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3D modeling and calculation of the fixed platform PFS-U (PESCARUS)

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Summary

Fixed platforms for the exploitation of petroleum deposits in the continental Black Sea Romanian platform are usually built like 3D welded truss structures (called "jacket"), using circular hollow sections. The structure has lateral inclined faces and it is grounded using steel driven piles disposed at the interior part of the jacket columns. This construction solution asks that the drilling can be done through the jacket slots only after the placement of the offshore into position.

For the "PESCARUS" location a fixed 4-faces platform was designed, in a new solution, having an open jacket at one of the faces (this was called "Fixed support platform PESCARUS"). This structure should allow the earlier drilling of the offshore wells (using a self-lifting platform) and in this case the construction of the PFS-U platform can be done at a later time.

For the design and calculation of this platform a full finite element 3D model was built. The model takes into account the real dimensions of element cross sections, the stiffness of the structure and the structure-soil interaction respectively. For the evaluation of the dynamic response of the structure a 3D dynamic finite element model was considered, the position of masses on the structure being very close to the real one.

This paper present important aspects regarding the finite element model, the evaluation of the location specific actions and loading hypotheses, but also the resistance and stability checks according to the accepted norms.

KEYWORDS: platform, jacket, structure-soil interaction, dynamic response, loading

1. INTRODUCTION

The fixed marine platforms in the petroleum deposits of the Black Sea erected for water depths between 40.0 and 60.0 m consist, generally speaking, in two parts:

ISSN 1582-3024

- a 3D welded truss structure (called “jacket”) with circular hollow sections, which is transported into position through floating, is placed in vertical position by bearing on the sea floor and then is fixed using steel driven piles disposed at the interior part of the jacket columns. The jacket height is established according to the water depth, so that his upper part reach the level +3.00 m above the water level (figure 1)
- a decks system, erected each like 3D steel parts above the jacket level, using columns and counterbraces. Those will sustain the equipments and materials which are necessary for the oil extraction technology (figure 1).

The jackets of the Romanian marine platforms were designed with 3, 4 and 6 steel columns and are called according to this number: tripod, tetrapod and hexapod respectively. In the same time the marine platforms are called according to their destination or functions in the oil extraction system::

- fixed platform flame support (tripod type);
- fixed platforms with offshore wells “PFS” for sustaining wells tubes (tetrapod type);
- central fixed platforms “PFC” for sustaining the installations necessary for the function and survey of the petroleum extraction equipments (hexapod type);
- platform for the social group and utilities “P.G.S.U.” (de tip hexapod).

Generally, for the marine platforms of P.F.S type, the drilling of the offshore wells is made through the jacket slots after the placement of the platform into position and this has as consequence a longer time for cosntruction. For the fixed platform sustaining offshore wells PFS-U PESCĂRUŞ a special jacket was designed, in order to allow the assembling into location of the platform after the previous drilling of the offshore wells. This procedure lead to a lower value of the self weight of the platform and in this way the transport into location and the assembly of the structure could be done with equipments existing in Romania. The platform PFS-U has a tetrapod type jacket, with steel driven piles and has the following characteristics:

- decks system has 3 levels (+20.500, +16.500, +12.500), with a cantilever structure at the first two, at the face side II;
- the jacket is U-type (with face IV open, without braces and horizontals), other 3 side faces being foreseen with braces and horizontals; all side faces are inclined;
- the open side face will be closed in the future with braces, at the upper part, between levels +3.00 and +1.00, after the installation of the jacket into position;

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The design and the final solution of the oil platform was made in two stages as a result of the revision in 1999 of the thicknesses for the marine growth, but also for the values and positions of the loading on the decks.

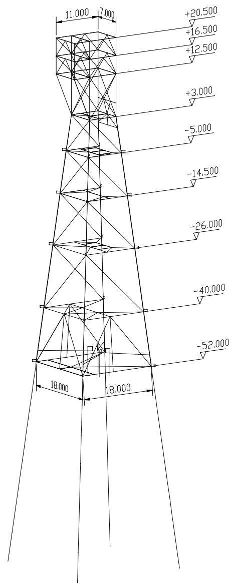

Figure 1 – The general scheme of the offshore wells platform PFS-U

2. FINITE ELEMENT MODELLING OF THE STRUCTURE

The finite element modelling of the fixed offshore wells platform PFS-U PESCĂRUŞ consist in a 3D model, with respect of the real geometry of the structure, of the structural elements stiffness, but also by taking into account the "jacket-piles-soil" interaction for the specific conditions of the platform. Thus, for the modelling of the jacket, decks and driven piles, two nodes straight beam elements were used (FRAME elements) and the steel plates at each deck level and at the level of the horizontal frames where the offshore wells are fixed in the slots were modelled using area elements with four nodes (SHELL).

