XLI CILAMCE
XLI Ibero-Latin American Congress on Computational Methods in Engineering
16-19 November, 2020 FOZ DO IGUAÇU PARANÁ - BRAZIL

Fixed offshore platform: assessment on the possibility of reuse of the jackets for offshore wind farms

Vencislau Manuel Quissanga[1], José Guilherme Santos da Silva[1]

[1]*Civil Engineering Postgraduate Programme (PGECIV), State University of Rio de Janeiro (UERJ)*
São Francisco Xavier St., Nº 524, Maracanã, 20550-900, Rio de Janeiro/RJ, Brazil
venmanquissan@gmail.com, jgss@uerj.br

Abstract. This paper focuses on the possibility of reusing (or not) a fixed offshore platform, with the purpose to understand whether the removal of the deck from the platform (partial decommissioning) would be considered including the replacement for a wind tower. Thus, the present research aims to verify the structural behaviour of a fixed offshore platform (jacket) when subjected to the loading of a 10 MW wind tower. However, in addition to the static loading coming from the wind tower, the environmental forces (variable loads) related to the waves and wind acting on the platform were pondered. The loading related to the 10MW wind tower was considered and distributed on top of the four legs of the jacket. The investigated structural system presents a 26 m height jacket (from the mudline); the lower area presents dimensions of 9.97 m x 9.97 m and the upper area 6.10 m x 6.10 m. The structure presents eight legs and three deck elevations between the upper and lower deck. The numerical model was developed based on the use of the SACS V12.0 computational software, and via the use of the Finite Element Method (MEF). Therefore, the resistance analysis is performed on the fixed offshore platform (jacket) considering the influence of the static and variable loadings in order to assess the current structural behaviour.

Keywords: fixed offshore platform, offshore wind farms, finite element modelling.

1 Introduction

Oil and gas industry has recently faced numerous problems related to the inoperability of many of its offshore platforms installed more than two or three decades ago, as the increase in operating costs followed by market competitiveness has directly influenced the economic viability of the oil sector. As a result, alternatives seeking to reuse out-of-operation platforms can be implemented, such as the replacement of the decks by wind power tower stations, in order to partially take advantage of the existing structure through adaptations.

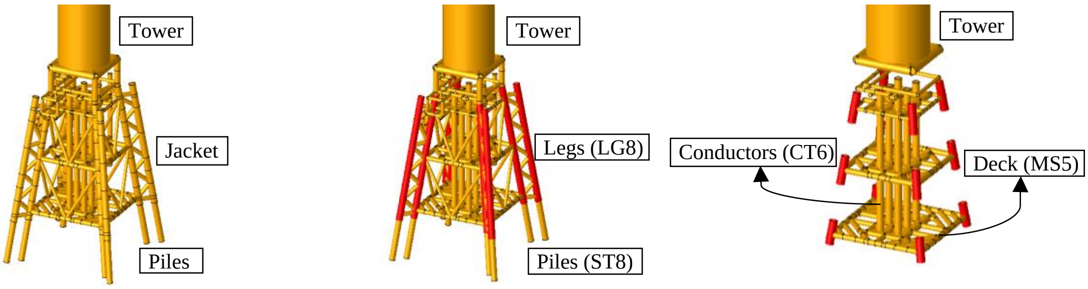
The platform (jackets) reuse results in one of the alternatives to maintain its operation since they are not profitable in the production of hydrocarbons to guarantee financial viability. Therefore, Brazil has a high concentration of platforms which are out of operation, mainly in the north-eastern region where there is a considerable incidence of winds, generating a favourable environment for the installation of wind towers [1].

Hence, the present research work aims to evaluate the behavior of the structural system of a typical Brazilian fixed offshore platform (jacket) when subjected to the loading of a 10MW (Fig. 1) wind tower and new operating conditions without the need for a major reinforcement that makes its reuse meaningless. The jacket consists of five decks, with the lower area (deck) of 9.97x9.97m^2 and the upper area of 6.10x6.10m^2. The numerical model was developed using the Finite Element Method (FEM) based on the use of the SACS V12.0 [2] program. Variable loads (wind, wave and current) are considered by simulating the constant impact on the structure. Results obtained throughout the study point to the fact that the investigated jacket does not meet the design criteria to support 10 MW wind towers.

