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# Inelastic Nonlinear Pushover Analysis of Fixed Jacket-Type Offshore Platform with Different Bracing Systems Considering Soil-Structure Interaction

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**Abstract:** In this study, inelastic nonlinear pushover analysis is performed on a 3-D model of a jacket-type offshore platform for the North Sea conditions. The structure is modelled, analyzed and designed using finite element software SACS (structural analysis computer system). The behavior of jackets with different bracing systems under pushover analysis is examined. Further, by varying the leg batter values of the platform, weight optimization is carried-out. Soil-structure interaction effect is considered in the analyses and the results are compared with the hypothetical fixed-support end condition. Static and dynamic pushover analyses are performed by using wave and seismic loads respectively. From the analyses, it is found that the optimum leg batter varies between 15 to 16 and 2% of weight saving is achieved. Moreover, it has been observed that the type of bracing does not play a major role in the seismic design of jacket platform considering the soil-structure interaction.

**Key words:** Fixed offshore jacket platform, pushover analysis, seismic analysis, leg batter, optimization, SACS.

## 1. Introduction

Advances in reservoir assessment and recovery techniques, subsea technology, seismic and directional drilling techniques extend field life and impose higher demands on existing offshore platforms to support additional vertical and lateral loads. Jacket platforms are fixed-base offshore structures that are used to produce oil and gas in relatively shallow water, generally less than 500 ft. These structures contribute to a significant percentage of the world's offshore platforms. They are subjected to various environmental loads during their life time. A jacket-type platform consists of three parts: Jacket, Pile and Topside. The primary function of a jacket structure is to transfer the lateral loads to the foundation and also support the weight of the topside structure. In seismically active areas, pile supported structures may be subjected to strong ground motions where the behavior of pile foundations under lateral seismic loads becomes an important factor in assessing the performance of such structures. Pile foundations are an essential structural component of jacket type offshore platform and the seismic soil-pile-superstructure interaction is an important concept in the seismic behavior. Strong ground motions have been a major cause of past damage in pile foundations.

Nonlinear static and dynamic analyses of fixed offshore platforms have been carried out by many researchers in last decades. Venkataramana et al. [1] proposed a method for dynamic response analysis of offshore structures subjected to simultaneous wave and seismic loading and concluded that the sea waves act as a damping medium and reduce the amplitude of seismic response of offshore structures. Stear et al. [2] validated the utility of static pushover analysis on two

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Gulf of Mexico platforms to assess its lateral load capacity. It was concluded that the presence of additional horizontal framing has little effect on the first member failure in the jacket. Mwafy et al. [3] assessed the applicability of static pushover analysis by comparison with dynamic pushover analysis, by developing complete pushover curves from incremental dynamic analysis up to collapse for RC (reinforced concrete) buildings. Mostafa et al. [4] investigated the response of fixed offshore platforms supported by clusters of piles and concluded that the top soil layers have an important role in the response of the structure. Nejad et al. [5] investigated the effect of leg batter on overall cost of the Persian Gulf platform and found the optimum batter as 11 which lead to weight reduction of steel by 4%. Bargi et al. (2011) performed nonlinear dynamic analysis of a Persian Gulf platform and showed that the maximum displacement response of platform under the combination of earthquake and wave loads were more than the maximum displacement response of earthquake load alone. Bonessio et al. [6] formulated an analytical model of BRBs (buckling restrained braces) for structural optimization. Potty et al. [7] assessed the ultimate strength of steel jacket offshore platform under wave loads and found that X-bracing contributes highest rigidity to the whole platform. Nasseri et al. (2014) applied genetic algorithm for optimizing the design of fixed offshore structure subjected to environmental loads and determined the contribution of member types in optimization. El-Din et al. [8] evaluated the seismic performance of a fixed-base jacket structure by nonlinear static analysis and incremental dynamic analyses, and compared the effects of various retrofit schemes. Nguyen et al. [9] investigated the nonlinear behavior of a fixed-jacket offshore platform with different bracing systems and found that X-bracing system has a high seismic performance.

Based on a simplified plastic collapse model, Lee [10], has reported the piles in the system collapse when one hinge forms at the pile head and a second hinge forms at some depth below the pile head. The collapse of the entire system occurs when two hinges form in each of the piles in the system, as shown schematically in Fig. 1.

A significant finding from the performance of jacket platforms in major hurricanes, including Refs. [11-15], is that pile foundations performed better than expected [16-18]. While there are few cases with “suspected” foundation damages or failures (Fig. 2), there are no direct observations of such damages and

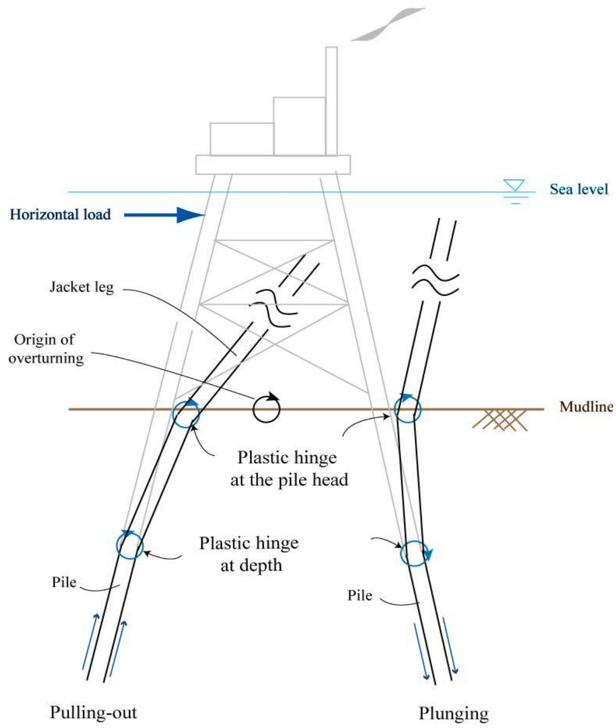


**Fig. 1 Schematic diagram of pile system collapse due to the formation of plastic hinges [10].**

**Fig. 2 Platform with "suspected" foundation failure [22].**

failures during these hurricanes. Assessment of jacket platforms subjected to environmental loads greater than their original design loading frequently indicates that the capacity of the structural system is governed by the foundation. In addition, there were several hundred platforms damaged in these hurricanes, and yet only a few cases of pile foundation failures have been reported. Therefore, there is a need to better understand and quantify the potential conservatism in foundation design for the purpose of assessing the platforms. Reliable evaluation of the dynamic response of jacket type offshore platform against lateral loads plays an important role in the design. From the literature, it is also evident that the difference between the response of the jacket structure with fixed support end condition and with PSI (pile-soil-structure interaction), under static and dynamic lateral loads, needs to be addressed.

