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In-Place Strength Evaluation of Existing Fixed Offshore Platform Located in Persian Gulf with Consideration of Soil-Pile Interactions

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ABSTRACT

Offshore jacket structures have been used in petroleum industry for decades. Due to increasing the age of operating platforms, structural damages will be generated by corrosion, fatigue, ship impacts and other reasons. Improvements in the oil and gas recovery from several fields have raised the interest for using these platforms well beyond their intended design life. Life extension of an existing jacket platform needs proper reassessment of its structural members, such as piled foundations. This paper represents a case study of the existing fixed offshore platform located in Persian Gulf by in-place strength analysis. The objectives of this analysis are to verify whether the platform can meet the structural requirements, as per API RP 2A and AISC, for the In-place extreme met-ocean loading. The structural assessment is performed based on the best estimates of the existing conditions of the structure data on the future corrosion allowance. Since the response of the jacket platform to the environmental loads is intensely affected by the pile soil interaction, in current study the foundation is modelled using uncoupled non-linear soil springs acting along the piles length. The load cases, which include all situations relevant in the In-place analyses are taken into account. Results of the In-place analysis of the drilling platform indicate that the jacket structure does not assure the code provisions.

1. Introduction

Offshore jacket structures have been used in petroleum industry for decades. Due to increasing the age of operating platforms, structural damages will be generated by corrosion, fatigue, ship impacts and other reasons. Improvements in the oil and gas recovery from several fields have raised the interest for using these platforms well beyond their intended design life. Extension of the life of an existing jacket platform needs satisfactory reassessment of its different structural members, such as piled foundations. The existing platforms in the Persian Gulf area have typically been designed for a design life of around 25 years. The age distribution of the jacket platforms of this area shows that a relatively large number of installations are been passed the 25 years. The outcome of reassessment determines the subsequent course of action. For example, if a pile is reassessed and found to be “unsafe”, structural intervention may be necessary or a new platform may be required. Both scenarios are very costly and this may eventually compromise the economic viability of a development. A rational reassessment of piles of existing platforms is therefore necessary to avoid costly solutions while ensuring that the underlying risk is as low as practically acceptable.

The process for assessment of existing platforms was proposed by Hugh Banon et all at 1994 [1]. Wisch et al [2] provided further background, clarification and proposal to update section 17 API RP-2A. API Recommended Practice 2SIM introduced to provide guidance to owner/operators and engineers in the implementation and delivery of a process to manage the structural integrity of existing fixed offshore platforms [3]. Reliability Based Assessment of Existing Fixed Offshore Platforms Located in the Persian Gulf was carried out by Alireza Fayazi and Aliakbar Aghakouchak [4]. A review of recent developments relating to structural reliability assessment of fixed offshore platforms can be found in [5] and [6]. This paper represents a case study for the structural evaluation of an existing platform located in the Persian Gulf. The platform is now 47 years old and the objective of the study is to verify whether the platform can meet the structural requirements, as per API RP 2A and AISC. To check if it is fit for purpose for a life extension of 20 years beyond 2016. To provide a more exact and effective evaluation of offshore pile foundation systems under axial functional loads and lateral environmental loads, a finite element model software (SACS) is employed to consider the pile soil interaction. It should be noted that the structural model is based on the best estimates of the existing conditions of the platform.

In-Place Analysis of Jacket

In-place assessment is the structural analysis to assess the structural behavior of the jacket to specify its response service life. In-place analysis of jacket is carried out to control the global completeness of the platform against too early failure. Among all analysis of jackets, in-place is the most critical one. In a linear structural analysis with respect to ultimate limit state design (ULS), the specific capacity is normally taken as first yield or first component buckling [7]. If tubular members of a structure do not satisfy the ultimate strength provisions, resulting in yielding or buckling, the tubular member would not be considered fit for the purpose.

Analysis Software

SACS (Structural Analysis Computer Systems), a Design and Analysis software for offshore structures and vessels, is used for the modeling and analysis of the jacket. SACS is a finite element program for linear and nonlinear static and dynamic analysis of frame structures. Its ability to dynamically iterate designs allows users to perform advanced analysis, comply with offshore design criteria, and visualize complex results. SACS provides reliable beam member code checking and tubular joint code checking capacity; therefore, it is very suitable for topsides structures consisting of plate girders and tubular columns/ braces [8].

