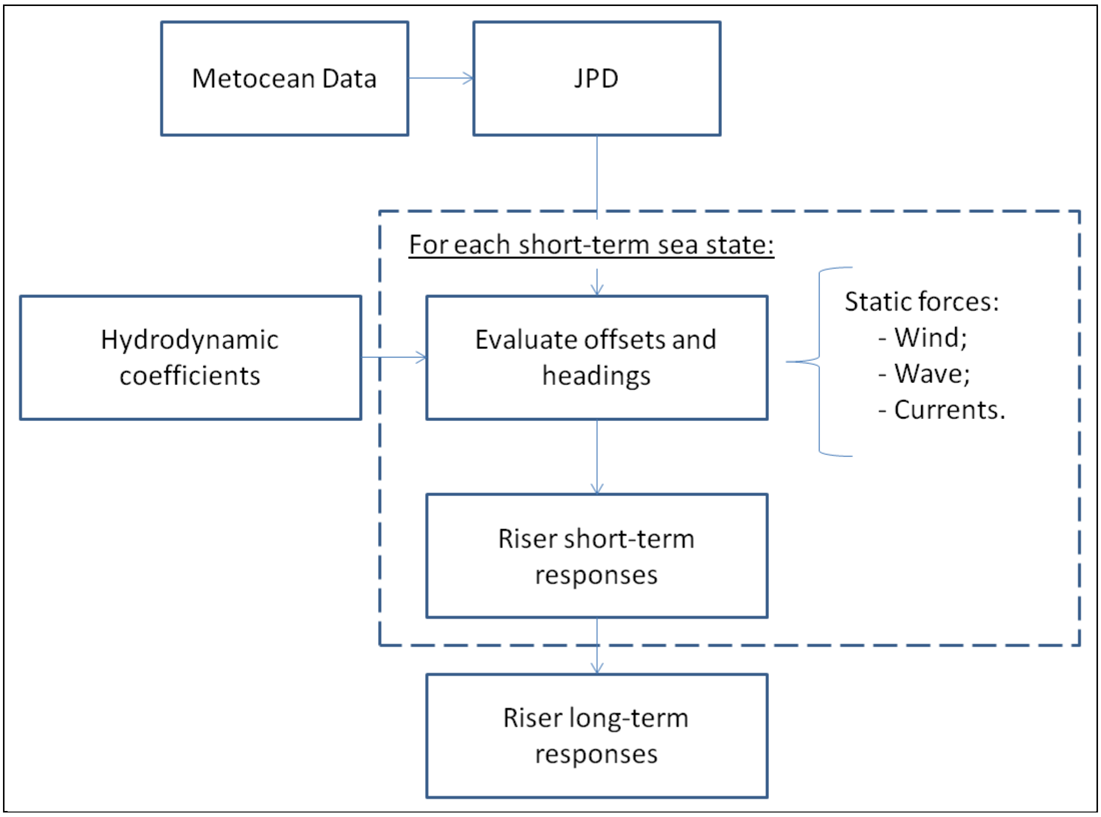

Figure 2 – Long-term response evaluation – main steps.

The methodology starts by the analysis of metocean data. The objective of this analysis is to obtain a joint probability distribution of environmental parameters (JPD), such as the one proposed by SAGRILO et al. (2011) for Campos' Basin. It characterizes short-term sea states (3h) by means of the 10 environmental parameters presented in Eq. (1). The JPD is defined as a function of the marginal distribution of the S_i variables and the correlation coefficients between each pair of them, including correlations between linear and circular data.

$$\boldsymbol{S} = \{H_{Ssw}, T_{Psw}, \theta_{SW}, H_{Sss}, T_{Pss}, \theta_{SS}, V_W, \theta_W, V_C, \theta_C,\}^T \quad (1)$$

where:

$\boldsymbol{H_{Ssw}}$	is the wind sea significant wave height,
$\boldsymbol{T_{Psw}}$	is the wind sea wave spectral peak period,
$\boldsymbol{\theta_{SW}}$	is the wind sea direction,
$\boldsymbol{H_{Sss}}$	is the swell significant wave height,
$\boldsymbol{T_{Pss}}$	is the swell spectral peak period,
$\boldsymbol{\theta_{SS}}$	is the swell direction,
$\boldsymbol{V_W}$	is the wind velocity,
$\boldsymbol{\theta_W}$	is the wind direction,
$\boldsymbol{V_C}$	is the superficial current velocity and,
$\boldsymbol{\theta_C}$	is the superficial current direction.

Next, the obtained JPD is used in the artificial generation of samples of short term sea states **S**. Each sample is obtained from the automatic generation of N_S values of statistically independent standard normal variables. Any number M of samples can be generated by repeating the procedure M times. As presented by SOUSA (2011), the necessary number of samples (N_S X M) can reach the magnitude of hundreds of thousands of sea states.

The third step of the methodology is the evaluation, for each sea state, of the static equilibrium positions of the floating unit (heading and offsets), according to the simplified procedure proposed by SOUSA (2012). The procedure is incremental and iterative. Forces and moments due to wind and current are obtained from classical drag expressions (CHAKRABARTI, 1987, and OCIMF, 1994), where the drag coefficients are functions of the incidence angle in the floating unit. The static forces and moments generated by waves, both for sea and swell, are computed considering only mean drift forces. The mooring stiffness is re-evaluated for each loading increment, and has to take into account risers and mooring lines. Current forces in risers and mooring lines are calculated for each loading increment, too.

The fourth step, which is the main focus of this work, is the evaluation of time series of tensions and bending moments (and consequently, utilization factors), considering the dynamic top connection motions induced by the environmental loads and the semi-analytical models (detailed in item 3). If time domain analyses were used in this evaluation, the computational costs associated would be prohibitive, because the methodology requires the evaluation of statistical parameters of the response for "all possible sea states" in 100y. In this way, analytical models become very attractive to help to reduce these costs.