In this study, at first, a typical four legged jacket-type offshore platform is investigated for its structural response by performing static and dynamic wave response analyses. Then, time history earthquake analysis is carried-out. Thirdly, inelastic nonlinear static pushover analyses have been performed using wave and seismic loads separately. Further, weight optimization has been carried out by analyzing different values of leg batter, varying from $3^\circ$ to $14^\circ$ slopes. The analyses were performed using finite element software, SACS, which are widely used in practice for modelling soil-pile interaction.

## 2. Methodology

### *2.1 Three-Dimensional Finite Element Method Model of Platform Structure*

2.1.1 Description of 3-D FEM Model

The objective of the quantitative analysis is to estimate the capacity of the foundation system. SACS™ (structural analysis computer software) was used to conduct 3-dimensional Finite Element Method analyses of the case study platforms. SACS™ is a suite of modular software developed by Engineering Dynamics, Inc. for use in both offshore structures and general civil engineering applications [19-21]. Use of this software was made possible to the senior author through the generous permission and support of NIOT (National Institute of Ocean Technology), Chennai. The inputs to this model are the structural properties of all members and connections including the piles, the behavior of the soil surrounding the piles (i.e., *T-z* and *p-y* curves as a function of depth along each pile and a *Q-z* curve at the tip), and the environmental loading including the magnitude and direction of waves and current. The primary output from this model includes the total load on the structure, typically expressed as a base shear, the displacement of the deck, and the loads, moments and deformations in individual members.

2.1.2 Theoretical Basis

The load-displacement relationship of a jacket structure is determined using large deflection, elasto-plastic, nonlinear, finite-element analysis. A full plastic collapse (pushover) analysis is performed to determine the load at which the structure collapses. The solution process involves three levels of iteration. For any global load increment, a beam-column solution is performed for each plastic member using the cross section sub-element details. The global stiffness iteration is then performed including the effects of connection flexibility, plasticity and failure and the foundation stiffness iteration including the nonlinear pile/soil effects. During any global solution iteration, the deflected shape of the structure is determined and compared to the displacements of the previous solution iteration. If convergence is not achieved, the new global displacements of the joints along with the beam internal and external loads are used to recalculate the elemental stiffness matrices. The structural stiffness iteration is then repeated including the effect of the foundation until the displacements meet the convergence tolerance.

**Non-linear pushover analysis.** Static pushover analysis is the application of a single load, applied to any specific location which is incremented in steps until collapse. COLLAPSE module is used to perform pushover analysis of the jacket type offshore platform. The COLLAPSE module employs elastic element, the plasticity model and the failure criteria and uses basic energy variation principles [23].

(1) The elastic element follows an updated Lagrange (incremental iterative) procedure and uses a nonlinear green strain formulation with the von Karman approximation. The COLLAPSE beam element is valid for large displacements and the stiffness formulation derived from potential energy consideration. Closed-form solutions of the nonlinear elastic stiffness matrix are derived using the shape function which represents the solution of the 4th order differential equation for a beam-column.

(2) The plasticity model is represented by concentrated yield hinges to reflect the nonlinear material behavior. Hinges may be introduced at element ends and/or element midspan. The plasticity model is formulated in stress resultant ("force") space based on the bounding surface concept. Two interaction surfaces are used, one yield surface representing first fiber yield and another bounding surface representing the full plastic capacity of the cross section. The model allows for an explicit formulation of the beam-element stiffness matrix, including geometrical nonlinearities and nonlinear plastic behavior with material hardening and gradual plastification of the cross section.

(3) Failure is predicted in SACS COLLAPSE module in accordance with failure criteria and design formulation. The formation of a yield hinge is not the limit of the load-carrying capacity and the structural failure is indicated once a sufficient number of plastic hinges have formed to make a kinematic mechanism. The peak force in the member $P$-$\delta$ behavior defines column buckling. The non-linear formulations automatically calculate the total ($1^{st}$ + $2^{nd}$ order) bending moments and also the effects of cross-section geometry, boundary conditions and loading.

**Pile-soil foundation models as per API RP2A LRFD.** Soil resistance is assessed at each node of the pile segment and represented as a nonlinear spring. The axial load transfer (*T-z*) normalized curves for clay and sandy soils as recommended by API RP2A LRFD are shown in Figs. 3 and 4. Fig. 5 shows the unit end bearing capacity and normalized *Q-z* curve for API clay and API sand. The ultimate lateral soil resistance and *P-y* curves may be calculated using sections G.8.2 to G.8.7 of API RP2A LRFD. The soil resistance equations are not applicable, if the strength