Structural Modeling

Platform Data

The jacket platform is eight-legged drilling jacket with grouted steel piles for the purpose of supporting 3415 tones maximum operation weight located in the Reshadat oilfield which is approximately located 108 km south west of Lavan Island in a water depth of around 58.8 m. The total height of the jacket is 87.4 m and the jacket footprint at sea floor is 30m×56m and leg spacing at working point is 41.15 m x 13.72 m. A perspective plot of the model is shown in Figure 1.

Three main components of the model are:
A) Substructure:

1. Jacket:
   a) Jacket legs
   b) Horizontal framings
   c) Elevation bracings and diagonals
2. Appurtenances

The following appurtenances are explicitly modelled for the hydrodynamic actions.
   a) Fifteen conductors 22’’ O.D (55.88cm)
   b) One riser 18’’ O.D (45.72cm)
   c) One riser 6” O.D (15.24cm)
   d) Two fire water pump caisson 18” O.D (45.72cm)
   e) One fire water pump caisson 26” O.D (66.04cm)
   f) Two J-tubes 8” O.D (20.32cm)

B) Foundation:
The foundation is modelled using uncoupled non-linear soil springs acting along the piles length. The load-displacement characteristics of these springs are defined by p-y, q-z and t-z curves based on geotechnical report. Based on pile makeup drawing the pile is modelled to a penetration of 41m below mud-line for all piles. Pile outer diameter is 762 mm. The scour readings by survey report ranged from 400mm to 900mm, so the final scour for modelling the platform was assumed equal to 1m on all pile locations.

C) Deck:
The topside has three deck levels and includes accommodations and different equipment. The model includes all the deck primary and secondary beams, truss chords, bracing and columns. Deck plates have been included as quadrilateral isotropic plate element for the in-plane stiffness of the deck.

Material

As per API RP 2SIM 2014 material specifications and properties of an existing structure are defined based on data from original design.

Table 1: Material properties [9]

Density		ρ=7850 kg/m3
Young’s modulus		E=2.1·1011 Pa
Poisson’s ratio		ν=0.3
yield strength (MPa)	$t \le 16$	235
	$16 < t \le 40$	225

