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Dynamic Analyses of Jacket Type Offshore Platforms against Progressive Collapse Considering Pile-Soil-Structure Interaction

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ABSTRACT

This research aims to present a practical framework to study the structural response of a jacket type offshore platforms subjected to a sudden member removal considering the pile-soil-structure interaction. To this end, a series of nonlinear dynamic analyses are performed, and the progressive collapse resistance of the generic structure is determined. Consequently, the members prone to failure are detected. As a case study, the application of the proposed framework to control the capability of these type of structures for the prevention of progressive collapse occurrence are investigated. In the model structure, some legs and vertical braces in different locations are eliminated, and the effect of each damage case on the performance of the structure is investigated while the environmental wind and wave loads are imposed to the platform. The simulation results demonstrated that although the jacket structure can sustain the loss of primary members safely, it is susceptible to failure progression while a leg and the connected brace are eliminated simultaneously. The safety margin, in this case, is about 20% only. In addition, it was revealed that in the case in which a leg and the connected brace are eliminated, progressive collapse resistance is about a third in comparison with the case of a leg damaged only.

1. Introduction

Progressive collapse is described as the extending of local damage to intact part of the structure, resulting in failure of the entire structure eventually[1]. The potential hazards and abnormal loads, including ship collision, explosions, fire, dropped objects, and extreme environmental events may lead to a progressive collapse in offshore structures. General structures are not generally designed for abnormal loads, which can lead to failure. Most of the current codes and standards have general recommendations for reducing the possibility of progressive collapse in structures that are overloaded beyond the design loads. American Society of Civil Engineers (ASCE) 7-05[1] pointed out the topic of progressive collapse in some detail. GSA[2] and UFC[3], which are US government documents, presented the noticeable guidelines for progressive collapse analysis and design of public buildings.

Oil and gas are known as vital sources of energy which are partly produced in the marine environment with significant threats, e.g. explosion, fire, drop objects, ship impact, and other hazards. The economic and environmental effects of the overall collapse of offshore platforms should be considered, so the structural system should be designed in such manner which accidental damage does not escalate into the global failure of the platform. In the confrontation of the platform to the accidental loads, two general structural behaviors can be considered; the structural system resists locally against accidental action without damage or the accidental loads lead to partial or overall damage of structural components. In the latter case, for avoiding the occurrence of progressive collapse, the structure should be designed so that the intact part of the structure provides an alternate path load in which the loss of structural member compensated by the surrounding elements.

During the past three decades, the progressive collapse of offshore platforms has been investigated by some researchers. Amdahl et al.[4] presented a new approach for non-linear analysis of truss work platforms. They used a practical design example on progressive collapse analysis of jacket type offshore platforms to verify the accuracy and efficiency of their procedure. They also applied their techniques for progressive collapse analysis of plate girder and truss type deck structures on offshore platforms. Soreide et al.[5] addressed various methods to analyze the behavior of truss and frame steel structures of fairly slender tubular members and joints under static and cyclic loads. Moan et al.[6] developed and applied a method for analyzing truss and frame steel structures considering elastic and plastic tubular joint and material and geometrical beam-column behavior using USFOS computer software[7]. Siguardsson et al.[8] studied on the randomness of the ultimate capacity of different types of jackets in the North Sea. They used USFOS program for the progressive collapse analysis. Amdahl[9] presented a design curve for bow impacts against jacket legs using non-linear finite element analysis and concluded that a leg should not be subjected to significant denting.

Besides, in the field of common buildings, during the past decades, there have been numerous studies that investigated different aspects of progressive collapse and assessing the capability of structures to withstand collapse. Kim et al.[10] observed that the linear static analyses provide lower structural responses than nonlinear dynamic analyses and the results varied more significantly depending on the variables such as applied load, the location of column removal, or the number of building stories. However, the linear static analysis procedure provides a more conservative decision for the progressive collapse potential of model structures. They observed that the potential for the progressive collapse was the highest when a corner column was suddenly removed in the steel moment-resisting frames. Fu[11] declared that under the same general conditions, a column removal at a higher level would induce larger vertical displacement than a column removal at ground level. Powell[12] utilized various analysis procedures and found that the impact factor of two regulated in the linear static analysis can display very conservative result. Ruth et al.[13] found that a factor of 1.5 better represents the dynamic effect, especially for steel moment frames. Khandelwal et al.[14] concluded that an eccentrically braced frame is less vulnerable to progressive collapse than a special concentrically braced frame. Strarossek[15] developed a typology and classification for the progressive collapse of structures. Kim et al.[16] depicted that the dynamic amplification can be larger than two, which is recommended by the GSA and UFC guidelines. Kim et al.[17] suggested that the performance of buildings using cover plate connections turned out to be most effective in resisting progressive collapse, especially in structures located in moderate-seismic regions. Kim[18] deduced that among different types of braced frames, the inverted-V type shows superior ductile behavior during the progressive collapse. Tsai and Lin[19] evaluated the progressive collapse resistance of reinforced concrete (RC) buildings and concluded that nonlinear static analyses provide a more conservative estimate for the collapse resistance than nonlinear dynamic analyses. Dawoon[20] investigated the effect of the catenary action on the progressive collapse potential of steel moment frame structures. According to the nonlinear static push-down analysis results, the potential of the structures increases as the number of stories and bays increase. Grierson et al.[21] presented a method for conducting a linear static progressive collapse analysis. They modeled the reduced stiffness during the progressive collapse using an equivalent spring method. Naji[22] presented a simplified analysis procedure for the progressive collapse analysis of steel structures using the load displacement and capacity curve of a fixed end steel beam. Asgarian[23] concluded that the frame with two braced bays had more robustness for mitigating of progressive collapse, at least to the rate of 17.21% comparing to the frame with three braced bays. Jiang[24] investigated the effect of various bracing systems on the fire induced progressive collapse resistance of steel-framed structures using OpenSees. They found that the application of vertical bracing systems alone on the steel frames to resist progressive collapse is unsafe and recommended a combined vertical and hat bracing system in practical design. Jiang[25] studied the possible progressive collapse mechanisms of planar steel frames when one column fails under elevated temperature through extensive case studies. Fu[26] used AP Method to study the dynamic performance of two-dimensional (2D) bolted-angle steel joints under a sudden column removal scenario. Gerasimidis[28] presented a new partial distributed damage method (PDDM) for steel moment frames by utilizing finite element computational tools that are able to capture the loss of stability phenomena. It is shown that the introduction of partial damage in the system can significantly modify the collapse mechanisms and affect the response of the structure.