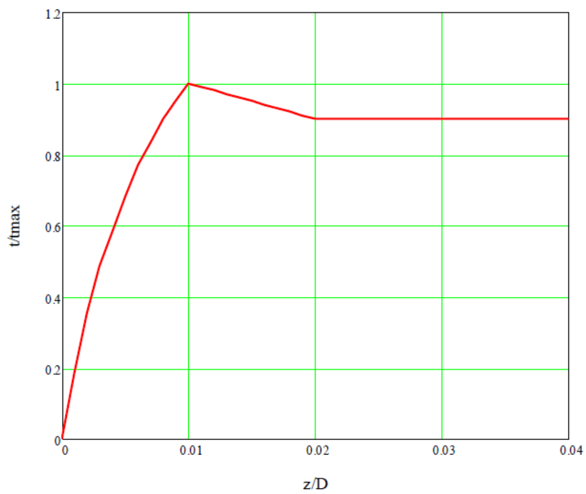


**Fig. 3 Normalized *T-z* curve for API clay [24].**

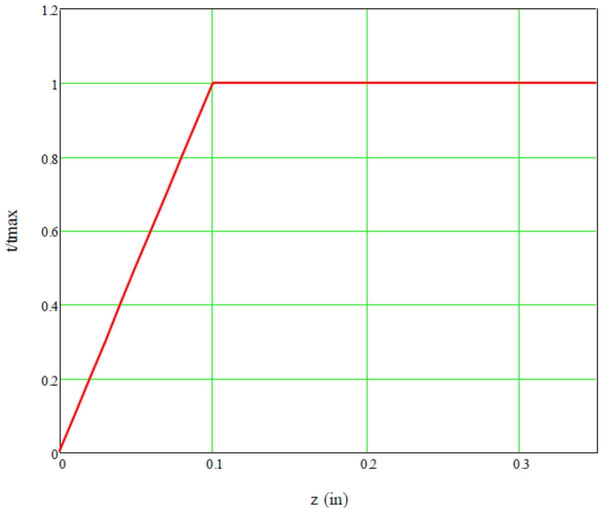


**Fig. 4 Normalized *T-z* curve for API sand [24].**

Notation used for axis labels is defined as

*t* = mobilized skin friction;

$t_{max}$ = maximum unit skin friction capacity (*f*);

*z* = local pile deflection;

*D* = diameter of pile.

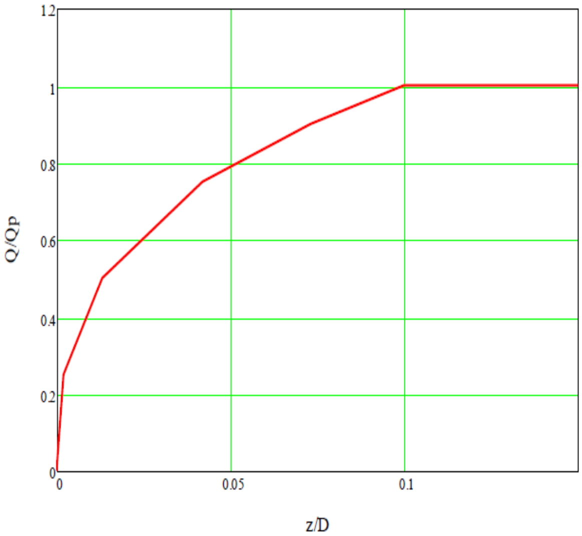


**Fig. 5 Normalized *Q-z* curve for API clay and API sand [24].**

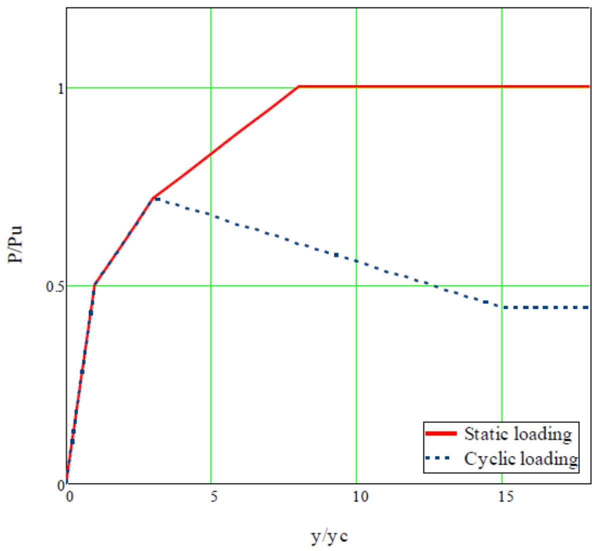


**Fig. 6 Normalized *P-y* curve for API clay ( for z < $X_R$ ) [24].**

variation along the depth of the soil is inconsistent. The ultimate unit lateral resistance, $p_u$, of soft clay under static loading conditions can vary between 8*c* to 12*c* (*c*: undrained shear strength of undisturbed clay soil sample) except at the shallow depths. The *P-y* curves of API clay under cyclic and static loading conditions are defined as a piecewise linear function and shown in Figs. 6 and 7.

Fig. 8 shows the 3D model of the jacket. The platform considered in the study is a typical four legged production platform made from tubular steel. Water depth at the North Sea location considered is 75 m. Total height of the platform (jacket and deck) is 90 metres and has a square cross-section in plan. The platform is designed based on the API RP 2A-WSD and DNV standards. The major deck framing is 20 m by 20 m in plan, dimension at the foot is 32 m by 32 m and the jacket legs are battered at 1 to 12.5 (horizontal to vertical) in both broad side and end on framing. The jacket is subdivided into 3 bays of 25

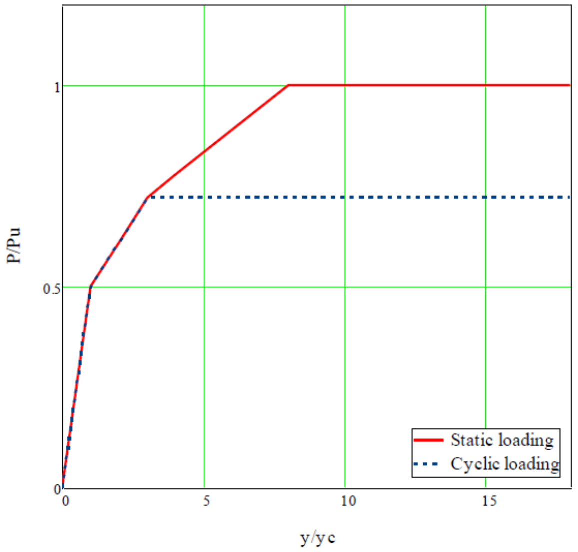


**Fig. 7 Normalized *P-y* curve for API clay (for $z \geq X_R$) [24].**
Notation used for axis labels is defined as
$P$ = actual lateral resistance in stress units;
$P_u$ = ultimate lateral bearing capacity in stress units;
$y$ = actual lateral deflection;
$y_c$ is defined as: $y_c = 2.5\ \varepsilon_c\ D$.
where $\varepsilon_c$ = strain occurring at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples and $X_R$ = depth from the ground surface to the bottom of reduced resistance zone.