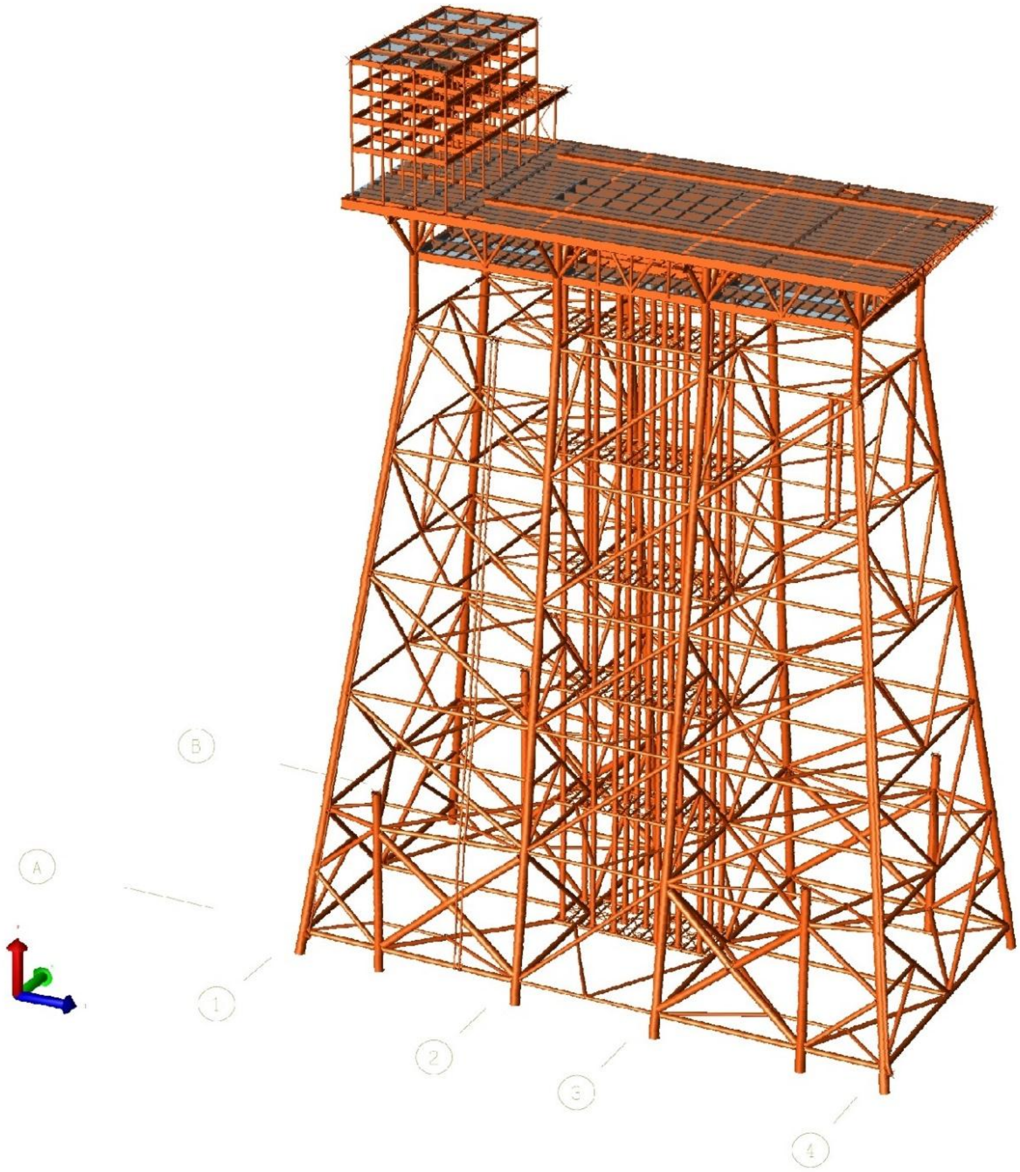

Figure 1: A perspective plot of the SACS model

Environmental Data

Water Depth

The platform is located in 193 feet (58.83 m) water depth. The design water levels and tidal range with 100 years return periods are summarized in Table 2.

Table 2: Water depth and surface fluctuations (m)

Description	One Year	100
Chart Datum Water Depth (To LAT)	58.8	58.8
Mean Sea Level (MSL)	1	1
MHHW	1.6	1.6
HAT	2	2
Storm Surge	0.2	0.3
Possible Subsidence*	+0.5	+0.5
Uncertainty Allowance	±0.5	±0.5
Maximum Water Depth	61.5	61.6
Minimum Water Depth	58.8	58.8

The maximum water depth considered in the analysis is 61.6m and the minimum water depth is 58.8m.

Max. Water Depth = Water Depth + HAT + Storm Surge + Subsidence

Wind

The wind loads are calculated based on the API RP 2A, using following-directional wind speeds for extreme storm conditions.

Table 3: 100-year return period wind speed (m/s)

Directio	NW	W	SW	S	SE	E	NE	N
wind	28.	27.	25.	25.	27.	27.	26.	27.

Shape coefficients for perpendicular wind approach angles with respect to each projected area should be considered as follows (API RP2A-WSD-2014):

Beams	1.5
Sides of buildings	1.5

Cylindrical sections	0.5
Overall projected area of platform	1.0

Wave and Current

Directional waves are used for the in-place analysis. Wave height with associated period for extreme storm conditions are as follows:

Table 4: 100-years wave heights and associated wave periods

Direction from TN	Maximum Wave Height, Hmax (m)	Significant Wave Height, Hs (m)	Wave Period, (sec)
NW	12.2	6.6	11
W	10.8	5.8	10.4
SW	8.8	4.7	9.5
S	10.2	5.5	10.2
SE	11.6	6.2	10.8
E	10.8	5.8	10.4
NE	8.8	4.8	9.6
N	9.7	5.2	10

The following currents are considered for the design of the platform.

Table 5: 100-years return period current profile (m/s)

Elevation	NW	W	SW	S	SE	E	NE	N
Surface	1.1	0.9	0.8	0.9	1.1	0.9	0.9	0.9
50% Water	0.8	0.6	0.5	0.6	0.8	0.6	0.6	0.6
5.0m above	0.5	0.4	0.3	0.4	0.5	0.4	0.3	0.4
0.5m above	0.4	0.3	0.3	0.3	0.4	0.3	0.3	0.3

Hydrodynamic Coefficients

Basic drag and inertia coefficients used to evaluate wave forces on cylindrical members are as follows:

Table 6: Hydrodynamic coefficients for calculating the storm wave loads on tubular members

Surface Conditions	Cm	Cd
Clean Steel	1.6	0.65
Marine Growth Fouled	1.2	1.1

For the In-place condition modelling, the wave kinematics factor should be taken as 0.9 as per API RP-2A. The current blockage factors for the 8 legged structures are as API RP-2A.