2 Investigated structural model

2.1 Description of the structural model

The structural model corresponds to a fixed offshore platform (jacket-tower) which includes the jacket, the conductors and the piles. It consists of a vertical structural system of tubular steel sections fixed to the seabed, with a 10MW wind tower attached to the top of the jacket. The jacket has 26 m high geometry, the lower area being 9.97 m x 9.97 m and the upper area 6.10 m x 6.10 m; and with three trays between the top and the bottom, according to Fig. 1. The geometric properties of the structural elements consist of bars with diameter and thickness respectively: Legs (LG8) 863.60 mm and 28.60 mm; Conductors (CT6) 762 mm and 25.40 mm; Diagonals (DV and DH) with 558 mm and 21.45 mm, 586.40 mm and 20.50 mm, 406.40 mm and 21 mm; Decks (MS5) with 609.60 mm and 21.85 mm; Piles (ST8) 762 mm and 38.10 mm. The piles are constituted by cross-sectioned elements smaller than the legs of the jacket, and of equivalent lengths inserted between them, driven into the soil, transferring the effort from the jacket to the deeper layers of the soil.



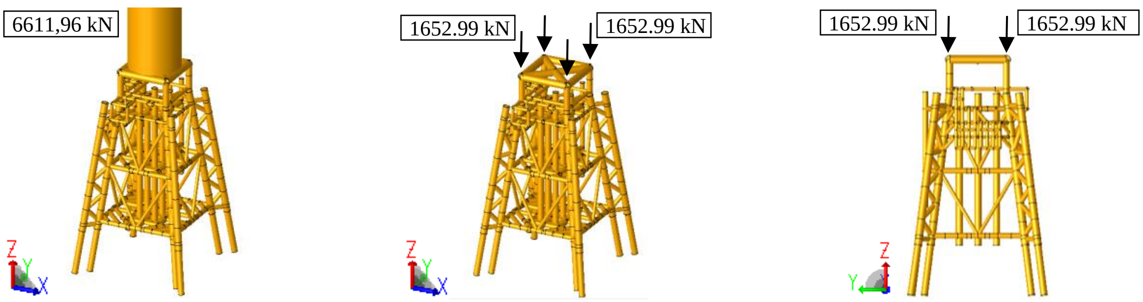
a) Overall jacket model b) Jacket legs and pegs c) Decks and jacket conductors

Figure 1. Structural model of the investigated tower jacket

Regarding the physical properties of the materials, steel has a yield stress (f_y) of 248 MPa, an elastic modulus (E_s) of $2.0\text{x}10^5$ MPa, a Poisson's ratio (ν) of 0.3 and a density (ρ) of 7850 kg/m³. The jacket was limited to a load capacity of around 2000 tf, being compatible with the weight of a 10 MW wind power tower (Quissanga, 2018).

2.2 Active loads

The modelled own weight ("PPM") of the structure (jacket-tower) was automatically generated by the SACS V12.0 program [2], from the geometric characteristics and the specific weight supplied to the program (γ = 78.5 kN/m³). In the structural model (Fig. 2), the loading of the tower was considered as acting punctually on the four knots at the top of the jacket. In addition, with the same program, the thrust loading was determined, being in the order of 6611.96 kN, taking into account the volume of the submerged structural elements.



a) Jacket under loading (3D) b) Jacket under punctual loading (3D) c) Jacket under loading (profile 3D)

Figure 2. Offshore platform Jacket under environmental and wind turbine tower load

Then, Tab. 1 and 2 present some of the characteristics of the marine environment; such as depth, from the mudline to the surface (water depth), the water density and the weights modelled on the structure. The gravity centre of the investigated structural system is also included in the table.