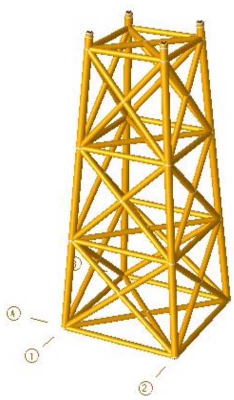

**Fig. 8 SACS 3D model of jacket.**

meters each with X-bracing. It is assumed that the jacket structure is analyzed with the load from topside acting as equivalent concentrated vertical load on the piles and the wind effect is neglected as the jacket is under water and wind effects are not considered in the seismic analysis.

The different types of bracing configurations considered in the study are shown in Fig. 9.

*2.2 Material and Geometric Properties*

Tables 1 and 2 show the material and geometric properties of the jacket respectively. The length of the pile is 100 meters below mud-line.

*2.3 Loading Data*

(1) Dead load

The total deck load of 4,800 tonnes is applied as concentrated vertical load on the 4 piles.

(2) Wave and current load

Wave load has been generated using Stoke's fifth order equation. The maximum directional wave heights for the 100-year return period are given in Table 3.

Surface current velocity is considered as 1 m/s for operating condition and 2 m/s for extreme condition.

(3) Soil data

The following (Table 4a) soil layering data are used for the analysis. Soil properties vary with depth and are characterized by the shear wave velocity $V_s$ and unit weight $\gamma$. Han et al. [25] have reported the soil properties as shown in Table 4b.

(4) Load combinations

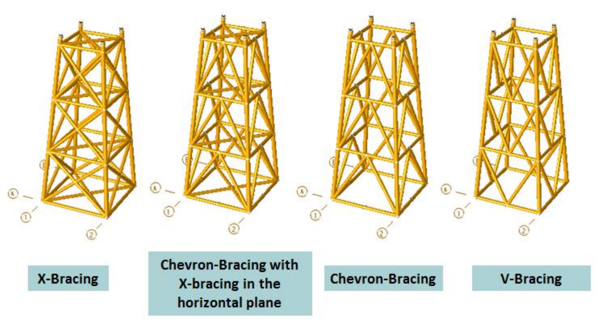


**Fig. 9 Different types of bracing configuration.**

**Table 1 Material properties and mass of deck [9].**

| Parameter | Value |
|---|---|
| Young's modulus | $2 \times 10^8$ kN/m$^2$ |
| Poisson's ratio | 0.3 |
| Steel density | 78.5 kN/m$^3$ |
| Yield stress | $3.2 \times 10^5$ kN/m$^2$ |
| Ultimate stress | $4 \times 10^5$ kN/m$^2$ |
| Mass of the deck | 4,800 tonnes |

**Table 2 Geometric properties [9].**

| Member | Diameter (mm) | Thickness (mm) |
|---|---|---|
| Horizontal bracings and | 1,500 | 20 |

| | | |
|---|---|---|
| diagonals | | |
| Diagonals (vertical plane) | 1,600 | 30 |
| Legs | 2,000 | 50 |
| Piles | 1,900 | 40 |

**Table 3 Wave load data [4].**

| Return period (year) | Wave height (H) in meter | Height above MSL (a) in meter | Wave period mean value (T) in seconds |
|---|---|---|---|
| 1 (operating condition) | 22.5 | 12.8 | 13.8 |
| 10 | 25.3 | 14.2 | 14.6 |
| 100 (extreme condition) | 28.5 | 16.1 | 15.3 |
| 10,000 | 36 | 20.4 | 17.1 |

**Table 4a Soil layering data [4].**

| Soil unit | Depth (m) | Soil description |
|---|---|---|
| A | 0-7.5 | Very soft to soft silty, sandy clay |
| B | 7.5-32 | Sandy, clayey silt |
| C | 32-47 | Very stiff to hard silty clay |
| D | 47-52 | Very dense fine sand |
| E | 52-125.5 | Very stiff to hard clay |
| F | >125.5 | Very hard clay |

**Table 4b Soil properties.**

| Soil | $\gamma$ (kN/m$^3$) | $V_s$ ( m/s) |
|---|---|---|
| Soft clay | 18 | 130 |
| Fine sand | 18 | 140 |
| Stiff clay | 20 | 300 |
| Silty sand | 19 | 240 |
| Silty clay | 18 | 300 |
| Shale | 18 | 200 |
| Sand | 20 | 300 |

The analysis is done for both operating and extreme conditions in eight directions as specified by the API code (Fig. 10).

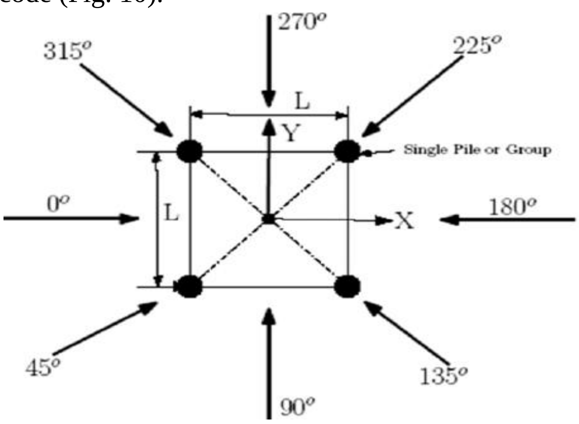


**Fig. 10 Wave load directions (API guidelines).**

## 3. Results and Discussions

*3.1 In-place Analysis*

In-place analysis is done to check the global integrity of the structure against premature failure. The following steps were performed in the in-place analysis.