Finally, long-term responses are computed considering the short-term response distributions and the probability of occurrence of each sea state. Considering the response as a narrow-banded random Gaussian process, it is possible to assume the cumulative distribution of response peaks for a given sea state **S = s_i** as:

$$F_R(r) = \int_0^\infty \frac{v_0(\boldsymbol{s})}{\bar{v}_0} F_R(r|\boldsymbol{S} = \boldsymbol{s_i}) f_{\boldsymbol{S}}(\boldsymbol{s})\, d\boldsymbol{s} \quad (2)$$

where:

$\boldsymbol{v_0}(\boldsymbol{s})$	is the mean zero up-crossing frequency of short-term response,
$\bar{v}_0$	is the mean up-crossing frequency considering the whole set of sea states, given by: $\bar{v}_0 = \int_{\mathbf{S}} v_0(\mathbf{s})\, f_s(\mathbf{s}) d\mathbf{s}$,

$f_S(\boldsymbol{s})$ is the joint distribution of environmental parameters evaluated for the sea state $\mathbf{S} = \mathbf{s_i}$.

The extreme response can be obtained from order statistics (ANG e TANG, 1984):

$$F_{Re}(r) = [F_R(r)]^{N_P} \tag{3}$$

and the most probable value of $F_{Re}(r)$ can be calculated from the inverse cumulative distribution, where N_P is the expected number of dynamic response peaks in a return period of N years. Considering 2920 seat states per year, 3h each, it can be calculated by the following expression:

$$F_R(r^N) = 1 - \frac{1}{N_P} = 1 - \frac{1}{\overline{v}_0 * 10800 * N * 2920} \tag{4}$$

3 SHORT TERM RESPONSES

3.1 Description

The evaluation of short term responses (utilization factors) at the TDZ is also a task that can be divided in steps, as shown in Figure 3.

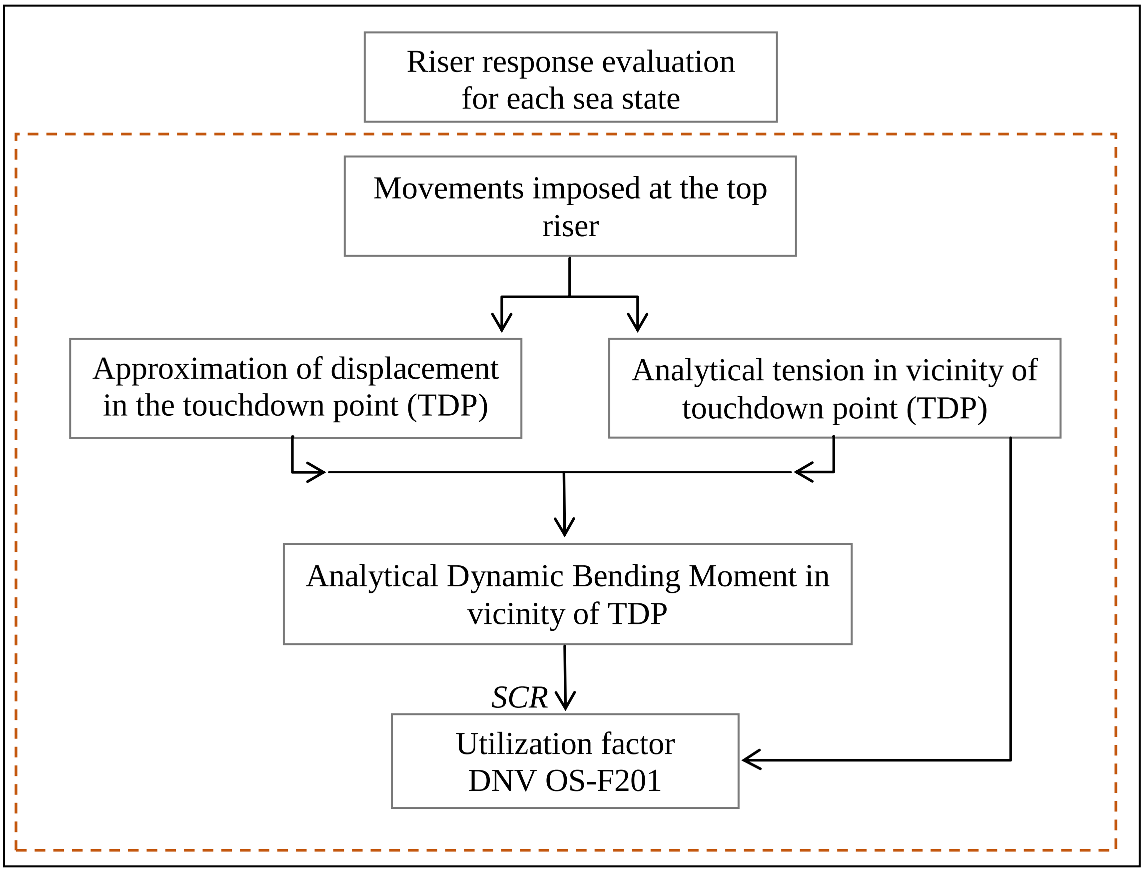

Figure 3 – Short-term dynamic bending moment – main steps (MARROQUIN, 2013).

3.2 Dynamic Tension at the Touchdown Point

ARANHA et. al. (1997) evaluate the dynamic tension supposing that the riser is subjected to harmonic motions applied in the direction defined by the top angle. The authors identify two extreme situations to explain the dynamic behavior of a riser: the first one is a slack cable, where the top displacements imply only in changes in the catenary configuration; the second one is a tight cable, where the top displacements force the riser to deform elastically. In this way, the dynamic tension can be evaluated by the following expression:

$$\tau(t) = T_e \sqrt{\frac{c_1(x_{TDZ})[\sqrt{b^2(\Omega)+4\xi_0^2 a(t)^2/\Omega^4} - b(\Omega)]^2 + 2c_2(x_{TDZ})[\sqrt{b^2(\Omega)+4\xi_0^2 a(t)^2/\Omega^4} - b(\Omega)]}{4\xi_0^2/\Omega^4}} \quad (5)$$

where:

T_e	is the elastic tension,
c_1, c_2, b, Ω *and* ξ_0	depend on the frequency of the imposed motion, current velocity and damping,
a	depends only on the imposed motion amplitude.