- End-on 0.70
- Diagonal 0.85
- Broadside 0.80

Marine Growth Profile

Marine growth may give rise to increased weight, increased hydrodynamic added mass and increased hydrodynamic actions, and may influence hydrodynamic instability. For typical design situations, global hydrodynamic action on a structure can be calculated using Morison's equation, with the values of the hydrodynamic coefficients for unshielded circular cylinders [10]. Table 7 presents the marine growth thickness measured by underwater survey. The specific weight of marine growth in air considered equal to 14 kN/m3.

Table 7: Marine growth thickness [3]

Top elevation (m)	Bottom elevation (m)	Thickness (mm)
0.00	0.3	5.0
0.3	14.9	5.0
14.9	15.5	5.0
15.5	27.1	5.5
27.1	27.7	4.0
27.7	39.3	4.0
39.3	39.9	6.0
39.9	51.5	6.0
51.5	52.1	6.0
52.1	63.7	6.0

Soil Condition

The analysis includes the effect of the non-linear soil stiffness through the soil-structure interaction software named SACS PSI. The soil model is subdivided into seven layers. The design soil parameters are presented in Table 8.

Table 8: Parameter values for R4 platform existing pile capacity

Layer number	Depth (m)	Soil type	δ (deg)	Cu top	Cu bot.	Sub. unit wt. (kN/m3)	Nq	f_{lim} (kPa)	q_{lim} (MPa)
1	0-16	clay	-	5	50	8	-	-	-
2	16-17.8	calcarenite	20	-	-	9	12	15	3
3	17.8-32	clay	-	55	85	8	-	-	-
4	32-49	calcarenite	23	-	-	9	15.8	15	5
5	49-60	clay	-	110	180	9	-	-	-
6	60-71.3	calcarenite	20	-	-	9	12	15	3
7	71.3-100.3	clay	-	150	220	9	-	-	-

Where:

δ = soil-pile friction angle
Cu = undrained shear strength
Nq = bearing capacity factor
f_{lim} = limit unit skin friction (kPa)
q_{lim} = limit unit end bearing pressure (MPa)

Methodology

The jacket components such as legs, primary and secondary braces and joints are designed to satisfy the strength and stability requirements specified in API RP2A-WSD and AISC-WSD. The check is performed through the use of the equations presented in these standards that can deliver the usage factor. If the usage factor is greater than 1.0 then the member is overloaded and does not meet the criteria for fitness for service. In-place analysis comprises three-dimensional static analysis with Pile Structure Interaction (PSI). As per API RP 2A the 100 years met-ocean storm data should be used for Design Level assessment [3].

Hydrodynamic Modelling

Jacket members in the splash zone are modelled with corrosion allowance (3mm during 20 years from now on [11]). The following consideration and parameters have been used in generating the water particle velocity and accelerations for wave loading in accordance with Morrison equation:

1. In-place analysis is carried out for minimum and maximum water depth cases.
2. Wave, current and wind loads are assumed to be collinear.
3. Stokes wave theory has been used for the wave load generation.
4. Seventy-two (72) wave positions of 5° intervals (full cycle) were stepped through the structure to identify the wave crest position causing the maximum base shear or the maximum overturning moment. The crest positions associated with the maximum base shear and the maximum overturning moment have been selected for the orthogonal and diagonal waves respectively.
5. A wave kinematic factor of 0.90, for extreme storm waves (API RP 2A-2014-5.3.1.2.4) has been considered.
6. Current blockage factors 0.80 (broadside), 0.7 (end-on) and 0.85 (diagonal) has been considered (API RP 2A-2014-5.3.1.2.5).
7. An apparent wave period, accounting for the Doppler effect of the current on the waves, has been determined (API RP 2A-2014-5.3.1.2.2).
8. A nonlinear stretching approach has been employed to stretch the current profile to the local wave surface.