(1) Static analysis of the structure under wave load acting in eight directions, as per API guidelines;

(2) Free vibration analysis to obtain natural time period for corresponding modes;

(3) Dynamic wave response analysis in 8 directions, using the significant wave height and time period.

3.1.1 Static Analysis

The combined utilization ratio check for the members showed that the stresses are within allowable limits and also the maximum utilization ratio (Eq. 1) of critical members was lesser than one for both fixed support condition and for PSI condition. The maximum horizontal joint displacements were within allowable limits (33% of length of the member) as per API. The utilization ratio is defined as

$$Utilisation\ ratio = \frac{actual\ performance\ value}{maximum\ allowable\ performance\ value} \quad (1)$$

3.1.2 Free Vibration Analysis

- Free vibration analysis was performed to obtain the natural time period of the structure.
- The natural time periods of the structure in the 1$^{st}$ and 2$^{nd}$ modes were very close with a value of 2.7 s under fixed support condition.
- The natural time periods of the structure with PSI condition for the 1$^{st}$ and 2$^{nd}$ modes were similar to that for the fixed condition with a value of 2.9 s.

3.1.3 Wave Response Analysis

Wave response analysis was carried out with P-M wave spectrum (Eq. 2). Maximum hydrodynamic force obtained was 7,880 kN in X-direction. Table 5 shows the results of wave response analysis.

$$S(f) = \frac{\alpha g^2}{(2\pi)^4 f^5} \exp\left(-\frac{B}{f^4}\right) \qquad (2)$$

where, α = 8.1 × 10−3 and is called the Phillips' constant and B = 0.74(g)/2πU)4.

Wave response analysis results are given in Table 5.

### *3.2 Earthquake Analysis*

Seismic analysis was carried out using El-Centro Time History (Fig. 11) which has duration of 31.18 sec and maximum acceleration of 312.76 cm/s$^2$.

The results of the earthquake analysis under both fixed support condition and with PSI are given in Table 6. The joint location considered for joint displacement is shown in Fig. 12.

**Table 5 Results of wave response analysis.**

| Response | Fixed support | PSI condition |
|---|---|---|
| Max. overturning moment (kN m) | $7.3 \times 10^5$ | $1.02 \times 10^6$ |
| Max. base shear (kN) | 12,083 | 16,608 |

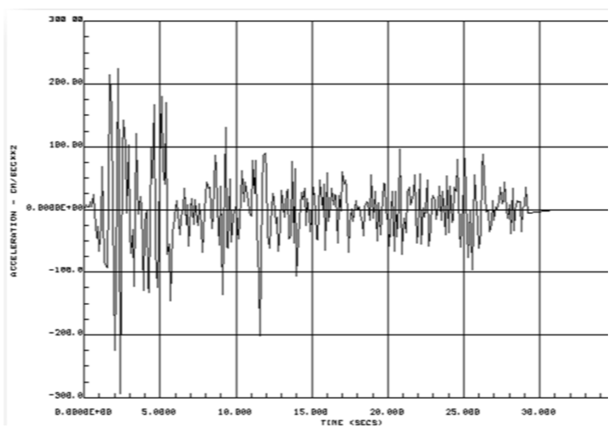

**Fig. 11 El-Centro time history.**

**Table 6 Results of seismic analysis.**

| Response | Fixed support | PSI |
|---|---|---|
| Overturning moment | $4.2 \times 10^6$ kN m at 22.6 s | $2.3 \times 10^6$ kN m at 23.4 s |
| Base shear | 114,097 kN at 6.3 s | 60,00 0 kN at 28 s |
| Joint displacement 401 L | 42.9 cm at 26 s | 28.3 cm at 23.6 s |

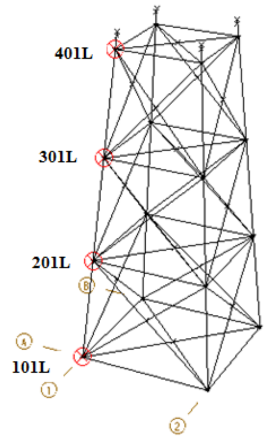


**Fig. 12 Wireframe 3D model in SACS.**

Seismic analysis is performed by varying the bracing configurations and the results are shown in Table 7.

It is evident that the maximum responses are greatly reduced by considering PSI under seismic analysis.

### *3.3 Pushover Analysis*

The SACS program module COLLAPSE is used to perform pushover analysis of the jacket-type offshore platform. Pushover analysis is a static nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement controlled lateral load pattern which continuously increases through elastic and inelastic behavior until an ultimate condition is reached. In this study, two separate pushover analyses were carried out, one with wave load and another with seismic load.

3.3.1 Static Pushover Analysis Using Wave Load

Wave load in a particular direction is incremented up to collapse of the structure. The base shear value at formation of first hinge is not the limit of the load carrying capacity. Pushover analysis is performed on the four chosen bracing configurations under both fixed and soil conditions. The results obtained in the static pushover analysis using wave load are shown in Figs. 13-19.

Important observations from the pushover analysis are as follows:

(1) Base shear value at the formation of first hinge

is not the limit of the load-carrying capacity.

(2) The type of bracing does not play a major role in the design of jacket considering soil-structure interactions.

(3) It is economical to use a batter that gives minimum weight of the structure and a realistic analysis that takes into account the soil conditions.