More details about all these equations can be found in ARANHA et. al. (1997).

3.3 Static Bending Moment

In the model presented by ARANHA et. al. (1997), the dynamic bending moment at the catenary riser's touchdown point depends on the TDP displacement $x_0(t)$ and the dynamic tension $\tau(t)$. Starting from the catenary equation and disregarding the influence of flexural stiffness in the riser, the static bending moment in the TDP can be written as:

$$M_0 = M(0) = EI\frac{q}{T_0} \quad (6)$$

where:

EI	is the bending stiffness,
q	is the weight per unit of length ,
T_0	is the static tension in the TDP.

The above expression is always exact for a catenary, and the effect of the current is took into consideration indirectly, through T_0.

The curvature of the catenary is not continuous at the TDP; it is zero before the TDP (from anchor) and different of zero after the TDP. The effect of the flexural stiffness *EI* is to smooth the transition between these two values of the curvature, while displacing to the right (In Figure 1) the actual position of the TDP. The "flexural length" λ is defined by the expression:

$$\lambda = \sqrt{\frac{EJ}{T_0}} \quad (7)$$

The touchdown point is displaced λ to the right and the static bending moment in vicinity of touchdown point is given by:

$$M_{est}(S) = \frac{1}{2}[1 + sign(x_s + \lambda)]\left[1 - e^{-\frac{s+\lambda}{\lambda}}\right] M_0 \quad (8)$$

3.4 Dynamic Bending Moment

The dynamic bending moment determination is more complex than the static solution. Movements that are intense in terms of speed and acceleration deform the catenary, and generate compression waves that propagate along the riser. ARANHA et. al. (1997) proposed the following speed limits for the instantaneous displacement in the TDP, when the horizontal velocity in this point is smaller than the "line" wave velocity:

$$C_0 = \sqrt{\frac{T_0}{m+m_a}} \quad (9)$$

with *m* being the riser´s mass per unit of length and m_a the added mass.

Setting the length of dynamic bending as: $\hat{\lambda} = \lambda/\sqrt{(1 + \tau(t)/T_0\,)}$, one may use a similar expression to the static bending moment for the dynamic bending moment, considering the instantaneous displacement in the TDP given by $x_0(t)$, and introducing the variable:

$$\beta(x_s, t) = \frac{\sqrt{1+\frac{\tau(t)}{T_0}}}{\lambda}\left[x_s - x_0(t) + \frac{\lambda}{\sqrt{1+\frac{\tau(t)}{T_0}}}\right] \quad (10)$$

Finally, the dynamic bending moment in the touchdown region can be written as:

$$M_{dyn}(x_s, t) = \frac{1}{2}[1 + sign(\beta(x_s + \lambda))]\left[1 - e^{-\beta(x_s,t)}\right]\frac{M_0}{1+\frac{\tau(t)}{T_0}} \quad (11)$$

After determining the tension and bending moments at the TDZ, the LRFD utilization factors can be calculated following equations presented by DNV (2001).

The determination of displacements at the touchdown point $x_0(t)$ is a hard limitation to the model proposed by ARANHA et. al. (1997). In this work it was decided to use a polynomial model (expressed by Equation (12)) for the evaluation of $x_0(t)$. The polynomial model used is based on the execution of some previous dynamic analysis to calibrate the polynomial coefficients, where $X\ and\ Z$ are the displacements at the top of the riser.

$$x_0(t) = B_1X + B_2Z + B_3V_X + B_4V_Z + B_5V_X|V_X| + B_6V_Z|V_Z| \quad (12)$$

4 CASE STUDY

4.1 Riser Model and Loading Cases

The semi-analytical model was applied to a free-hanging steel catenary riser connected to a FPSO spread mooring in a water depth of 1000m. Table 1 shows the most important riser's properties.

Table 1- Riser's properties.

Azimuth (deg)	270
Top angle (deg)	9
External diameter (in)	8
Thickness (in)	1
Dry weight (kN/m)	1.2
Axial stiffness (kN)	2.94E+06
Bending stiffness (kN*m^2)	11842

Nine cases of short-term analysis composed by wave, current and three offset levels (2%, 4% and 6%) in far, near and transverse conditions were examined as shown in Table 2 and 3. Waves, current and offset aligned according to the reference system is shown in Figure 4.

Table 2- Loading cases employed in the SCR example.

Case	**Type**	**Wave**	**Current**	***Offset***
1	Far	1y, W	1y, E	2%
2	Far	10y, W	10y, E	4%
3	Far	100y, W	100y, E	6%
4	Near	1y, E	1y, W	2%
5	Near	10y, E	10y, W	4%
6	Near	100y, E	100y, W	6%
7	Transverse	1y, S	1y, N	2%
8	Transverse	10y, S	10y, N	4%
9	Transverse	100y, S	100y, N	6%

Table 3- Waves and currents.