Studying existing scientific resources, the dynamic behavior of jacket type offshore structures, when structural members are lost, has not been investigated completely. Therefore, this paper focuses on applying an advisable progressive collapse analysis procedure to study this issue using a finite element framework that considering geometric and material nonlinearities. As a case study, this approach is applied on a 3D model of realistic jacket type platform, and capacity of such structure for preventing progressive collapse and its failure mode is determined using vertical incremental dynamic analyses. Toward this objective, a newly designed functional platform at Persian Gulf region has been investigated. In this structure, some damage cases are defined by introducing damage through the loss of structural members, i.e. legs and corresponding braces in different positions and the effect of each damage scenario on the response of the structure has been investigated. Additionally, by applying this procedure, the critical places of the removals are determined.

2. Progressive collapse analysis method

In this study, a new framework for progressive collapse analysis is applied for jacket platforms. In this approach, for each alternate path method (APM) case, primarily, by executing a nonlinear dynamic analysis the response of the structure is investigated, and secondly, the maximum value for all structural element effort is checked by the nominal capacity. For each member, the demand over-capacity ratio (DCR) is calculated from Eq. (1).

$$\mathrm{DCR} = \frac{Q_{UD}}{Q_{CE}} \qquad (1)$$

Where Q_{UD} is the acting force (demand) which are determined in element (moment, axial force and shear, etc.); and Q_{CE} is the expected ultimate, unfactored capacity of the element.

If the maximum value exceeds the capacity of the element, it implies that the local damage has spread to other elements and structure is vulnerable versus progressive collapse. If not, it means that the structure is capable of attaining the alternate load path after element removal, but exclusively for the inflicted loads. In the next step, in order to estimate the structural capacity, dynamic overload factor is determined by performing the vertical incremental dynamic analysis (IDA). For each predefined scenario, the vertical load factor is increased until the first mode of the structural failure is reached. The overload capacity of the structure is expressed in terms of overload factor, as demonstrated in Eq. (2).

$$\text{Overload Factor} = \frac{\text{Failure Load}}{\text{Nominal Steady Load}} \qquad (2)$$

For progressive collapse analysis, some damage cases are defined by introducing damage through the loss of a primary structural element in the splash zone which are highly affected by corrosion and ship impact. Then, the effect of defined damage case on the performance of the structure is investigated. In this procedure, the vertical loads are linearly increased during 5 seconds to reach the final values; after that, they are kept constant for 2 seconds to avoid dynamic effects. The environmental loads are applied as time history load to jacket from the beginning of the analysis. In 7th seconds of the analysis, the related elements to the APM case are eliminated from the finite element model, and afterward, the following response of the jacket structure is investigated. The analysis is performed with 5% proportional mass and stiffness damping.

By performing nonlinear dynamic analysis in each scenario and consequently, by comparing the peak values of responses with those in the steady state before removing the elements, the element demand over-capacity ratio (DCR) is calculated for all structural elements including legs, horizontal and vertical braces. The element with maximum demand-over capacity ratio would be the most probable element for the progression of failure and would be the first try for dynamic overload factor determination in the incremental dynamic analysis. This method is used to determine the so-called parameters and the probable critical removal location in the structure.

2.1. Structural modelling

OpenSees, [29] a finite element program has been used for modelling and analysis of the structure. All structural tubular members are modelled through the employment of beam-column element in OpenSees. The cross-section of members is defined by the fiber element. Steel02 material, in combination with fatigue material from the program library, is assigned to members. The maximum ductility ratio of 15 is allowed for the extension of the material non-linearity in the plastic region. The strain hardening of 2% is defined for steel behavior beyond the yielding point. For considering the effects of large deflection, corotational transformation object is used for transforming element stiffness and force coordinate in each step of the analysis. An initial mid-span imperfection of $\mathrm{L}/1000$ for all elements is considered as depicted in **Figure 1**. Also, a fiber cross-section element is considered for plasticization of the element over the length and cross-section of members.