Table 1. Description of the marine environment

Gravity	Water depth	Mudline elevation	Water density
Direction	(m)	(m)	(t/m^3)
-Z	16	-16	1.028

Table 2. Modelled own weight of the "PPM" structure

Water depth	Jacket PPM	Tower PPM	Total PPM	Gravity centre		
(mm)	(kN)	(kN)	(kN)	X (cm)	Y (cm)	Z (cm)
16000	3069.58	6611.96	9681.54	-35	15	-210

With reference to overloads ("SCARGA"), the recommendations in section 3 (Loads and Load Effects) of the DNVGL-OS-C201 [3] standard were considered, which prescribes the use of distributed load over the topside surface. Thus, a distributed load ("EQUIP") was applied over the free area of the jacket top corresponding to 4 kN/m² (in operating condition), even with the topside removed for greater safety. However, the impact of wave forces on the model was considered using the formulation of Morison [4], in three directions (0^0, 45^0 and 90^0) eq. (1), which considers the sum of a force resulting from hydrodynamic pressures with intensity proportional to the acceleration of the fluid mass (inertial forces) and a force of viscous origin proportional to the speed of the fluid particles (drag forces). Tab. 3 presents the description of the wave and the current in the direction of 0^0, 45^0 and 90^0, considering the load cases, respectively.

$$F = C_d \frac{\rho}{2} D|u|u + C_m \frac{\pi}{4} D^2 \rho \dot{u} \tag{1}$$

Table 3. Description of wave and current loads

Case	Wave type	Condition	Load				Wave incidence
			Wave		Current (m/s)		
Wave case		Structural situation	Height (m)	Period (s)	Sea surface speed	Speed under the sea	(Degree)
OPE0	5º Stoke	Operation	5.90	9	1.35	0.48	0
OPE45		Operation	5.90	9	1.35	0.48	45
OPE90		Operation	5.90	9	1.35	0.48	90
OPE0, OPE45 and OPE90: wave and current operating in directions 0^0, 45^0 and 90^0.							

Wind forces were calculated based on the basic speed of 25.7 m/s, taken at 10 m above the water depth. The total design tide was taken according to API RP 2A-WSD [5], and the determination of the water depth was 7.25 m. Thus, the water level for the application of wave and current loads was determined based on the sum of the water line and the total tide, resulting in 17.25 m. Next, based on the use of eq. (2), taken from the API RP 2A-WSD [5], the drag force (F) was calculated. Shape coefficient used for the type of structure in question is equal to 1, as recommended by the API RP 2A-WSD [5]. F being the drag force in (kN), ρ the air density (kg/m3), V the wind speed (m/s), Cs the form factor and A the projected area in the wind direction (m^2).

$$F = \frac{\rho V^2 C_s A}{2} \tag{2}$$

Wind loads, which act on the obstruction areas of the structure in different directions, are described as: in the direction of 0^0 (VOP0) 58.40 kN; at 45^0 direction (VOP45) 41.30 kN; and the direction of 90^0 (VOP90) 39 kN. Following, in Table 4, a global summary of the loads (F and M) used in the analysis is presented.

Table 4. Global sum of loads

Basic loading	FX (kN)	FX (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
PPM	0	0	9681.54	8288.70	17851.22	0
SCARGA	0	0	953.14	1121.50	980.85	0
EQPM	0	0	804.30	2240	1617.90	0
VOP0	-58.40	0	0	0	-1050.50	174.9
VOP45	-41.30	-41.30	0	-1042.82	748.22	-423.67
VOP90	0	-39.3	0	709.56	0	0

The Tab. 5 shows the load combinations used in the analysis for operating conditions. The Allowable Stress Method (ASD - Allowable Stress Design, WSD - Working Stress Design) was considered, as in current jacket design practice it is, therefore, the main criteria to be used [6]. Table 6 shows the global sum of the load combinations used in the investigation. The resulting moments are calculated based on the (0,0,0) point of the model.

Table 5. Load combinations used in the analysis

Operation		Contingency factor	Combinations		
Loads considered	Loading directions		0^0	45^0	90^0
PPM	Own weight Z	1.15	1.20	1.20	1.20
	Own weight X		0.065	0.055	-
	Own weight Y		-	0.056	0.077
EQPM	Operation Eq. Z	1.15	1.20	1.20	1.20
	Operation Eq. X		0.065	0.055	-
	Operation Eq. Y		-	0.056	0.077
SCARGA	Overload Z	1.15	1.05	1.05	1.05
	Overload X		0.06	0.05	-
	Overload Y		-	0.05	0.07
VOPER	Wind X	1.15	1	0.71	1
	Wind Y		-	0.71	0.71