Recurrence period (y)	**Wave E, W, S (origin)**		**Currents: E, W, N Speeds at (surface / sea bottom**
	H_S (m)	**T_p (s)**	**(m/s)**
1	3	10	0.25 0.025
10	5	12	0.5 0.05
100	7	14	1.0 0.1

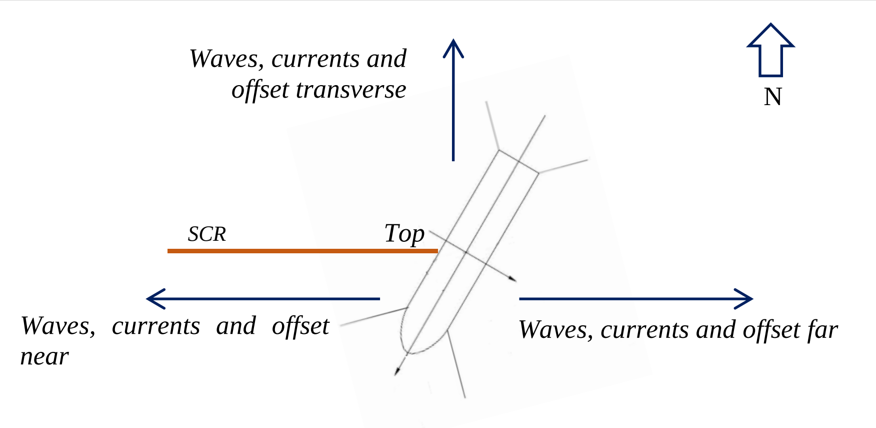

Figure 4 – Riser connected to Spread Moored FPSO.

The analytical model's results were compared with the results obtained by dynamic analysis in the time domain performed by the finite element software (FEM), with 2400s time analysis.

The analyses performed to calibrate the polynomial model that approximates the displacement in the touchdown point case were related to near and far conditions (1y, 100y waves and currents).

4.2 Results

For this riser, the tension at the TDP in the neutral position is 166.5 kN. Thus, the λ parameter calculated by Eq. (7) is 8.4m, and the maximum static bending moment is 65kN.m.

The tables below present the results summary in terms of statistical parameters of dynamic tension and bending moment time series obtained for each one of the nine analyzed cases. For each case, six different positions (-λ to 6λ from TDP) were analyzed, covering the entire range of displacement variation at the touchdown zone.

Table 4 presents a comparison between the dynamic tension results in the TDZ. It can be seen that the differences between maximum and minimum values obtained by FEM model and the analytical formulation were always lower than 10%, except in cases 3, 5 and 6. In such cases, they represent cases near and far with most significant wave height and resonant periods of the ship. In these cases, the riser has an intense dynamic behavior, which produces the dynamic tension, not exactly depending on the effective length of the riser on sea floor.

Table 4- Statistical parameter of dynamic tension (kN) obtained by FEM program and analytical methodology proposed.

Case	Model	Mean	STVD	Max	Min	Skewness	Kurtosis
1	FEM	190.2	6.2	219	168.6	0.3	4.4
	Analytical	188	5.6	209.7	167.6	0	3.8
2	FEM	219.2	18.3	309.2	150.8	0.5	5.1
	Analytical	215	17.2	283.2	147.1	0	4.6
3	FEM	259.1	35.8	421.2	132.4	0.6	5.3
	Analytical	251	28.8	380.2	124.4	0	5.6
4	FEM	144	4.6	163.9	123.2	-0.1	5.1
	Analytical	142	4.2	157.6	125.7	0	3.6
5	FEM	122.4	11.6	172.5	73.4	-0.2	5.3
	Analytical	121	8.5	155.6	86.2	0	4.6
6	FEM	97.2	16.8	167.6	16.7	-0.2	4.9
	Analytical	97	11.1	142.2	47.7	0	4.5
7	FEM	166.4	0.7	168.9	164	0	3
	Analytical	166	0.9	169	163.2	0	2.9
8	FEM	167.9	1.5	174.3	162.5	0.1	3.6
	Analytical	168	1.5	173.2	163.1	0	3.2
9	FEM	171.1	3	183.1	159.2	0	4
	Analytical	172	2.4	179	163.2	0	3.3

Figure 5 shows the dynamic tension time series for case 2, where it can be seen that the signals are much closed, having small differences in the maximum and minimum peaks.

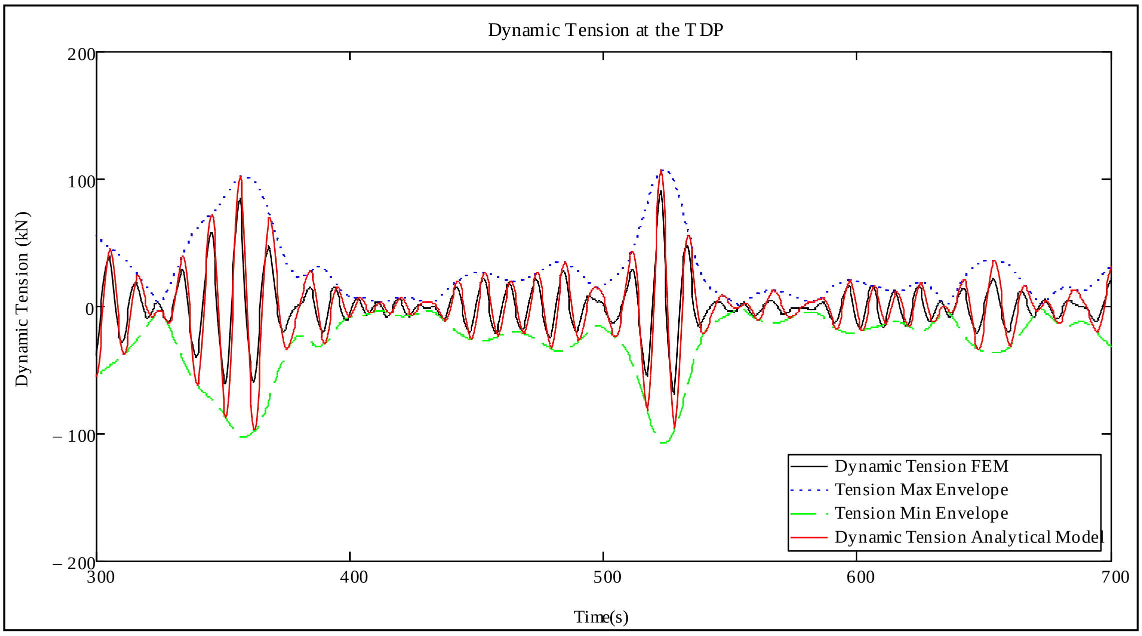

Figure 5 – Comparison between dynamic tension FEM and analytical model proposed, Case 2.