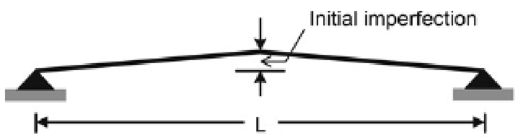

Figure 1. Initial imperfection in compression members

2.2. Buckling Verification

The estimation of structural response due to failure depended on the proper prediction of compression elements' behavior. The starting point for any attempt to assess the performance of the structure is improving the computational model to simulate the behavior of bracing members in different conditions such as yielding, buckling or failure in various events.

Buckling and post-buckling behavior of compression members are verified by comparison of numerical results with the test data obtained by Sherman[30]. The test result for a brace with cross-section of$Pipe\ 11.44 \times 0.23cm$ was used for model verification. **Figure 2** shows a comparison between finite element model results and the test results data. As it is portrayed, the model accurately represents the buckling and post-buckling resistance of the tested specimen.

2.3. Wave Force Calculation

According to API recommended practice[31], after accidental loads imposed on the platform, it should retain sufficient residual strength to withstand the one-year environmental loads in addition to the regular operating load. In this paper, the environmental loads that are implemented in all analysis are wind and wave loading. For the wave force calculation, a FORTRAN code has been developed. This program is capable of solving the wave equations using Stokes fifth order wave theory. Water particle velocity and acceleration at different depth are calculated at each time step[32], and wave force is calculated using the Morison equation. This force is applied to the jacket structure joints in OpenSees model as a time-dependent loading. For considering the effect of non-modelled members such as boat landing, conductors, risers and anodes, the wave force is increased 10% in all states. The drag and inertia coefficient are considered 1.05 and 1.20, respectively. The wave height and period at position and direction of platform extracted from meteorological data and were assumed to be 5.1m and 7.6 seconds respectively for operating one-year storm. Calculation results have been compared with the more accurate model developed using SACS computer software[33]. The results are in good agreement, and the differences for both conditions are acceptable for engineering purposes, as seen in **Figure 3**.

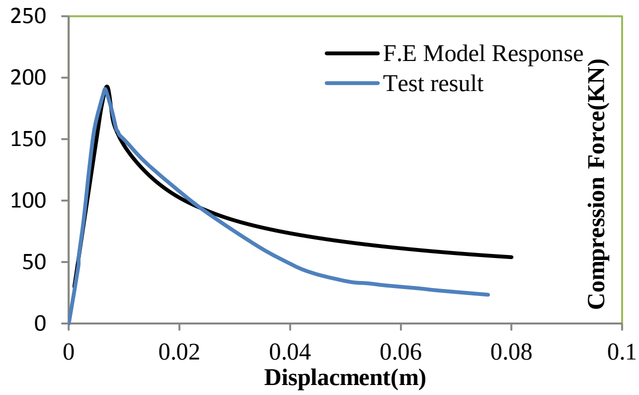

Figure 2. Verification of Brace Behavior

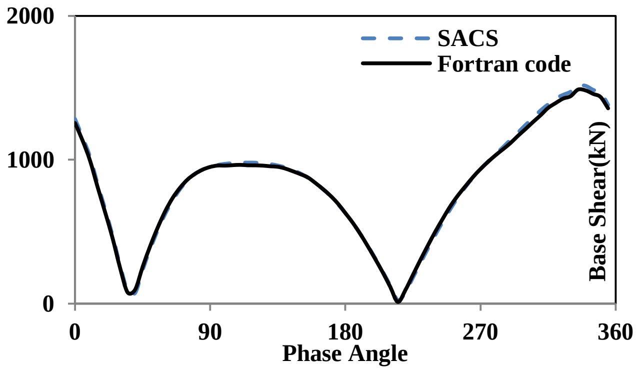

Figure 3. Verification of Wave Force Generation

2.4. Soil-Pile-Structure Interaction

The accurate considering of pile foundation behavior affects significantly on jacket platforms performance, especially in failure cases. Dynamic interaction between piles and surrounding soil is a complicated matter involving consideration of soil profile specifications, superstructure response and soil-pile-structure interaction. In the past studies, various numerical and experimental methods have been used for the response of pile foundations. There are two main numerical approaches for predicting the response of a pile foundation. The first method is a continuum based method, and the second one is a discrete element method[34,35]. Finite difference, boundary element and finite element methods are categorized as a continuum based method and Beam on Nonlinear Winkler Foundations (BNWF)[36–38], equivalent base spring model, and equivalent cantilever model[39] are categorized as a discrete element method. Discrete element method is a simplified model, and it is an efficient approach and considerably less complex than continuous based methods.

In the method that is used here, the pile is modelled as a beam element, and the peripheral soil is considered using continuous springs and dashpots. Pile nonlinearity has been considered using an appropriate material model in the analysis. For taking into account the soil nonlinearity, nonlinear springs with stiffness and resistance parameters and dashpots for energy dissipation in dynamic load cases which are placed in parallel condition are used.