Results and Discussion

Linear static analysis is performed for the eight legged jacket in 8 loading directions, 4 in orthogonal direction and 4 in diagonal directions. Post, a sub program of SACS VI, is used to calculate element stresses and compare them to allowable stresses. The API RP2A-WSD [12] and AISC-WSD [13] code are selected to check stresses in the elements. The check is performed through the use of the equations presented in these standards that can deliver the usage factor. If the usage factor is greater than 1.0 then the member is overloaded and does not meet the criteria for fitness for service. The basic allowable stresses are increased by 1/3 for the storm load cases.

Member Unity Check

A member check of a frame’s structural member is performed to assess whether the member is subjected to acceptable stress levels. Calculation results show that under 100-year storm load condition, a total number of 44 members have had stress utilization factors (UCs) greater than unity. 24 members are located in the splash zone of the platform. They are believed to suffer from serious corrosion. Five critical members are secondary beams in main deck that are affected by local loading of equipment. These members have poor section (channel and angle sections) which seems have not sufficient section for local loadings and need to be reinforced or stiffened. Figure 2 presents the graphical summary on the whereabouts of code-noncompliance members identified in the course of the assessment analysis in existing condition.

Joint Unity Check

A punching shear check is carried out on the brace member at a joint to assess the shear through the chord. As for the other checks, these assessments are made through the use of a punching shear interaction equation that delivers a usage factor. A total number of 52 joints have had utilization factors (UCs) greater than unity. A lot of these joints are located in splash zone.

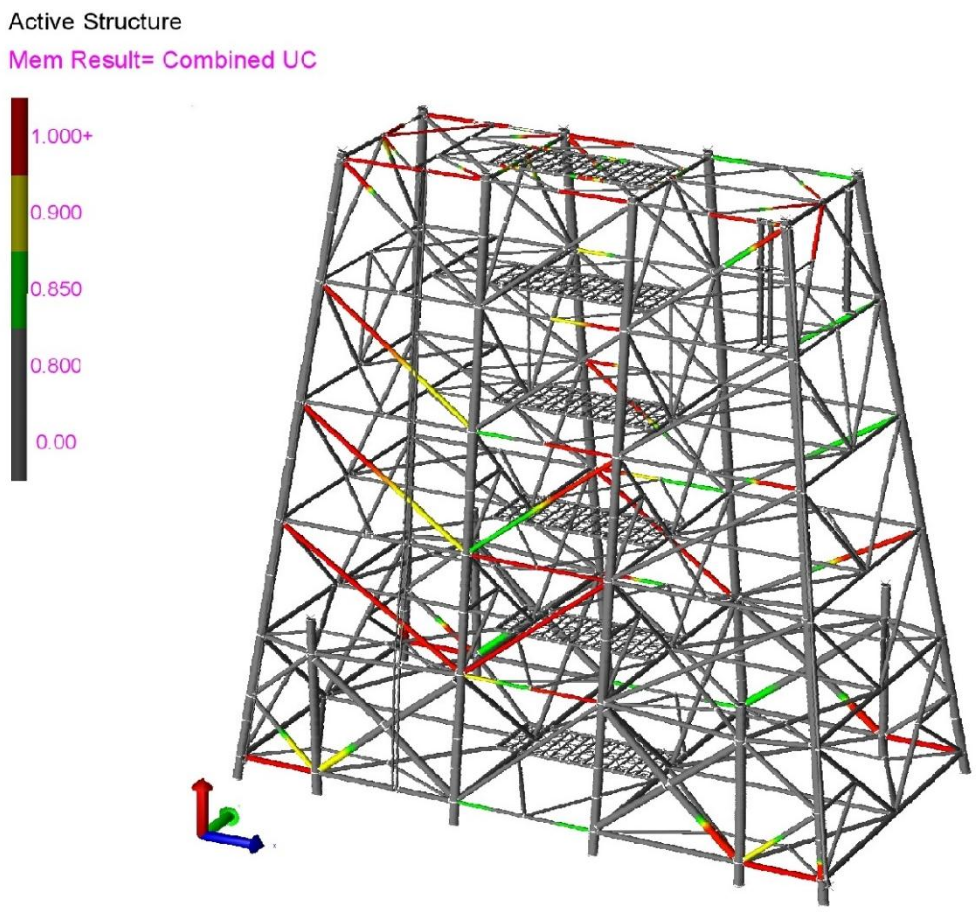

Figure 2: 3D Model for Jacket Members with UC >1.0

Piles Structural Strength (below Mud-line)

Piles do not meet the In-place structural code provisions as all piles have stress utilization ratios above unity. It should also be noted that no soil aging effect (the increase of soil shear strength with time) has been considered in obtaining the pile load bearing capacity.