Figure 6 shows the approximation of displacement in the touchdown point obtained by the polynomial model (case 6). It is possible to see that the error associated to the evaluation of maximum and minimum displacement peaks are small.

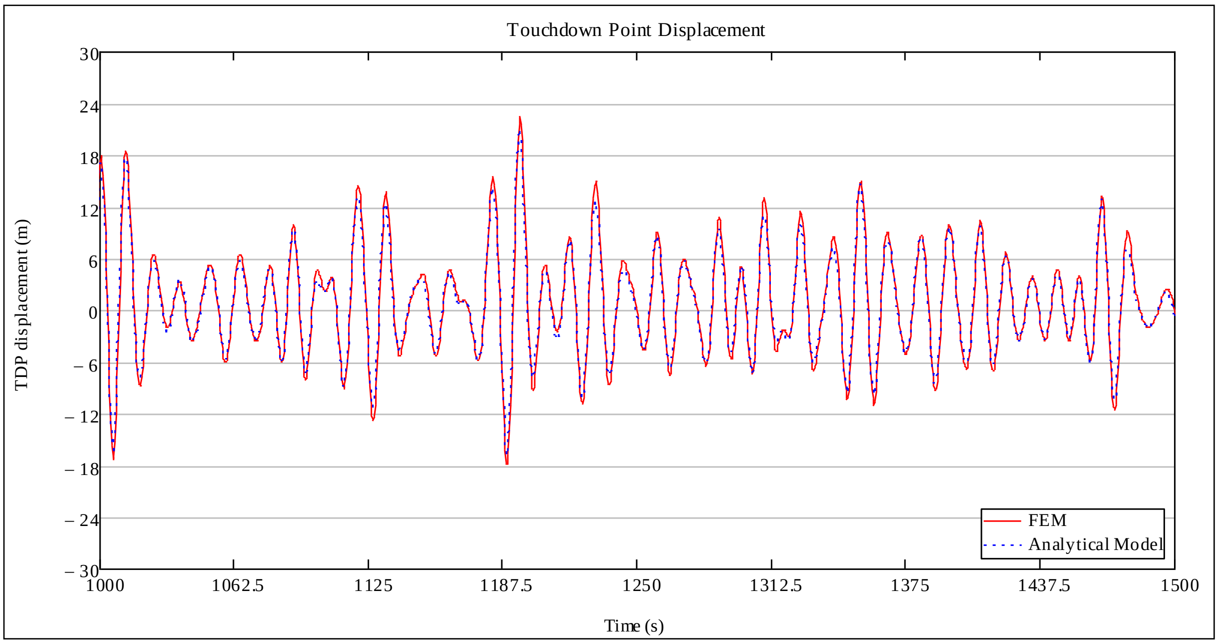

Figure 6 – Displacement time series in the touchdown point, case 6.

Table 5 show the results obtained for the dynamic bending moment for all cases, and Figures 7 and 8 show a comparison between the results for case 2 (3λ position) and 6 (2λ position) . The results include the estimation of statistical parameters (mean, standard deviation, skewness, kurtosis and maximum observed). It can be seen that the analytical model was able to approximate the maximum bending moment efficiently, since the differences observed were below 10% in almost all the analyzed cases. However, for cases 5 and 6, values with an average of over 10% were observed, which generated maximum values of bending moment over dimensioned.

Figure 8 present the times series bending moment for case 6 in the 2λ position. It is possible to observe a difference between the average and a greater difference on the maximum peak of bending moment. These results cannot be justified neither by the differences in the approach of displacement in the touchdown point nor by the differences between dynamic tensions, because the dynamic tension obtained by the FEM program is greater than the analytical model. In this case, the differences can suggest a limitation of the model. Also significant differences were observed in the values of the dynamic bending moment at the $-\lambda$ *position*. At this position the oscillatory variation of the TDP makes the riser to "*lift off*" and "*lay down*" on the sea floor causing large curvature in this region. The times series bending moment is limited to the minimum of zero and becomes highly dependent on an accurate approximation of displacement in the TDP.

*Table 5- Statistical parameters of dynamic bending moment (kN*m) obtained by FEM program and analytical methodology proposed.*