In this study, nonlinear p-y element for the consideration of lateral resistance, t-z elements for the consideration of pile skin friction and q-z elements for consideration of end bearing resistance are implemented in the nonlinear structural program OpenSees for progressive collapse analyses of the platform. The configuration of p-y, t-z and q-z are shown in a sketch in **Figure 4**.

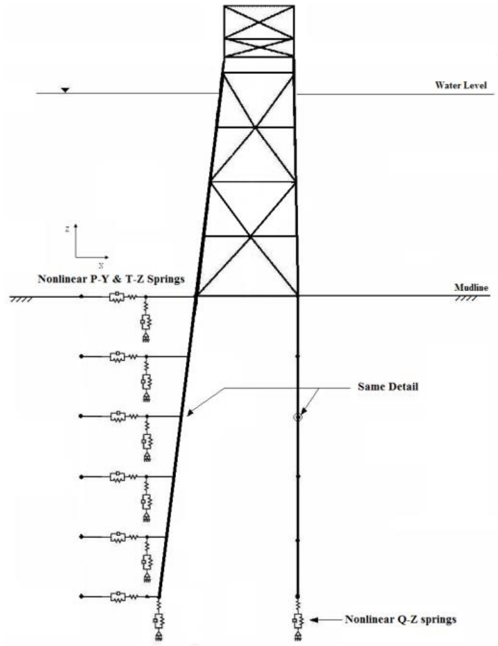

Figure 4. Pile-Soil System Modeling in OpenSees

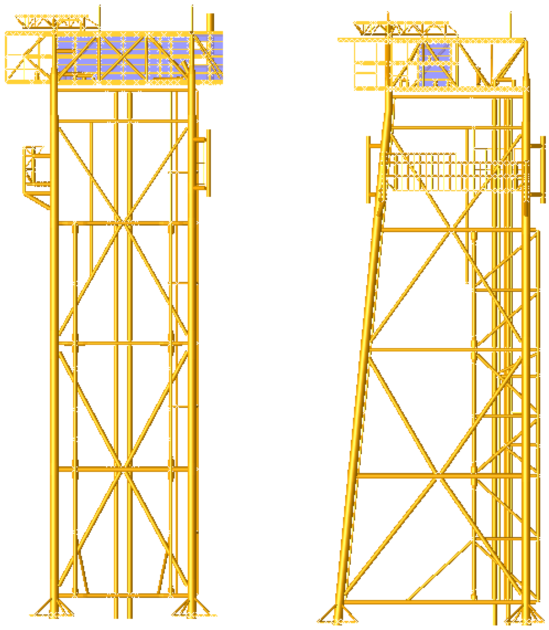

Figure 5. Side View of Investigated Platform

3. Case Study

In this paper, a platform with three-story decks and jacket with five horizontal levels and a total height of 96.5m which located in the water depth of 74.85m in the Persian Gulf region has been studied. Jacket Legs, diagonal and horizontal bracing, piles and the main structures of the deck have been modelled numerically using nonlinear Beam-Column element in OpenSees software. The structure is analyzed by considering the non-linear interaction between soil and piles. In the mentioned structure, four grouted piles with a penetration depth of 95.0m and outside diameter and wall thickness of 1.32m and 4cm respectively has been considered. Pile elements due to their connections to the soil are divided into 60 segments that at each node, two p-y elements in the horizontal direction and one t-z element in vertical direction have been considered. Also, One Q-z element for end bearing of the pile is considered at the end of piles.

Deck weight includes self-weight of topside elements, topside functional loads (includes mechanical, piping, electrical, instrument, HVAC loads, etc.) and the live load is equal to the 3260 tons that are imposed to the primary nodes of the deck as a concentrated force. Jacket weight, including piles, is considered as 1900 tones. The service loads which are considered to be imposed on the damaged structure have been comprised of 100% of dead load, 50% of live load and one-year return period wave loads. Schematic view of the investigated offshore platform beside the numbering of elements is depicted in **Figure 5** and **Figure 6**.

3.1. Primarily Analyses

The summary of APM analysis cases with the structural elements that were eliminated in each scenario is presented in Table 1.

Table 1. APM Analysis Cases (Scenarios)

Scenario APM Case	Element Removed
1	Vertical Brace no. 23 (In X-Direction)
2	Vertical Brace no. 23,24 (In X-Direction)
3	Vertical Brace no. 1 (In Y-Direction)
4	Vertical Brace no. 1,2 (In Y-Direction)
5	Leg no. 21
6	Leg no. 21 & Vertical Brace no. 23
7	Leg no. 21 & Vertical Brace no. 1