3.3.2 Pushover Analysis Using Seismic Load

Pushover analysis of the Jacket type offshore platform due to earthquake loads is performed to ascertain the response and reserve strength against future extreme intensity events. Pushover analysis performed on X-bracing with leg batter value of 12.5 for fixed

**Table 7 Seismic analysis on various bracing configurations.**

| | X-bracing | X+ Chevron bracing | Chevron bracing | V-bracing | X-bracing | X+ Chevron bracing | Chevron bracing | V-bracing |
|---|---|---|---|---|---|---|---|---|
| [1] Fired support condition | | | | | [2] PSI condition | | | |
| Natural time period (s) | 2.61 | 2.86 | 2.84 | 2.62 | 2.92 | 3.15 | 3.11 | 2.91 |
| Max. base shear (kN) | 53,661 at 11.2 s | 48,212 at 6 s | 47,818 at 6 s | 44,741 kN at 7 s | 63,857 kN at 10.3 s | 53,252 at 13.6 s | 47,405 at 13.5 s | 56,785 kN at 11.9 s |
| Max. overturning moment (kN m) | $3\times10^6$ kN m at 5.7 s | $3.25\times10^6$ kN m at 11.6 s | $3.22\times10^6$ kN m at 11.6 s | $2.81\times10^6$ kN m at 12.5 s | $3.2\times10^6$ kN m at 11.6 s | $2.33\times10^6$ kN m at 12.3 s | $2.32\times10^6$ kN m at 13.8 s | $3.24\times10^6$ kN m at 11.6 s |
| Max. joint displacement (joint 401 L) (cm) | 32 cm at 7 s | 44 cm at 11.6 s | 43.2 cm at 11.6 s | 32.5 cm at 12.5 s | 42 cm at 13.2 s | 35.5 cm at 10.7 s | 36.08 cm at 10.6 s | 42 cm at 13.2 s |
| Weight (kN) | 28,353.13 | 26,236.78 | 24,079.47 | 23,824.14 | 28,353.13 | 26,236.78 | 24,079.47 | 23,824.14 |