At the upper part of the jacket (at level +3.00) the piles ends are directly welded at the corresponding columns and for this reason, at this level, the ends of piles and columns are connected through a common joint. In the model, the piles are disposed parallel to the jacket columns at the maximum distance allow by the movement between the pile and movable devices at the interior part of the pile (guiding devices).

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The working connection between piles and jacket is modelled with elastic small frames placed at the joints of the jacket bellow level +3.00 and dimensioned in such a manner to allow elastic independent deformations between piles and jacket. This solution lead to the values of the horizontal displacements at each pile joints correlated with the displacements of the piles at the interior part of the columns and with the displacements of the jacket joints under loads. The calculation is made in some successive steps, for a applied load on the pile about 1000 to (value which is near to the maximum capacity of the pile), establishing in this way the cross section characteristics of the finite elements forming the frames according to their real stiffness. The first two elements of the working frames (the horizontal element at the bottom of the frame and the vertical element) results with a circular cross section having a 0.428 m diameter, and the top horizontal element, which allow relative independent displacements along the piles, between piles and jacket, was designed as a steel plate, having a very high stiffness value in horizontal plane and very small stiffness value in vertical plane (figure 2).

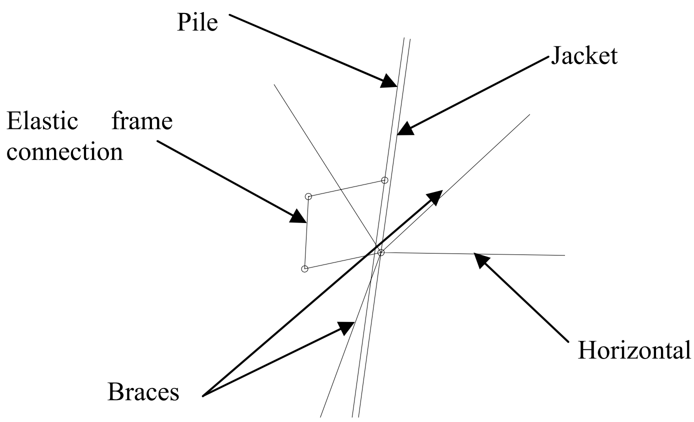

Figure 2 – Detail for the elastic frame connection between pile and jacket

The soil piles bearing is made through straight beam elements with two nodes which can take only axial force (pin ended beams) and having a unity length. These elements are placed along the piles at distances established according to each soil layer depth. The variable cross sections of these frames comes out from the condition that the settlement (which is dependent on the coefficient of soil reaction at the level of the corresponding frame element) under the pressure produced by the soil on the interaction surface of a pile from a unity load is equivalent with the axial deformation produced by the same load to a pin ended beam having 1m length, cross section ***A*** and elasticity modulus of the material ***E*** (2.1x 10^7 to/m^2 for steel).

ISSN 1582-3024

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The values for the coefficients of soil reaction k_h [kN/m^3] for each characteristic layer and their variation on the depth were established according to the physical-mechanical characteristics of the soil layers existing on the piles length.

3. LOAD HYPOTHESES FOR RESISTANCE AND STABILITY CHECKS

The loading hypothesis corresponds to the demands in Section C, "Loads" of API - RP 2A-LRFD and API RP 2A-WSD:

a) Hypothesis Q1 = 1,3(D1 + D2) +1,5(L1 + L2) where:

D1 = self weight of the structure and equipments permanently mounted on the platform;

D2 = self weight of the equipments mounted on the platform. Their position can change according to the mode of operation;

L1 = includes the weight of consumable supplies and fluids in pipes and tanks;

L2 = live load exerted on the structure from operations such as lifting by cranes, machine operations vessel mooring etc.

b) **Hypothesis Q2 = 1,1(D1 + D2 + L1) +1,35(We + Dn)** where:

D1; D2; L1 have been defined at point a);

We = sectional stresses in the analysed structural element produced by wind, wave and current loads action, in the most disadvantageous situation.;

Dn = sectional stresses in the analyzed frame coming from earthquake action (only for structures having a fundamental period over 3 sec.).