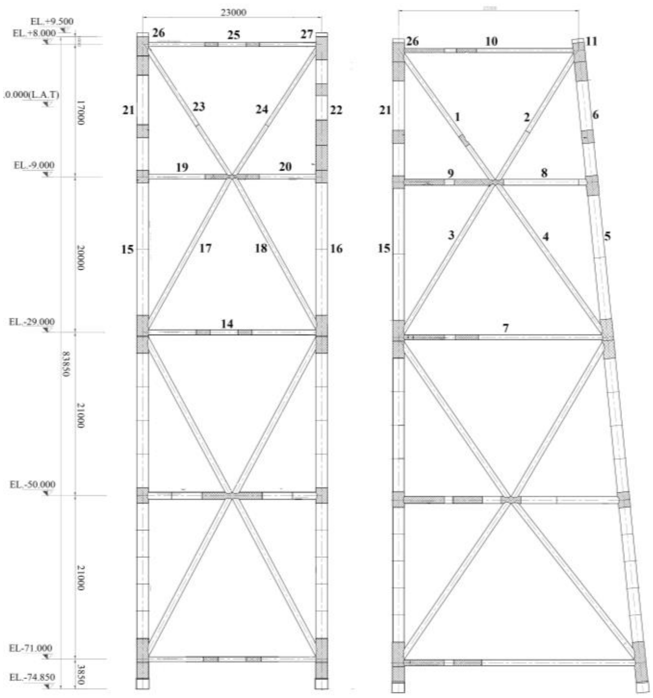

Figure 6. Coding System of Legs and Braces

In Figure 7 to Figure 15, responses of the 3D model of jacket platform for elements removal have been illustrated. For the first four scenarios related to the removal of the vertical brace in different locations, in the worst case, the axial force in leg no. 21 in the first scenario spiked from 5131.9kN to the extreme value of 5640.0kN and after that reduced to the steady value of 5557.0kN. The axial force in leg no. 6 in the fourth scenario spiked from 6165.0kN to the extreme value of 7633.3kN and then decreased to the steady value of 7468.0kN. In this scenario, after removal of the two braces, the side legs sustained a load of these members without being overloaded. In these scenarios, the maximum DCR occurred in brace no. 25 with a value of 0.22 and the highest relative increment in DCR value occurred at Leg No. 6 in the fourth scenario (approximately 110%).

For the last three scenarios related to the removal of legs, the condition is more critical. For example, the axial force in brace no. 23 in the fifth scenario spiked from 520kN to the extreme value of 4308.9kN and then reduced to the steady value of 3568.0kN. By assuming an effective length factor, K = 0.8 the axial capacity of this brace is 6350.0kN, which means that the brace is not overloaded. In this case, after removal of the leg, the load demand in braces in the vicinity of lost member increased significantly. For example, the highest increase in DCR in vertical and horizontal braces occurred in element no. 1 and no.10 with a value of 2200% and 627% respectively.

In the sixth scenario, the axial force in brace no. 2 spiked from 1275.3kN to the extreme value of 7474.0kN before decrease to the steady value of 5488.0kN. By assuming an effective length factor, K = 0.8 the axial capacity of this brace is 7527.0kN, which means that the brace is not overloaded. The highest relative increment in the DCR, in this case, occurred in element no. 1 with a value of 4900% increment and reached to the 1.51. It should be noted that the DCR greater than 1.0 does not necessarily mean entering to the material nonlinearity region; this is because in DCR calculation the allowable axial capacity derived based on the buckling capacity (Stability criteria) which is usually less than member resistance capacity (strength criteria). So the value of DCR is not an appropriate criterion for judgment about the behavior of the material. Thus the amount of strain in critical sections was calculated in the numerical model and compared to the yield strain. Results show that all the members have elastic behavior in this case, although DCR value is greater than 1.0 in some members.

In the seventh scenario, the axial force in brace no. 18 spiked from 563.7kN to an extreme value of 6371.9kN and then reduced to the steady value of 4443.3kN.By assuming an effective length factor, K = 0.8 the axial capacity of this brace is 7527.0kN, which is substantially less than the extreme value computed in this brace. The highest increment in the DCR, in this case, occurred in element no. 23 with a value of 1400% increment. In the last three scenarios, the side leg of the removed member (leg no. 21) has a small contribution in the absorption of the loss of a member and primary load path will be vertical braces in these kinds of damage cases. For example, the maximum DCR in these scenarios in the leg no.22 and 6 are 0.23 and 0.24 respectively that increased 289% approximately.

The analysis results illustrated that the structural system is capable of absorbing the lack of members correspond to scenarios presented in Table 1 successfully. In these cases, a broad distribution of forces was observed to take place after the loss of member. One-year environmental loads lead to small fluctuation in time history response of axial loads in all cases.