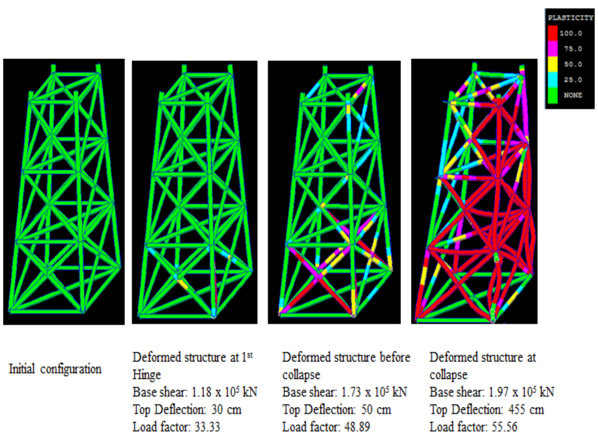


**Fig. 13 Deformation stages due to pushover analysis on X-bracing configuration-fixed end condition.**

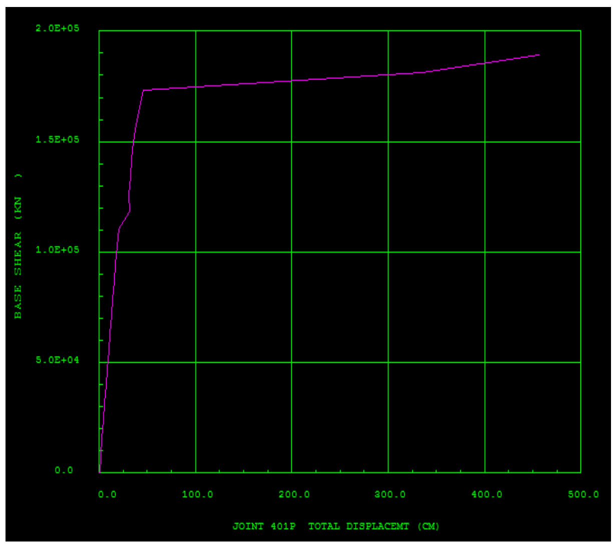


**Fig. 14 Pushover curve on X-bracing configuration-fixed end condition.**

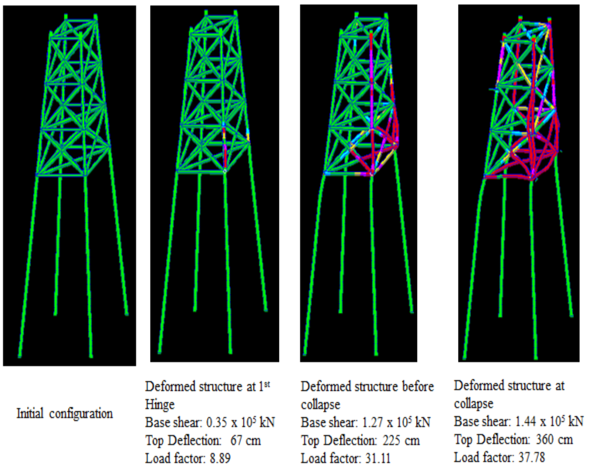


**Fig. 15 Deformation stages due to pushover analysis on X-bracing configuration-PSI.**

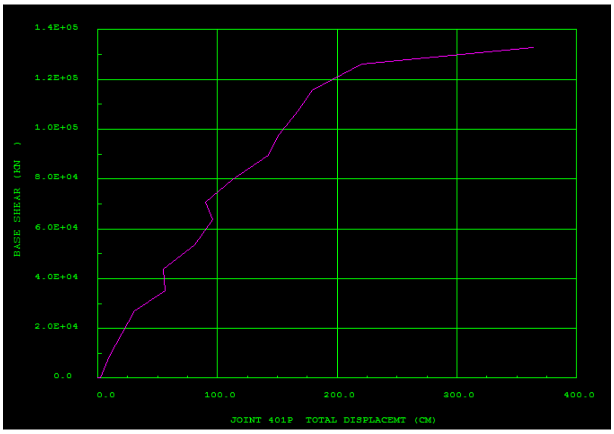


**Fig. 16 Pushover curve on X-bracing configuration-PSI.**

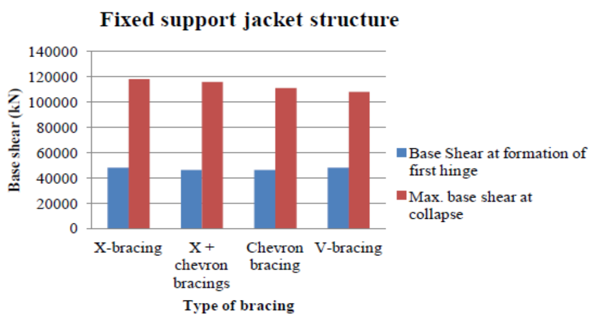


**Fig. 17 Ultimate strength of jacket with different bracing configurations-fixed end condition.**

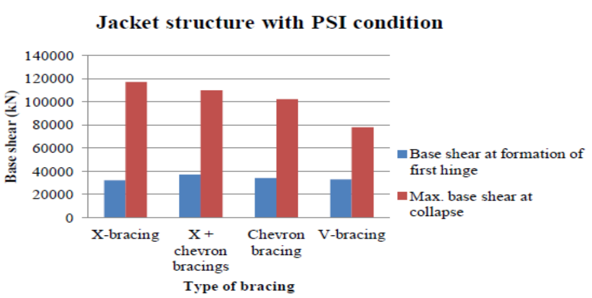


**Fig. 18 Ultimate strength of jacket with different bracing configurations-PSI.**

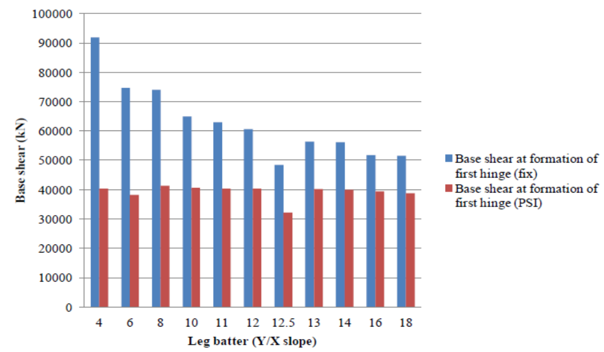


**Fig. 19 Graphical representation of results for pushover analysis with wave load.**

end-support condition and with soil-structure interaction. Equivalent seismic load was calculated using the lumped mass values, peak acceleration value and zone factor. Figs. 20-23 show the results obtained by performing pushover analysis with seismic load.