Case	Position	$-\lambda$		0λ		λ		2λ		3λ		4λ	
	Parameter	**FEM**	**Analytical**	**FEM**	**Analytical**	**FEM**	**Analytical**	**FEM**	**Analytical**	**FEM**	**Analytical**	**FEM**	**Analytical**
1	*Mean*	3.5	5.6	32.9	36.2	47.4	49.8	53.1	54.7	54.8	56.5	55	57.1
	STVD	5.6	6.9	5.8	4.3	2	1	1.4	1	1.7	1.4	1.9	1.61
	Skewness	2.1	1.2	-0.8	-1	-0.5	-0.9	0.2	0	0.1	0.1	0.1	0.2
	Kurtosis	7.4	3.4	4.6	4.9	4.3	5	3.8	4	3.9	3.8	3.8	3.9
	Max	32.7	31.9	46.9	45.1	53.9	52.7	59.1	58.5	61.7	62.0	62.2	63.4
2	*Mean*	6.7	10	25.7	30.8	40.1	43	46.1	47.7	47.7	49.4	48.3	50.1
	STVD	10.6	11.4	11.6	9.7	5.7	3.2	3	2.5	3.4	3.4	3.8	3.9
	Skewness	1.5	0.8	-0.7	-1.5	-2.4	-2.1	0.3	0.2	0.3	0.5	0.5	0.7
	Kurtosis	4	2.2	2.7	4.9	12.9	12.6	4.6	5.7	4.7	5.3	4.8	5.6
	Max	42.7	38.6	49.6	45.8	55.4	53.4	60.7	60.6	64.7	66.5	67.4	70.1
3	*Mean*	10.8	12.5	23.3	25.3	33.1	35.4	38.3	40.2	40.4	42.3	41.2	43.1
	STVD	12.8	13.2	12.8	12.4	8.7	7.1	5.4	4	4.7	3.9	5.1	4.7
	Skewness	0.7	0.7	-0.6	-0.9	-2	-2.5	-2	-2.1	0.2	0.9	0.8	1
	Kurtosis	2	1.8	2.2	2.6	7.7	12	16.3	26.3	6.4	7.8	5.7	7.1
	Max	45.0	42.0	50.7	48.6	55.3	55.9	59.8	63.0	65.4	67.7	71.0	69.9
4	*Mean*	1.1	6.1	38.8	48.2	61.9	65.2	69.2	72.1	71.1	74.7	70.7	75.6
	STVD	3.3	7.8	7.2	4.9	2.3	1.2	1.8	1.5	2.2	1.9	2.3	2.1
	Skewness	3	1.2	-1.1	-0.8	-0.7	0	0.3	0	0.5	0.1	0.5	0.2
	Kurtosis	14.8	3.6	5.3	4.3	4.5	3.3	4.1	3.7	4.7	3.7	4.7	3.7
	Max	25.6	36.0	54.7	58.9	68.7	69.7	77.1	78.1	81.4	82.7	81.9	84.7
5	*Mean*	5.2	7.6	44.1	51.9	71.1	75.6	80.2	84.5	82.2	87.8	80.8	89
	STVD	10	11.6	16.2	11.8	6.2	3.7	5.2	4.3	6.5	5.5	6.6	6.1
	Skewness	2	1.5	-1	-1.4	-1.9	-0.5	0.5	0.2	0.9	0.5	1	0.7
	Kurtosis	6.7	4.2	3.5	6	11.9	4.9	4.7	4.9	5.3	5.5	5.5	5.3
	Max	51.7	52.0	72.8	73.8	89.7	91.4	105.1	106.7	115.8	116.6	115.6	121.6
6	*Mean*	8.9	12.3	53.6	61.7	85.5	93.1	97.2	105.4	99	110	94.7	111.8
	STVD	15.4	17.3	24	21	11.3	8.9	10.3	9.5	12.5	11.8	11.6	12.9
	Skewness	1.7	1.2	-0.9	-1.3	-1.9	-0.7	0.9	0.7	1.2	0.9	1.3	1
	Kurtosis	4.7	3.2	2.9	4.3	12.2	6.1	5.1	5.7	5.9	5.9	6	6.3
	Max	70.2	71.0	97.0	101.9	124.3	137.7	157.5	161.9	177.7	173.0	167.3	179.4
7	*Mean*	0.7	2.3	36.8	41	54.3	56.2	60.4	61.8	62.2	63.9	62.2	64.6
	STVD	0.5	2.2	1.1	0.8	0.4	0.2	0.3	0.2	0.3	0.3	0.3	0.3
	Skewness	0.7	0.7	-0.1	-0.1	-0.1	-0.1	0	0	0.1	0	0	0
	Kurtosis	3.7	2.8	3	2.9	2.9	2.9	3	2.9	3.1	2.9	3.1	2.9
	Max	3.3	10.4	40.3	43.3	55.8	56.8	61.3	62.5	63.2	64.9	63.2	65.7
8	*Mean*	0.7	2	36	40.6	53.7	55.6	59.9	61.1	61.6	63.1	61.7	63.9
	STVD	1.2	2.9	2.4	1.8	0.8	0.5	0.5	0.3	0.6	0.5	0.6	0.5
	Skewness	1.7	1.5	-0.3	-0.3	-0.2	-0.2	0.1	0	0.1	0	0.1	0
	Kurtosis	8.3	4.9	3.5	3.1	3.2	3.1	3.2	3.1	3.6	3.1	3.7	3.2
	Max	9.4	16.1	43.6	45.8	56.4	57.0	61.8	62.2	63.9	64.7	64.2	65.6
9	*Mean*	1.2	4.1	34.5	39.6	51.9	54.3	58.5	59.7	60.4	61.7	60.6	62.4
	STVD	2.8	5.5	4.9	3.7	1.7	1.1	0.9	0.5	1	0.7	1.1	0.8
	Skewness	2.5	1.3	-0.7	-0.5	-0.6	-0.5	0.1	0.1	0.2	0.1	0.2	0.1
	Kurtosis	11	3.9	3.8	3.2	3.6	3.2	3.3	3.2	3.9	3.3	4	3.3
	Max	19.5	25.9	46.4	48.3	56.5	57.1	61.9	61.7	64.6	64.3	65.3	65.5