Taking into account the from, in plane, of the platform (figure 3) and the presence only of a load type L2, insignificant as value, the following load cases of the jacket, for resistance and stability checks, are considered:

a) <u>For permanent actions D1+D2 and L1 type</u>

Case 1: Loads coming from D1 and D2;

Case 2: Loads coming from L1 (having load L2 included).

b) <u>For loads coming from wave actions H=14.1 m, current and wind</u>

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Case 1: Wave forces H=14.10 m, wind and current θ = 0° with respect to Y axis, (W_e for θ = 0°)

Case 2: Wave forces H=14.10 m, wind and current θ = 45° with respect to Y axis, (W_e for θ = 45°)

Case 3: Wave forces H=14.10 m, wind and current θ = 90° with respect to Y axis, (W_e for θ = 90°)

Case 4: Wave forces H=14.10 m, wind and current θ = 225° with respect to Y axis, (W_e for θ = 225°)

Case 5: Wave forces H=14.10 m, wind and current θ = 270° with respect to Y axis, (W_e for θ = 270°)

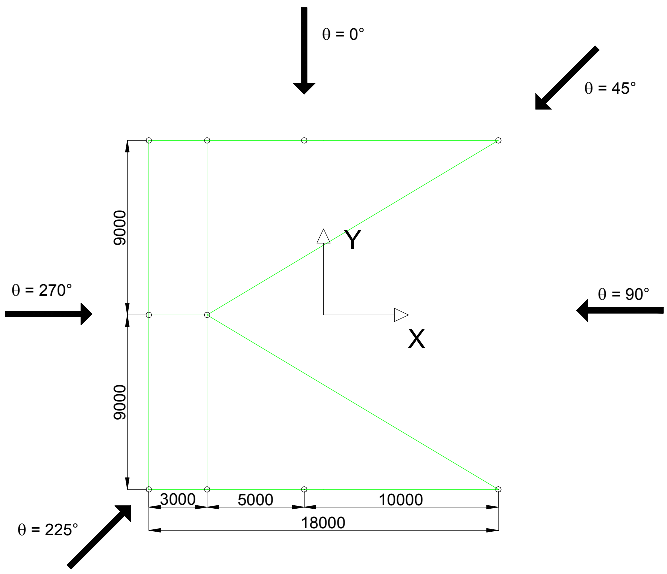

Figure 3 – Directions for wave and current loads

The sectional stresses (characteristic values) coming from these 2+5 loading cases are necessary to perform the resistance and stability checks for the structural elements of the jacket and for the resistance checks of the tubular joints respectively.

The maximum values of the sectional stresses are obtained by multiplication of the sectional stresses, determined according to the procedure presented above, through coefficients corresponding to hypotheses Q1 and Q2.

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4. EVALUATION OF LOADS COMING FROM D1, D2, L1, L2 AND WIND ACTION

The values and positions of the loads coming from D1, D2, L1 and L2 on decks were established through survey of the specific equipments existing on decks but also by considering the technical data concerning the volume and the self weight of some installations, including live loads. For the evaluation of wind loads, the appropriate exposed surfaces are calculated, inclusive the perimeter protection shields disposed at levels 12.500 and +22.500.

The exposed surfaces for the wind action directions (θ), are:

For θ = 0° and θ = 180° .... A_1 = 110.0 m^2

For θ = 45°,θ = 135° ; θ = 225° and θ = 315°........... A_2 = 137.0 m^2

For θ = 90° and θ = 270° ....A_3 = 56.0 m^2

The wind loading hypotheses are taken according to the extreme wave loading hypotheses θ = 0°, θ = 45°, θ = 90°, θ = 225° and θ = 270° and they were coupled with each wave loading direction according to the load cases 3, 4, 5 and 6.

5. THE EVALUATION OF MAXIMUM WAVE LOAD FORCES AND CURRENT FORCES

For the computation of the maximum values of sectional stresses in structural elements from wave and marine current action, necessary to form Q2 hypothesis, in order to perform the resistance and stability checks, the load components at the joints of the finite element model are used.

The forces corresponding to waves are computed for H=14.1 m water height with a period of 10.2 sec. For the forces coming from marine current, the current velocity was taken 1.10 m/s at the water surface and 0.28 m/s at sea floor, according to the data from ICIM Bucureşti, given for all 5 directions of extreme wave.