Table 9: Pile below mud-line structural UC

Grid Location	Pile Joint No.	Pile below Mud-line Maximum Structural UC
A1	101L	1.11
A2	107L	1.10
A3	113L	1.10
A4	119L	1.16
B1	181L	1.22
B2	187L	1.18
B3	193L	1.18
B4	199L	1.21
Skirt Piles	J111L	1.25
	J113L	1.24
	J171L	1.32
	J173L	1. 36

Pile-head Displacement

The displacements of the platform in existing condition are not in allowable range. It seems this is because of unacceptable performance of the piles. The allowable lateral displacement at the pile-head is taken to be 10% of the pile diameter (7.6 cm =0.1D) as stated by ASTM STP-835 (1983) [14]. A similar ratio has been considered for the allowable axial displacement at the pile-head.

Table 10: Maximum pile-head displacements

Grid Location	Pile Joint No.	Displacements	Pile-head Displacement (cm)
A1	101L	Axial Dis.	38.87
		Lateral Dis.	14.95
A2	107L	Axial Dis.	38.25
		Lateral Dis.	13.68
A3	113L	Axial Dis.	35.55
		Lateral Dis.	13.94
A4	119L	Axial Dis.	32.17
		Lateral Dis.	14.91
B1	181L	Axial Dis.	41.95
		Lateral Dis.	12.22
B2	187L	Axial Dis.	44.9
		Lateral Dis.	12.55
B3	193L	Axial Dis.	45.88
		Lateral Dis.	12.55
B4	199L	Axial Dis.	46.55
		Lateral Dis.	11.95

Skirt Piles	J111	Axial Dis.	35.87
		Lateral Dis.	13.36
	J113	Axial Dis.	32.00
		Lateral Dis.	14.16
	J171	Axial Dis.	41.51
		Lateral Dis.	13.67
	J173	Axial Dis.	44.28
		Lateral Dis.	14.45

Topside Displacement

Topside displacements for different nodes at the main deck elevation and helideck elevation were obtained from the analysis. The H/200 limit for the deck deflection under the 100 year is based on AISC-2005 Commentary on Specification for Structural Steel Building-L4 Drift [13].

Table 11: maximum Topside displacements for different scenarios

Location	Maximum Overall Horizontal Displacement	Pile-head Lateral Displacement	Maximum Drift Relative to the Pile-head	Height Above Mud-line (m)	Allowable Drift
Main Deck	103.22	14.95	88.27	74.37	37.18
Helideck	140.95		126	87.62	43.81

Summary and conclusions

The structural integrity of the existing platform has been carried out as per In-place assessment provisions of API-RP-2A. The structural assessment was performed based on the best estimates of the existing conditions of the structure data on future corrosion allowance (3mm during 20 years from now on). The summary of the analysis result for the existing condition of platform are as below:

- Results from the analysis show that the existing platform does not assure the code provisions. This is mostly due to the poor performance of the piles, in addition to some members and tubular joints which experience utilizations factors greater than unity.
- Piles does not meet the code provisions as all piles have stress utilization ratios above unity. It should also be noted that no soil aging effect (the increase of soil shear strength with time) has been considered in obtaining the pile load bearing capacity.
- Calculation results show that under 100-year storm load condition, a total number of 44 members have had stress utilization factors (UCs) greater than unity. Of these, 24 members are located in the splash zone of the platform. They are believed to suffer from serious corrosion. Five critical members are secondary beams in main deck.
- In addition, a total number of 52 joints have had utilization factors (UCs) greater than unity. A lot of these joints are located in horizontal plans.
- The displacement of the platform in existing condition are not in allowable range. It seems this is because of unacceptable performance of the piles.
- Results of the In-place analysis of the drilling platform indicate that the jacket structure does not assure the code provisions. Therefore, it is mandatory to verify that the structures can fulfill the requirements of the nonlinear analysis. So, a nonlinear quasi-static (push-over) analysis is suggested for the further research about this specific platform.

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