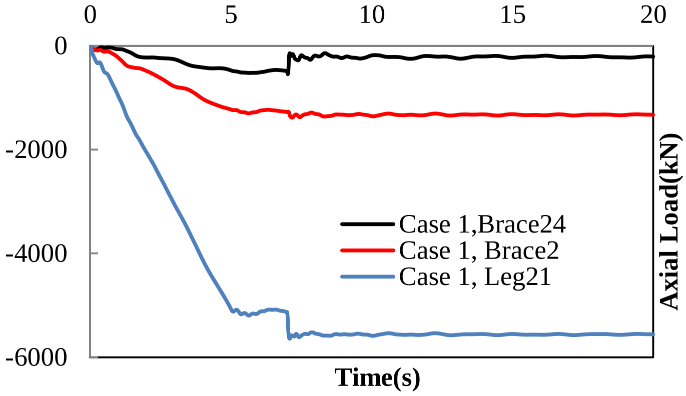

Figure 7. Time History Response of Axial Load in Sc. 1

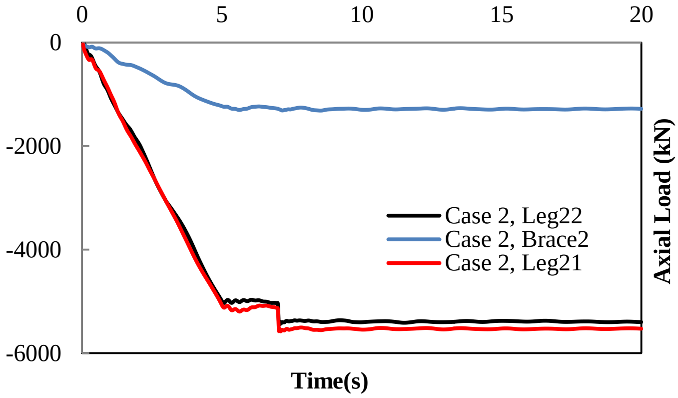

Figure 8. Time History Response of Axial Load in Sc. 2

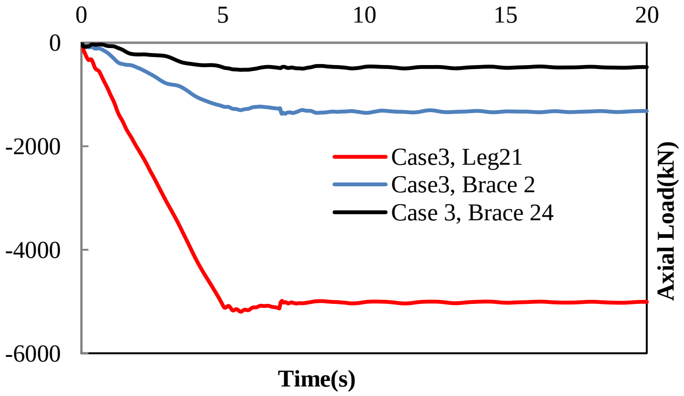

Figure 9. Time History Response of Axial Load in Sc.3

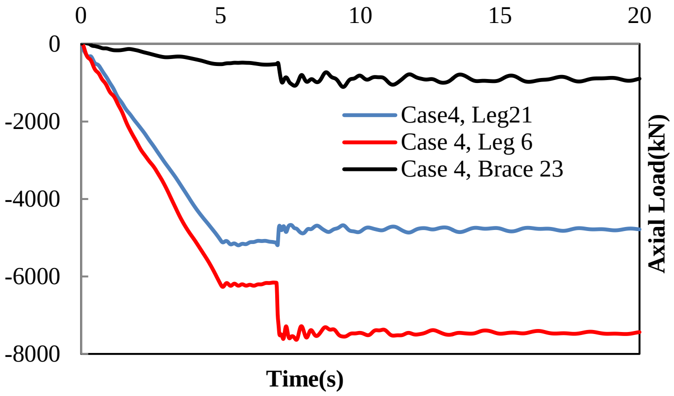

Figure 10. Time History Response of Axial Load in Sc.4

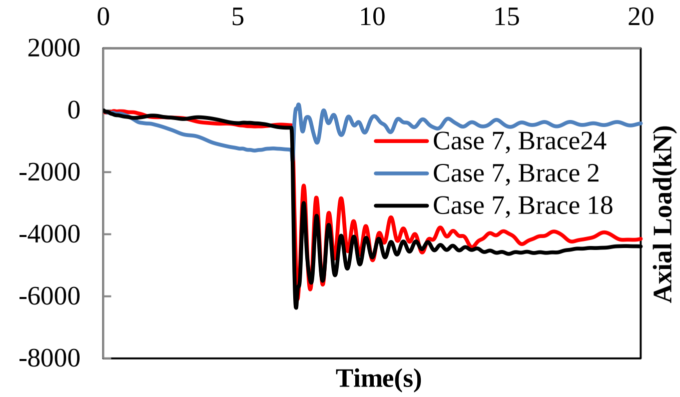

Figure 14. Time History Response of Axial Load in Sc. 7

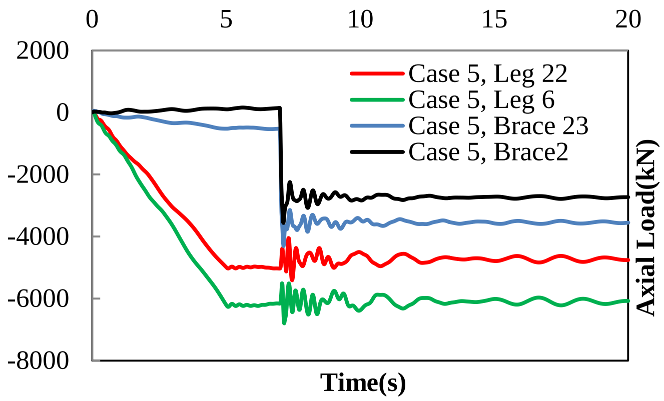

Figure 11. Time History Response of Axial Load in Sc. 5

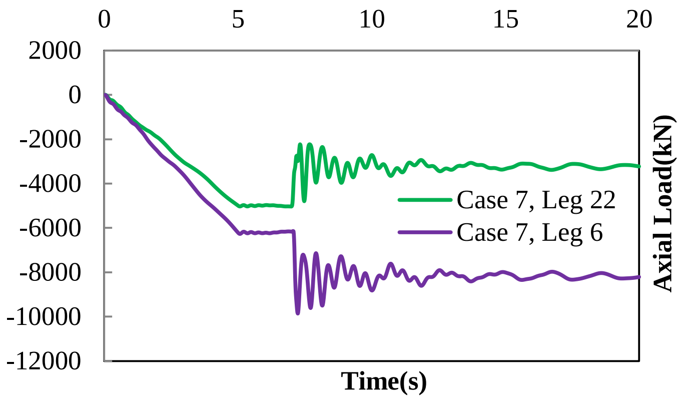

Figure 15. Time History Response of Axial Load in Sc. 7

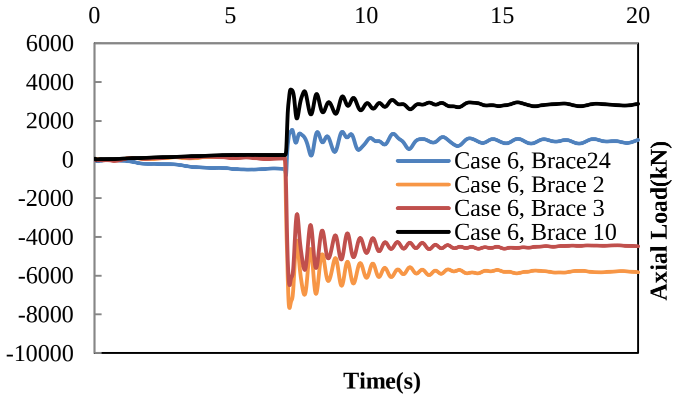

Figure 12. Time History Response of Axial Load in Sc. 6

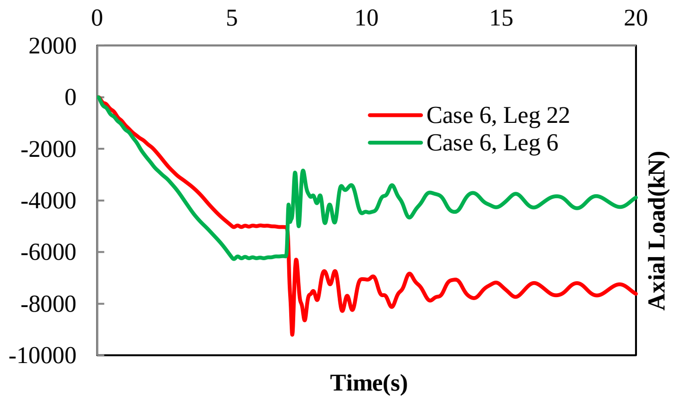

Figure 13. Time History Response of Axial Load in Sc. 6

3.2. Overload Capacity and Failure Mode Detection

As described in the previous section, the alternate path method was used for analyzing the jacket structure, which has suffered from the loss of elements and the structure condition after damage case was determined. Nevertheless, this procedure cannot be used for estimating the remaining capacity of a damaged structure in the cases where the structural system absorbs the loss of structural elements and determination of the future collapse modes. So for such cases, gravity and environmental loads are increased by multiplying rising factor until the structure reaches to its ultimate capacity. The load factor related to failure condition is defined as the failure overload factor.

Table 2. Overload Factors in Failure

Scenario	Critical Member	Overload Factor
1	Pile-Soil System	4.85
2	Pile-Soil System	4.85
3	Pile-Soil System	4.85
4	Pile-Soil System	4.85
5	Brace no.1	3.2
6	Brace no.2	1.2
7	Brace no.24	1.5