The considered marine growth have a thickness about 0.08 m/radius till a water depth of 25 m and a thickness of 0.04 m/radius below this level.

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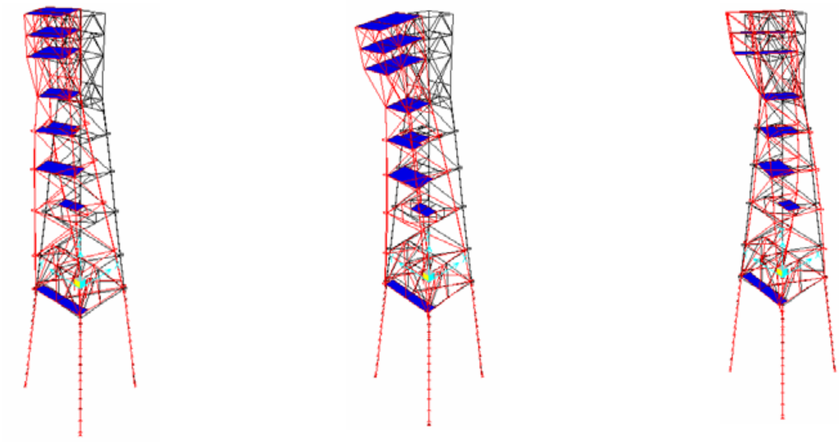

a) Mode 1 – T=1.207 sec. b) Mode 2 – T=0.985 sec. c) Mode 3 – T=0.686 sec.

Figure 4 – Deformed shapes from dynamic analysis

6. DYNAMIC MODEL AND EIGENMODES

The finite element model for the dynamic analysis has the same configuration like for the static analyses, but at joints, where the concentrated forces coming from dead loads acts (cases 1 and 2), are placed the additional masses corresponding to these loads. Using a linear eigenvector analysis, three eigenmodes are computed. The deformed shapes are presented in figure 4.

7. RESISTANCE AND STABILITY CHECKS

The design values for the sectional stresses, obtained through multiplication of characteristic values (from finite element analyses) with the corresponding coefficients (hypotheses Q1 and Q2) were used to perform the resistance and stability checks for structural elements and joints. In figure 5 is presented the distribution of axial force for jacket and piles under static loads.

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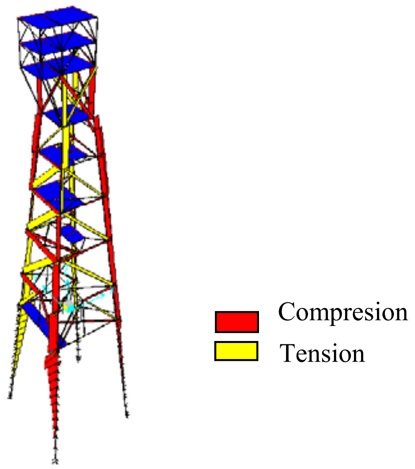

Figure 5 – Axial forces from wave and wind action (Case 1)

7.1 Tubular joints checks (API-RP 2A WSD)

a) The geometrical condition for tubular joints check in tension and compression:

$$\left[F_{yb}(\gamma r \sin\theta)\right] / \left[F_{yc}(11 + 1.5/\beta)\right] \le 1 \qquad (1)$$

where:

F_{yb}	the yield strength of the brace member
F_{yc}	the yield strength of the chord member
β, γ, r, θ	joint geometry parameters

b) The adequacy of the joint may be determined on the basis of:
Punching shear, considering the equations:

$$v_p = rf \sin\theta / \beta \le v_{pa} = Q_q \cdot Q_f \left(F_{yc} / 0.6\gamma\right) \qquad (2)$$

$$\left(v_p / v_{pa}\right)^2_{IPB} + \left(v_p / v_{pa}\right)^2_{OPB} \le 1 \qquad (3)$$

$$\left| v_p / v_{pa} \right|_{AX} + (2/\pi)\arcsin\sqrt{\left(v_p / v_{pa}\right)^2_{IPB} + \left(v_p / v_{pa}\right)^2_{OPB}} \le 1 \qquad (4)$$

In the above equations Q_q and Q_f are factors and *AX*, *IPB* and *OPB* means axial, in plane bending and out of plane bending moments respectively.