Table 6. Sum of load combinations

Load Combination	FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
OPE0	-4858.10	16.20	13997.85	45550.20	8798.15	998.51
OPE45	-4032.40	-4001.90	14569.40	43214.70	9895.24	3777.12
OPE90	17.70	-4894.20	14579.96	43904.40	7969.11	5244.9
AX0	-7522.30	20,40	1002.70	20701.62	575.84	4978.76
AX45	-5766.94	-5695.67	10784.48	21947.45	3608.65	5416.71

OPE0, OPE45 e OPE90: wave and current operating in directions 0^0, 45^0 and 90^0.
AX0 e AX45: axial in directions 0^0 e 45^0.

3 Structural analysis: results and discussion

Results of the numerical analysis of the investigated structural model were obtained considering the criteria based on the in-service stresses, the admissible stresses method, currently in almost all countries, including Brazil. The API RP 2A-WSD [5] code recommendations are adopted and API RP 2A [6] was chosen for this type of analysis, specifically, as it is one of the most relevant and specific codes for jacket type offshore structure projects.

Throughout the analysis, the "Unity Check" (UC) was used, which consists of a specific command of the

SACS V12.0 [2] software used for the verification of the structural elements, according to the provisions of the structural design standards steel API RP 2A [6], which allows the calculation of the maximum stress values and determination of critical sections along each element. The UC is represented by eq. (3); where Rn is the nominal resistance of the structural element, Q the efforts acting on the element resulting from the various actions, γ/∅ the safety factor (FS), including resistance reduction and action increase.

$$\frac{\emptyset R_n}{\gamma} \geq \sum Q. \tag{3}$$

4 Stress analysis

From the evaluation of the analysed results (numerical) of axial flexural stresses carried out using the software SACS V12.0 [2], Fig. 3, it is possible to verify through the UC's, that the jacket supports the imposed loads, as it is verified that the values of the acting voltages are lower than the allowable bus voltages (UC < 1). Thus, the members and the joints meet the design criteria, based on the values of the voltages in service, indicating that the structure is adequate to support the different loads, especially the 10 MW wind tower. Fig. 3 shows the most requested elements located in the lower region of the jacket and the element subjected to maximum stress (DH12) has a tubular section of 609.60 mm in diameter and 21.85 mm in thickness.

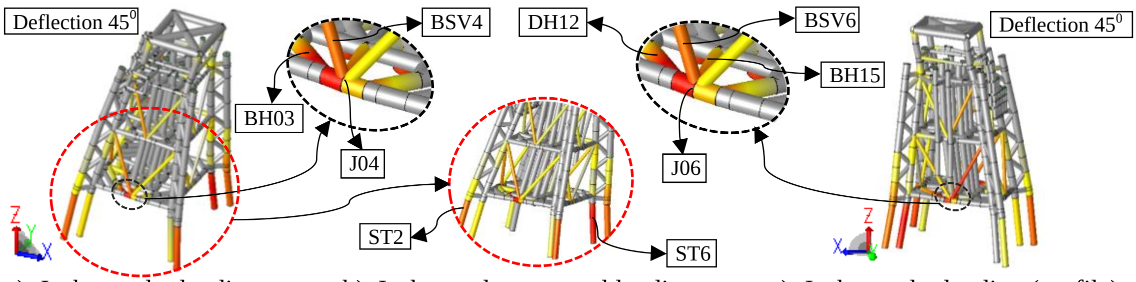


Figure 3. Maximum stresses in the structural elements of the analysed jacket

The calculation of the UC's parameters, for the evaluation of the most requested structural elements (DH12: see Fig. 3) is based on the AISC [7] code. However, the members that presented higher stress ratios (close to the unit) are shown in Tab. 7, following the AISC [7] and API RP 2A-WSD [5] criteria.