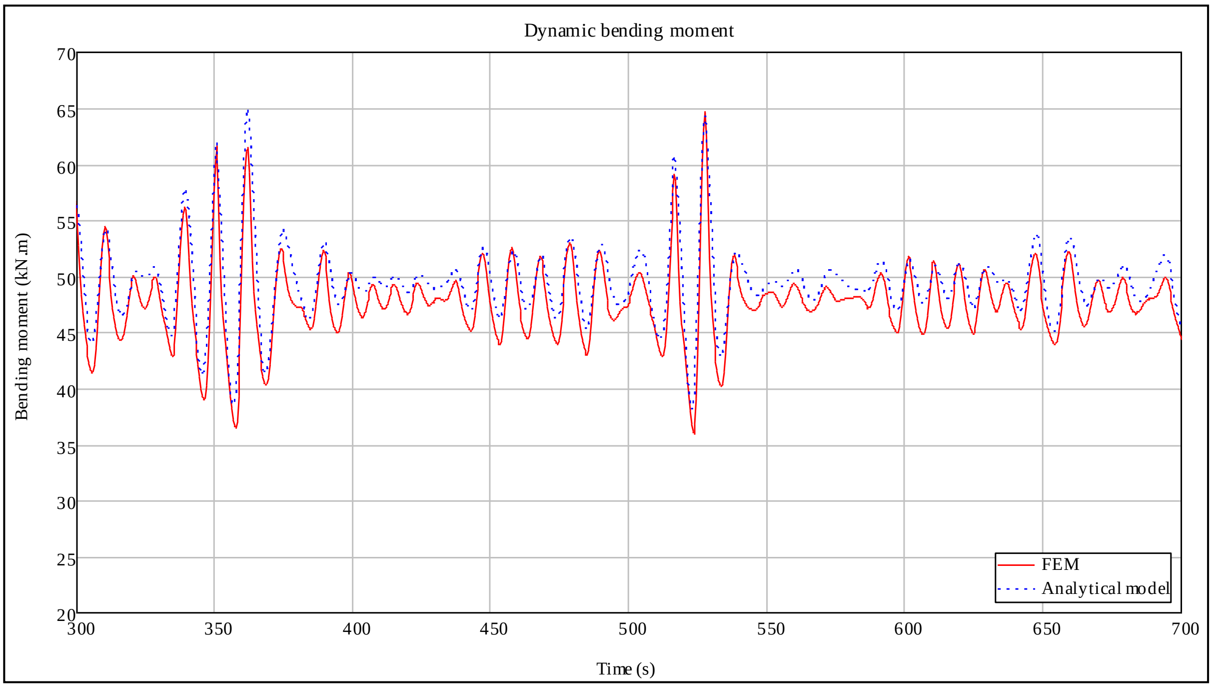

Figure 7 – Dynamic bending moment, case 2, position 3λ.

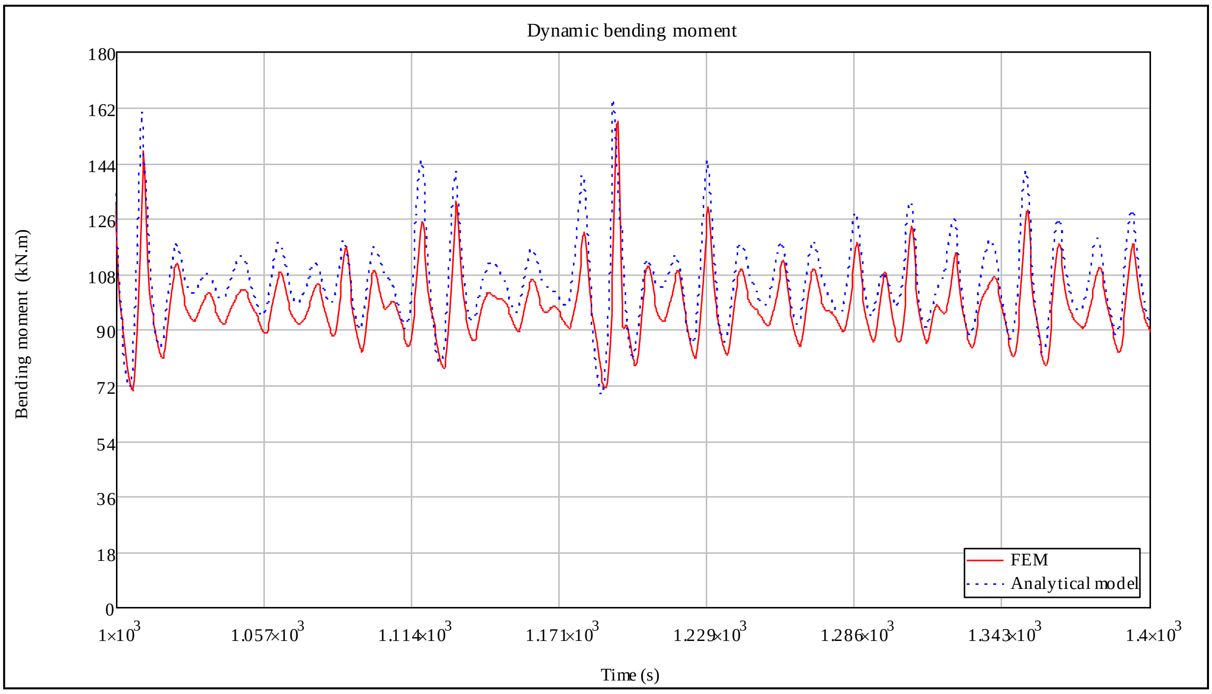

Figure 8 – Dynamic bending moment, case 6, position 2λ.

Table 6 present the 3-h extreme values of the bending moment by the Weibull-Tail approach (ZURITA, 1999) for the three "*100y*" cases. As can be seen, for cases 3 and 6, the differences between extreme values are more significant than those observed for the maximum values. As was previously mentioned, in case 6 was found a difference in the average value of bending moment, this difference could contribute and increase the error in calculating the extreme values.

*Table 6- Extreme dynamic bending moment (3-h) by Weibull-Tail distribution (kN*m) obtained by FEM program and analytical methodology proposed.*

Case	Position	-λ		0λ		λ		2λ		3λ		4λ	
	Parameter	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical
3	*3-h extreme*	49	46	54.3	52.9	60.3	63.8	62.7	77.9	76.1	85.4	85.3	90.4
6	*3-h extreme*	80.5	78.9	99.5	102.7	130.5	142.9	170.3	181.8	194.3	204.9	183.4	217.8
9	*3-h extreme*	28.1	31.4	48.7	50.1	57.4	57.6	62.8	62.1	65.8	64.9	66.6	66.1

Figure 9 show the cumulative probability function of extreme peak values of bending moment for case 3 in *2λ position* using the Weibull-Tail distributions. When comparing the results obtained by the analytical model, for the FEM program, it can be seen high levels of accuracy.