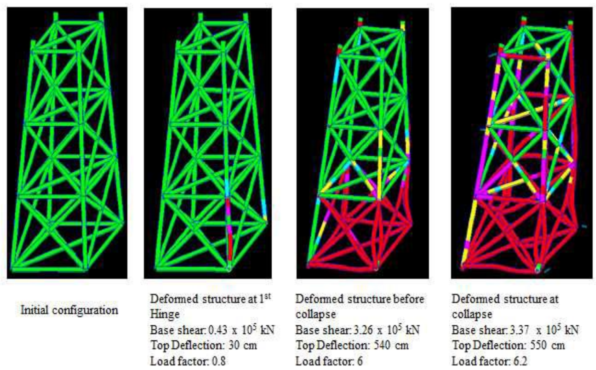


**Fig. 20 Deformation stages due to pushover analysis with seismic load results-fixed end condition.**

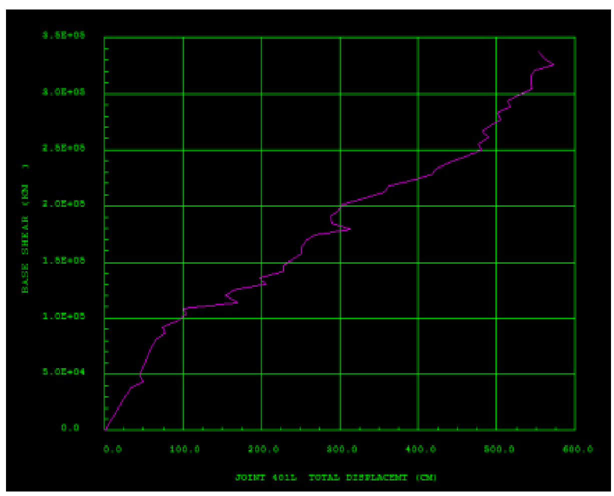


**Fig. 21 Pushover curve on X-type bracing configuration**

**for seismic load-fixed end condition.**

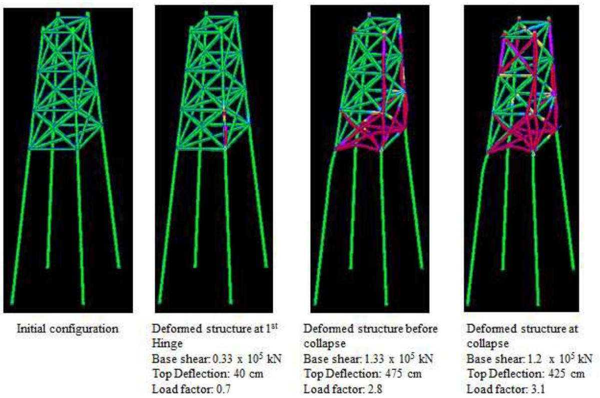


**Fig. 22 Deformation stages due to pushover analysis with seismic load results–PSI condition.**

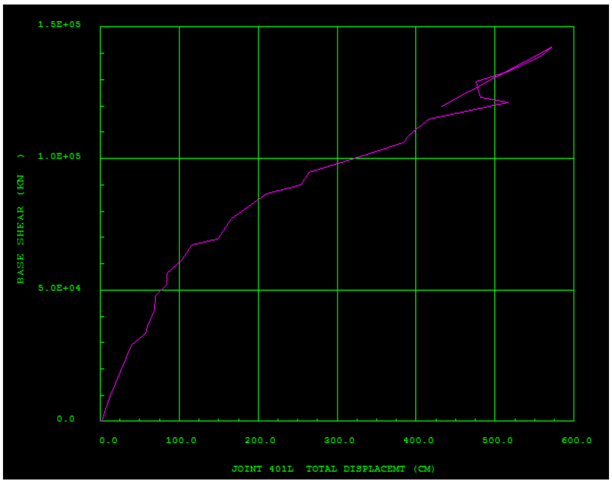


**Fig. 23 Pushover curve on X-type bracing configuration for seismic load-PSI condition.**

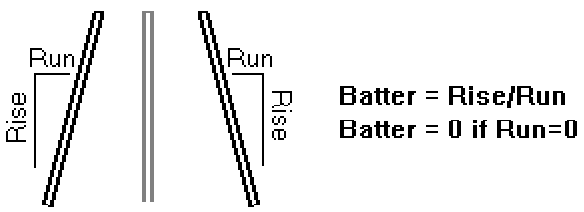


**Fig. 24 Jacket leg batter.**

The maximum base shear at first hinge formation is 30% more in fixed end support structure in comparison with structure with soil conditions.

### *3.4 Optimization of Leg Batter*

In this study, the chosen jacket type platform is investigated for optimum weight. The jacket leg batter (Fig. 24) is an important parameter which influences the jacket total weight. Therefore, eleven jackets with a range of 3° to 14° (4 to 18 vertical to horizontal) slope in leg batter have been modelled and analyzed by a finite element program, SACS.

For optimization with variation of leg batter, members are redesigned until stress ratio for critical members in 11 jackets became equal to each other. The optimum leg batter obtained is 16. Fig. 25 shows the graphical representation of variation of weight with leg batter and Fig. 26 shows the variation of natural frequency of structure with leg batter.

The results of nonlinear static and dynamic analysis of control structure and the optimized structure are presented in Table 8.

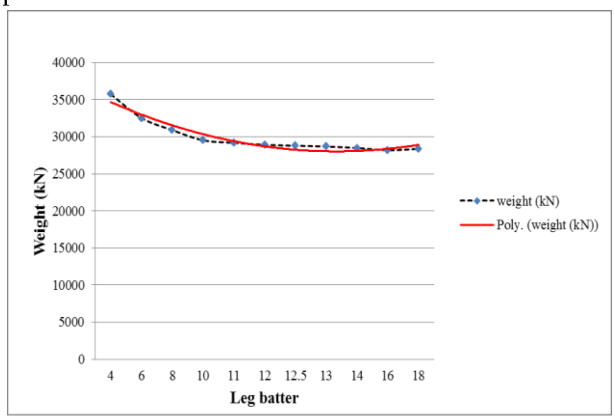


**Fig. 25 Optimisation of leg batter.**

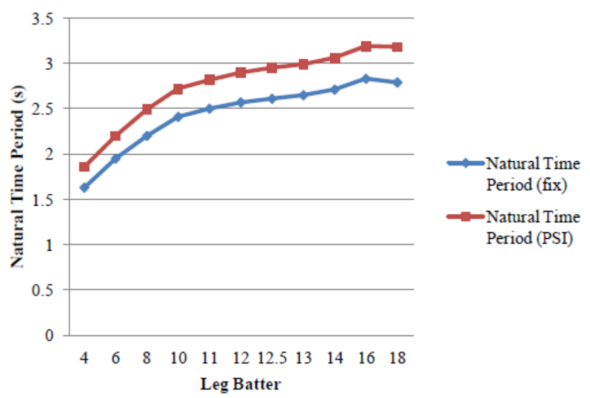


**Fig. 26 Leg batter vs. Natural time period.**

**Table 8 Control structure vs. optimized structure results.**

| Analysis type | Control structure (fix) | Control structure (PSI) | Optimized structure (fix) | Optimized structure (PSI) |
|---|---|---|---|---|
| Weight | 28,793 kN | | 28,178 kN | |
| Free vibration (time period) | 2.61 s | 2.95 s | 2.75 s | 3.13 s |
| Wave response (base shear) | 13,252 kN | 9,795 kN | 10,777 kN | 10,552 kN |
| Response spectrum (base shear) | 46,000 kN | 52,000 kN | 54,595 kN | 49,200 kN |
| Time history (base shear) | 53,661 kN | 63,857 kN | 50,000 kN | 51,442 kN |
| Pushover wave (base shear) | 48,000 kN | 32,000 kN | 52,000 kN | 39,000 kN |
| Pushover seismic (base shear) | 43,000 kN | 33,000 kN | 63,000 kN | 41,000 kN |

## 4. Conclusions

The following conclusions are drawn based on the analytical investigations carried out using SACS to study the nonlinear static and dynamic response of offshore jacket structure.

(1) The natural frequency of the structure is around 2.9 seconds which is well within the range for fixed offshore structures.

(2) The maximum base shear obtained from dynamic wave response analysis for fixed support condition is about 35% more than that of analysis with soil-structure interaction.

(3) Base shear and overturning moment values due to earthquake forces for jacket with soil condition are about 25% more in time history analysis than response spectrum method. The response spectrum method is an approximate method used to estimate maximum peak values of displacements and forces and it has significant limitations. It is restricted to linear elastic analysis in which the damping properties can only be estimated with a low degree of confidence.

(4) The pushover analysis on the jacket structure shows that it possesses strength substantially in excess of original design loads.

(5) Base shear value at formation of first hinge is not the limit of the load-carrying capacity and structural failure is indicated once a sufficient number of plastic hinges have formed to make a kinematic mechanism.

(6) The type of bracing does not play a major role in the seismic design of jacket considering soil-structure interaction since the first member failure occurs in almost similar base shear values for jacket structure with various bracing conditions.

(7) The ultimate strength of jacket structure under seismic load and wave load is nearly the same with marginal variation of 5%.

(8) The weight of the chosen jacket has been reduced by 2% with optimized leg batter of 16.

(9) The optimized structure has more flexibility and has 25% more ultimate strength capacity in comparison with the actual control structure.

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