Table 7. Maximum axial flexural stresses per group of members

Critical member	Load	Unity Check	Applied stress (kN/mm^2)			Allowable stress (kN/mm^2)		
			F_a	F_{by}	F_{bz}	F_a	F_{by}	F_{bz}
BH03	COPE0	0.85	-54.27	18.18	241.87	247.96	-327.72	-327.72
BSV4	COPE0	0.74	187.07	-18.85	7.69	248	-297.3	-297.3
DH12	COPE45	0.95	-79.48	20.75	239.13	247.16	-327.66	-327.66
BH15	COPE45	0.76	-70.61	-14.31	-182.01	247.33	326.37	326.37
BSV6	COPE45	0.87	-138.03	-36.77	-26.23	203.24	-336.51	-336.51

BH03, BSV4, DH12, BH15 and BSV6: identification of members in Figure 3
COPE: load combination operating in directions 0^0 and 45^0.

Hence, since no member had a UC above 1, it can be considered that they present satisfactory structural behavior. In addition to the focus on stress analysis of the legs, decks, and diagonals, the verification of the connections' puncture among these elements was also considered, based on the criterion of the active and admissible stresses of the API RP 2A-WSD [5]. Tab. 8 shows the results for the joints with the highest Punching UC found throughout the study.

Table 8. Joints with higher stress ratios (UC) of punching

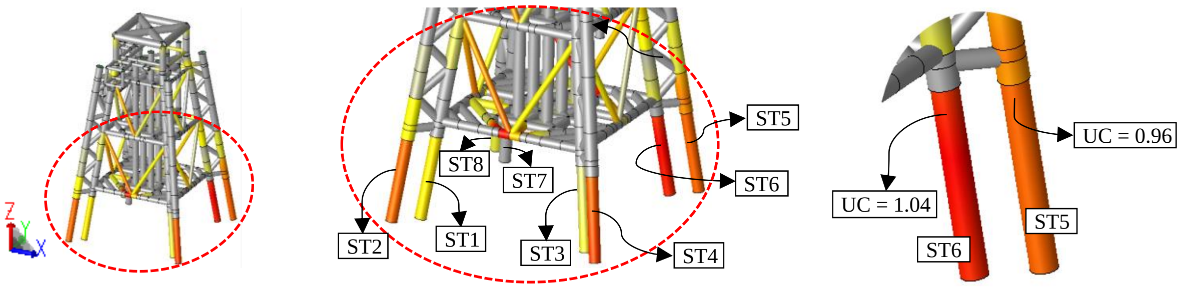
Structural model	Joint	Vertical support	Horizontal support	Punching UC	Combined load
Jacket	J06	P15	DH12	0.89	OPE45
	J06	BSV6	--	0.78	OPE0
	J04	--	BH03	0.84	OPE45
	J04	--	DH23	0.70	OPE0

4.1 Load capacity of piles

The load capacity value of the piles was calculated based on the API RP 2A [6] criterion. This load capacity is given by the most resistant lateral friction of the tip, according to eq. (4); being k_i coefficient of lateral pressure, p_s effective pressure, δ friction angle between soil and pile and N_q tip capacity factor.

$$Q = Q_s + Q_p = A_s f + A_p q = A_s k_i p_s tg(\delta) + A_p p_s N_q \tag{4}$$

Based on the variation in the lateral load capacity of the pile, and depending on the penetration (28.10 m), the area was integrated to obtain the total load capacity, performing the sum of the side friction load (Q_s) with tip resistance (Q_p), totalling a compression capacity of 13694.28 kN, with a tensile load (Q_t) of 6721.54 kN. In Fig. 4, it can be seen that ST6 and ST2 stakes have UC above 1, indicating that they are overloaded.



a) Jacket under loading b) Jacket under punctual loading c) Jacket under loading (profile)

Figure 4. Piles under jacket and tower loading

Next, Tab. 9 presents the results of the piles alone in terms of safety factor, taking into account the sum of the side friction load with the tip resistance and the tensile load of the piles. However, six out of eight piles meet the project conditions, while the remaining two (ST2 and ST6) do not attend the requirements.