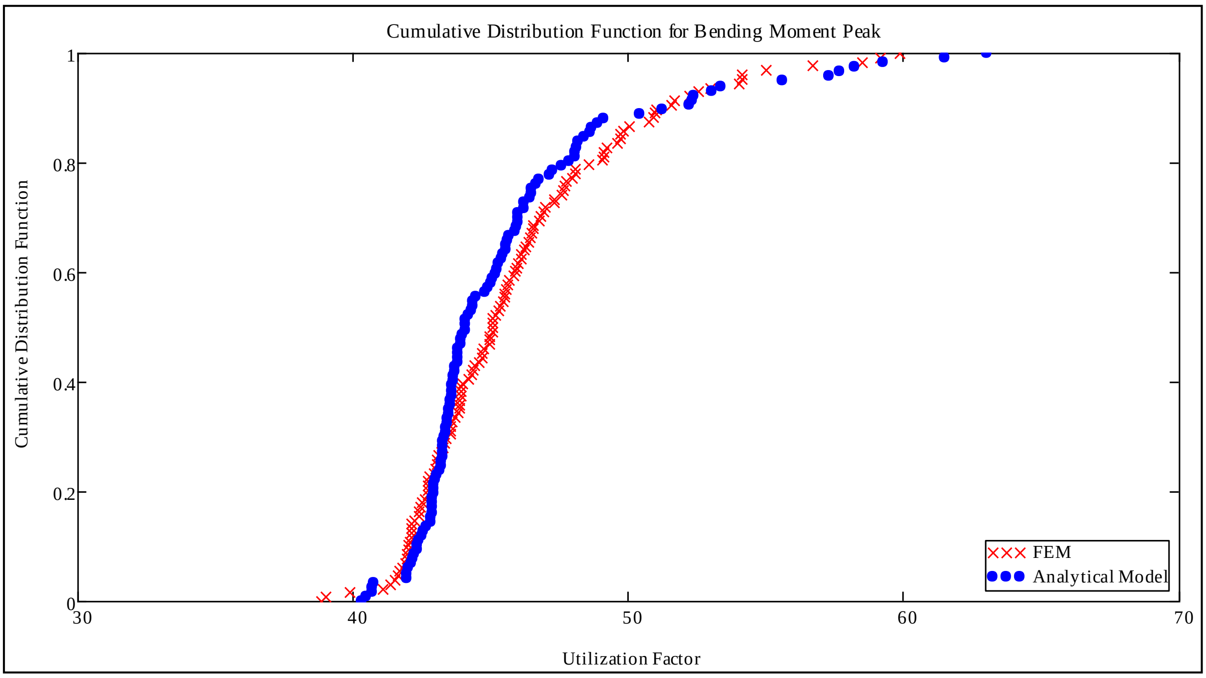

Figure 9 – Cumulative Distribution Function for bending moment peak, Case 3, position 2λ.

Finally, when calculating the short-term utilization factor associated with these cases, it is observed than the influence of the error in the evaluation of dynamic bending moment series is reduced.

Table 8 present the more likely extreme values in short-term factors (DNV-OS-F201), also calculated according to the Weibull-Tail approach, assuming a normal safety class and serviceability limit state (SLS). As can be seen, the factor obtained were low, depending on the riser's thickness, but the differences between the analytical model values and those obtained by FEM program are around10%.

Table 7- Short-term utilization factor for bending moment (100y).

Position	**-λ**		**0λ**		**λ**		**2λ**		**3λ**		**4λ**	
Case	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical	FEM	Analytical
3	0.15	0.15	0.17	0.17	0.19	0.2	0.21	0.24	0.23	0.26	0.26	0.27
6	0.24	0.24	0.3	0.31	0.39	0.42	0.5	0.53	0.57	0.6	0.54	0.63
9	0.09	0.1	0.15	0.16	0.18	0.18	0.19	0.19	0.2	0.2	0.2	0.2

Even considering that just a few cases have been analyzed to the rigid riser (9 cases), a long-term estimate was made for obtaining the "100y" utilization factor, exclusively aiming to verify if the values obtained through FEM program short-term data are in agreement with the values obtained through the Analytical model presented here.

Therefore, it was supposed that all the short-term cases analyzed show the same occurrence frequency and that the points -*λ* to 4*λ* from all cases are coincident, which is not real because the TDP of each individual case are shown in distinct positions. Table 8 introduces the positions -*λ*, 0*λ* and *λ* results. One may notice that the results were quite similar despite the difference noted for position *λ*, in which the analytical model results were slightly conservative.

Table 8- Long-term utilization factor for bending moment (100y).

Position	**FEM Model**	**Analytical Model**
-λ	0.33	0.3
0	0.34	0.34
λ	0.46	0.53

5 FINAL REMARKS

The objective of this work is to present a semi-analytical methodology to calculate short-term responses at the TDZ risers, using the model proposed by ARANHA et al. (1997, 2001). In future works, this model will be used to evaluate long-term responses at the TDZ.

The application of the proposed methodology is exemplified by a case study where a rigid riser connected to a spread moored FPSO was analyzed. The results obtained using the proposed analytical model can be considered good approximations of the time domain analyses results. However, the analytical model has significant limitations associated primarily with the exact position of the TDP, limiting for now the use of this methodology to rigid risers. For flexible risers, even small errors when evaluating TDPs positions can generate great differences between results, as flexural stiffenesses are quite low.

This methodology can be applied in the initial phases of the riser design, accelerating the selection of a structure that satisfies the design criteria, because in this case the computational cost is lower.

Finally, the main goal of this research is to get long-term DNV utilization factors for SCRs. The proposed methodology is being implemented in a computer program called "MYLongTerm" in Laboratory of Analysis and Reliability of Offshore Structures, (LACEO/COPPE).

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