ISSN 1582-3024

c) **Nominal loads**, considering the equations:

$$N \le N_a = Q_q^N Q_f^N F_{yc} T^2 / (1.7 \sin\theta) \tag{5}$$

$$M_{IPB} \le (M_a)_{IPB} = \left(Q_q^M\right)_{IPB} \left(Q_f^M\right)_{IPB} F_{yc} T^2 (0.8d)/(1.7 \sin\theta) \tag{6}$$

$$M_{OPB} \le (M_a)_{OPB} = \left(Q_q^M\right)_{OPB} \left(Q_f^M\right)_{OPB} F_{yc} T^2 (0.8d)/(1.7 \sin\theta) \tag{7}$$

$$(M/M_a)_{IPB}^2 + (M/M_a)_{OPB}^2 \le 1 \tag{8}$$

$$|N/N_a| + (2/\pi)\arcsin\sqrt{(M/M_a)_{IPB}^2 + (M/M_a)_{OPB}^2} \le 1 \tag{9}$$

In the above relationships N, M_{IPB} and M_{OPB} are the axial force, in plane bending moment and out of plane bending moment respectively, N_a, $(M_a)_{IPB}$ and $(M_a)_{OPB}$ are the allowable values for the same stresses.

T thickness of the chord wall

d diameter of the brace

7.2 Buckling check of the structural elements

The buckling strength of the platform structural elements is checked according to the relationship given in Germanischer Lloyd's, Section 3, point 22.4:

$$(\Gamma_m N)/(kN_p) + \beta_m (\Gamma_m N)/M_p + \delta_n \le 1 \tag{10}$$

where:

Γ_m, k, β_m, δ_n coefficients

N elastic axial force in the brace

N_p, M_p plastic axial force and in plane bending moment in the brace.

8. RESULTS AND CONCLUSIONS

The analyses results, which are given for some elements in tables 1-3, have shown that for some structural elements, the checks are satisfied near to the limit value.

Table 1 Results for tubular joints checks

Joint / Frame	Equation number						
	1	**2**	**3**	**4**	**5**	**8**	**9**
Joint **10** Frame **204**	0.66 <1.0	321.4 < 411	0.012 <1.0	0.849 <1.0	294 to <376.2 to	0.012 <1.0	0.851 <1.0
Joint **19** Frame **205**	0.76 5 <1.0	342.5 < 411	0.018 <1.0	0.918 <1.0	271.3 to <325.2 to	0.018 <1.0	0.920 <1.0
Joint **31** Frame **206**	0.72 6 <1.0	**near limit 400.0 < 430**	0.012 <1.0	**near limit 0.999 <1.0**	**near limit 242.2 to <262.3 to**	0.011 <1.0	**near limit 0.992 <1.0**
Joint **39** Frame **207**	0.66 2 <1.0	122.0 < 470	0.007 <1.0	0.313 <1.0	55.1 to <216.6 to	0.006 <1.0	0.305 <1.0
Joint **60** Frame **88**	1.06 <1.0	58.1 < 421	0.008 <1.0	0.196 <1.0	16.4 to <121.1 to	0.007 <1.0	0.189 <1.0

Table 2 Results for buckling checks for jacket columns (chords) and horizontals

Frame type	Frame number	Buckling checking condition
Column	135	0.509 < 1.0
Column	148	0.261 < 1.0
Column	164	0.517 < 1.0
Column	167	0.278 < 1.0
Column	171	0.406 < 1.0
Horizontal	25	**1.03 < 1.0** (near limit)

Table 2 Results for buckling checks for braces

Frame type	Frame number	Buckling checking condition
Diagonal	136	0.513 < 1.0
Diagonal	143	**0.952 < 1.0 (near limit)**
Diagonal	151	0.352 < 1.0
Diagonal	205	0.794 < 1.0
Diagonal	206	**0.933 < 1.0 (near limit)**
Diagonal	207	0.450 < 1.0

The fundamental period of the structure resulting from the dynamic analysis is 1.207 sec. < 3 sec. so that the consideration only of the first 3 eigenmodes according to API is allowed.

The resistance checks results for the joints and also the buckling check results for braces shows that, reconsidering the position of some structural elements and also their dimensions, the check criteria in API are satisfied. For the joints where these conditions are satisfied near limit it is necessary to introduce a chord clutch with larger wall thickness in the joint region.

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