Table 9. Results in terms of safety factor (FS) of piles

Piles	Member	Load to compression capacity (kN)	Traction load capacity (kN)	Active loads (kN)	FS for compression (FS > 2)	Result
ST1	M015	13694,28	6721,54	3951.02	3.47	> 2 Ok
ST2	M027			**6979.9**	**1.96**	**Unacceptable**
ST3	M019			2147.89	6.38	> 2 Ok
ST4	M023			4228.65	3.24	> 2 Ok
ST5	M017			5436.83	2.52	> 2 Ok
ST6	M062			**7089.63**	**1.93**	**Unacceptable**
ST7	M025			738.34	18.55	> 2 Ok
ST8	M021			1745.16	7.85	> 2 Ok

Based on the analysis performed, it was verified that the maximum load produced by the tower jacket in operational condition was 7089.63 kN. The FS in operation is equal to 2, and the resistant pile loading is 13694.28 kN, so the corresponding value of the UC check results in 14179.26/13694.28. Therefore, two of the eight piles are overloaded, as they presented UC above 1.

4.2 Analysis of the displacement values

Displacement values were checked based on DNVGL-OS-C101 [8] (Tab. A1 Travel limits), for the service limit state. The maximum deformation found throughout the study via report of the SACS V12.0 [2] program shows that the maximum value is 18.92 cm (δ_{max} = 18.92 cm). The length of the associated element is 800 cm (L/200 = 4 cm), and the verification for the displacement values is not satisfactory (δ_{max} = 18.92 cm > δ_{lim} = 4 cm).

5 Conclusions

This research work presents the results of the structural analysis of a typical Brazilian platform (jacket), when subjected to new operating conditions including the positioning of a 10MW wind tower at the top. The conclusions of this study are intended to alert professionals (engineers) from the offshore industries not to be compliant of the possible reuse of platforms that are out of operation when subjected to the loading of a tower of 10 or more MW. However, based on the results presented in this research, the following can be concluded:

1. Analysed structural members of the jacket (braces) have demonstrated an adequate structural behavior, as their stress ratios (UC) result in values below the unit, with a maximum value of the UC equal to 0.95.
2. However, the piles presented results of stress ratios higher than the unit (UC > 1) which shows that the 10MW wind tower cannot be used in a jacket of a typical Brazilian fixed offshore platform, considering the fact that the overturning moments have increased much more than the corresponding weight loads.
3. Due to the maximum displacement values of the structural system, it is emphasized that they do not meet the design limit (δ_{max} = 18.92 cm > δ_{lim} = 4 cm) recommended by the DNVGL-OS-C101 [8], as far as the analysis of the combined loads and the different design parameters are concerned.
4. Despite these conclusions being closely related to the Brazilian reality, as it refers to a typical platform used in Brazil, it can be seen as an alert for platforms both in Brazil and other countries, since it clearly shows that the platforms (jackets) subjected to new operating conditions, including the positioning of wind tower (10 MW or greater), needs to be rigorously evaluated aiming the possibility of reuse.

Acknowledgements. The authors gratefully acknowledge the financial support for this research work provided by the Brazilian Science Foundation's CNPq, CAPES and FAPERJ.

Authorship statement. The authors hereby confirm that they are the sole liable persons responsible for the authorship of this work, and that all material that has been herein included as part of the present paper is either the property (and authorship) of the authors, or has the permission of the owners to be included here.

References

[1] V. M. Quissanga. Descomissionamento de plataforma offshore fixa e utilização para base de unidade de geração de energia eólica. MSc Dissertation. Fluminense Federal University (UFF), Rio de Janeiro/RJ, Brazil, 2018.
[2] SACS V12.0: Structural Analysis Computer System. Engineering Dynamics, Inc-Kenner, Louisiana, USA, 2019.
[3] Offshore Standard DNVGL-OS-C201. Structural Design of Offshore Units (WSD). Norway, Edition July, Norway, 2015.
[4] Recommendations of the Committee for Waterfront Structures Harbours and Waterways, 2012.
[5] API (American Petroleum Institute). *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms* (API RP 2A-WSD). Working Stress Design. 1st Edition, Washington D.C., USA, 2015.
[6] API (American Petroleum Institute). *Recommended Practice for Planning, Designing & Constructing Fixed Offshore Platforms* (API RP 2A). Working Stress Design, 22nd Edition, Washington D.C., USA, 2014.
[7] AISC (American Institute of Steel Construction). *Manual of Steel Construction*. Allowable Stress Design. 9th Edition, Chicago, USA, 2016.
[8] Offshore Standard DNVGL-OS-C101. Design of Offshore Steel Structures. General (LRFD Method). Edition July, Norway, 2015.