In primary analyses, once the structural response illustrated that structural collapse has not occurred, the incremental dynamic analysis (IDA) was performed for each damage scenario. This analysis is similar to the primarily nonlinear dynamic analysis but with one significant difference, i.e. after the loss of the elements, vertical and environmental loads are increased gradually till the first failure mode is detected. For this, several analyses may be required in order to a load factor related to the collapse mode is obtained. This analysis method accounts for the dynamic effects, which may be vital for all failure cases and is similar to the incremental dynamic analysis used in earthquake engineering[40]. In **Figure 16** to **Figure 18**, vertical incremental dynamic analysis curves for investigated cases are shown. In **Table 2**, the overload factor and failure modes that are detected in the platform is declared. As it is illustrated in **Table 2**, in scenario 1 to 4, in the mentioned factor of loads, the axial forces of piles below mudline is more than the allowable axial capacity of the pile-soil system, and the structure cannot withstand applied loads and the global failure occur before any local failure in jacket structural members. Among overload factors, for the fifth scenario related to one leg removal, the brace no. 1 had the lowest overload factor which means that this kind of removal in upper elevation is important and has a critical influence on progressive collapse. Same as a previously discussed scenario, in the 6th scenario, the brace no.2 is the most important and has a critical influence on progressive collapse occurrence. Also, in the 7th scenario, the brace no. 24 had the lowest overload factor. In this case, after removal of a leg and vertical side braces in one row, the load path changed and the vertical brace in perpendicular row transfer the load.

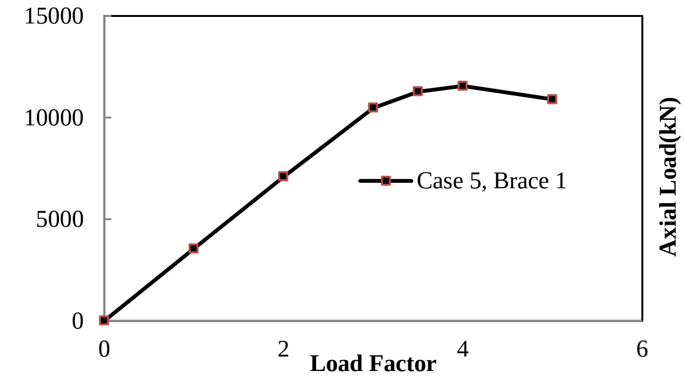

Figure 16. Vertical IDA curves- Sc. 5

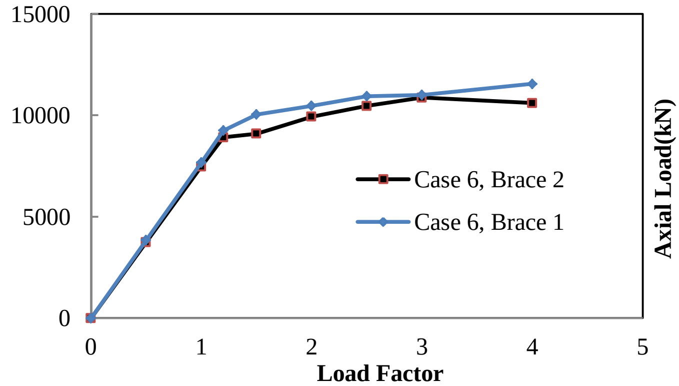

Figure 17. Vertical IDA curves- Sc. 6

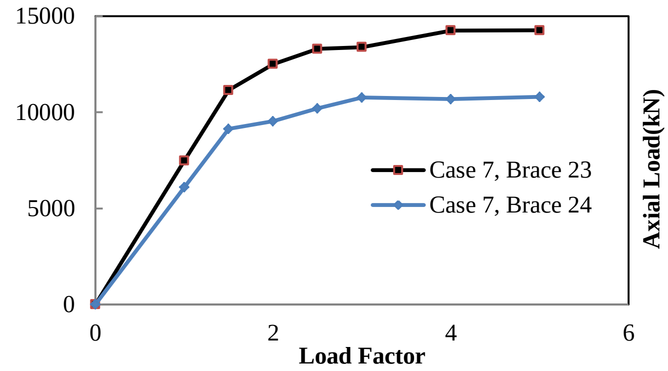

Figure 18. Vertical IDA curves- Sc. 7

5. Conclusion

In this research, a new method for assessment of progressive collapse in jacket type platforms was applied by which capability of these type of structures for the prevention of progressive collapse occurrence were investigated while some structural members damaged. After that, the failure modes were determined using vertical IDA. This approach was applied for progressive collapse analysis of specified jacket platform with different location of removal of elements, in order to determine and quantify the influence of element loss location on the jacket response. The simulation results demonstrated that the jacket structural system, in all predefined considered damage cases, can successfully absorb the lack of structural elements under the defined loading and the progressive collapse is not predicted for this structure. This matter is because the structural system of jacket type platforms is sufficiently redundant, and the intact portion of the structure sustain loads well in a damage case when the load in structural members exceeds the design values. Moreover, this platform has been designed to sustain 100-years extreme environmental loads and pre-installation condition together with using the appropriate safety margin. So the jacket is still able to successfully carry all the gravity and environmental loads in damage scenarios in operating condition. In such structures, the portion of the jacket influenced by leg removal, derive their stability from vertical & horizontal braces and side legs, and as a result, the collapse does not occur. Transmission of load between the damaged leg and intact members takes place through horizontal braces that are connected to legs in certain elevation. Though these members are under significant tension force, the members can fruitfully transmit the loads.

Comparing the failure modes and the corresponding overload factors, in the scenarios in which one leg and side brace was removed, the structure has the lowest overload factor equal to 1.2. It means that in the 6th scenario, a 20% increase in normal loads leads to the beginning of the progressive collapse. So practical actions should be applied to prevent events that cause the failure of leg and brace simultaneously. Also, in damage scenarios in which, one or two number of vertical braces were removed, the structure has the highest overload factor equal to 4.85, and the pile-soil system failed. It means that the axial forces of piles below mudline are more than the axial capacity of the pile-soil system and the pile was punched through the soil, and the global failure occurs before any local failure in jacket structural members.

Finally, for the investigated jacket type platform, comparing the overload factors for the last three discussed scenarios, it can be concluded that the failure of one brace while one of the jacket legs have been damaged totally, leads to 62.5% decrease in the strength of the structure for mitigating the potential of progressive collapse.

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