International Journal of Marine Engineering Innovation and Research, Vol. 10(3), Sept. 2025. 705-726
(pISSN: 2541-5972, eISSN: 2548-1479)

Sensitivity Study of Dynamic Response and Seismic Performance of Offshore Tripod Structure Under Variation of Center of Gravity Location

Novan Arif Hidayat[1], Daniel Mohammad Rosyid[2], Yoyok Setyo Hadiwidodo[3]

(Received: 26 Juni 2025 / Revised: 18 July 2025 /Accepted: 25 July 2025 / Available Online: 5 September 2025)

Abstract*: An effective weight control is critical for managing gravity loads and the center of gravity (CoG) evolution in offshore platforms. International standards, including ISO 19901-5:2021, require that CoG shift envelopes—defined in 2D (X–Y) and 3D (X–Y–Z) coordinate systems—be considered when assessing pile foundation capacity and global structural behavior. This study examines the sensitivity of the dynamic response and seismic performance of a three-legged fixed offshore structure (tripod) due to variation of CoG location. Tripod known have characteristic as slender structure and long natural period that make it particularly sensitive to gravity loads, lateral forces, and soil variability. A systematic sensitivity analysis was performed using 75 cases, spanning four directional quadrants in both 2D and 3D (upward and downward) associated with 0%, 10%, and 20% of topside operating mass increment. Dynamic and in-place analyses incorporating pile-soil interaction (PSI) were conducted using SACS software. Seismic performance was evaluated through response spectrum–based equivalent static methods. The results show variation of CoG location influenced the dynamic response and structural performance. The response determined significantly amplify if the CoG variation associated with mass increment, particularly under in-place conditions that exhibited greater sensitivity than seismic condition. The study also proposes a CoG shift envelope that maintains acceptable structural performance and recommends its adoption in future design and modification planning. Emphasis is placed on active CoG management and the necessity for future non-linear assessments to capture post-elastic behavior and ensure the robustness of tripod platform design.

Keywords – Center of Gravity (CoG); Dynamic Response; Natural Periods; Offshore Tripod Structure; Seismic Performance.; Sensitivity Study;

Corresponding Author:6020211006@students.its.ac.id

I. INTRODUCTION

Offshore structure is a supporting system utilized in the exploration and production activity in the industry of oil and gas, harnessing power from the sea telecommunication terminal system, etc. For oil and gas industry in Indonesia waters particularly, there are around 634 offshore platform facilities installed. About 527 platforms are still actively in operation condition and 107 units are no longer operational in which including seven units that have been decommissioned as part of effort for environmental conservation and safety initiatives [1].

There are two types of offshore structure; one is fixed to the seabed (called as fixed structure) and the other is floating in the sea water [2]. A steel jacket platform is one of common types of fixed substructure that exists worldwide. This type of substructure consists of a tower that supports the superstructure [3] with various shape such as monopod, bipod, three legged (or tripod), four legged, etc.

The integrity and sustainability of the structure on its position during all weather conditions is required to keep the continuous production until its desire design life or beyond. Life extension of the facility to earn more benefit in economic and minimize the impact to the environment also important aspect aimed by the facility operator. Structural integrity refers to structure's capacity to endure the intended design load without failures on its components, whether primary or secondary parts [4]. Common principle of "design it right, build it right, operating it right, and maintain it right" play an important role and as a strong basis to achieve reliable structure along the desired or even beyond its design lifespan. During the design stage, the engineer shall properly perform the structural analysis (includes modelling of the structure, loadings, etc.) and set any design assumptions and contingencies to consider potential changes during construction up to in operation condition.

There are two of three basic requirements of ability to be sustained by the fixed offshore structure [5] relates with gravity load:

a) All design loads that expected or potential expose to structure during fabrication (onshore construction), transportation (land or sea), and offshore installation.
b) Loads imposed from severe environmental and earthquakes.
c) Function safely as combination of drilling, production, and offshore accommodation (LQ).

Novan Arif Hidayat is with Department of Ocean Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, 60111, Indonesia. E-mail: 6020211006@students.its.ac.id
Daniel Mohammad Rosyid is with Department of Ocean Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, 60111, Indonesia. E-mail: dmrosyid@oe.its.ac.id
Yoyok Setyo Hadiwidodo is with Department of Ocean Engineering, Institut Teknologi Sepuluh Nopember, Surabaya, 60111, Indonesia. E-mail: yoyoksetyo@students.its.ac.id

The gravity load contributes 60% to 70% of the total imposed load to the offshore structure [6]. Therefore, it is important to carefully evaluate the topsides gravity loads, particularly in the early stages of the project as neglecting this load – which will influence the structure's center of gravity (CoG) – can result in an underestimation of fatigue damage by as much as 24%. Impact to slenderer offshore structure type could be more significant [7].

Weight Control Procedure is a soft tool widely used to record and monitor the presence of gravity load on the offshore platform. Through this procedure, the CoG of the corresponding weight is regulated or managed from the design stage and is expected to be maintained throughout the facility's life, even up to preparation of decommissioning activity [8],[9],[10]. Most likely any unmature information available in the beginning of design stage such as non modelled structural element, equipment, piping, etc., will be assumed based on the historical data. Consequently, there is a significant likelihood of weight overruns occurring throughout the project lifecycle.

A comprehensive examination performed by the Independent Project Analysis to 153 offshore greenfield projects (from concept selection till authorization gates). The growth of weight about 21% found started from FEL–2 (Front End Loading–2) end stage to the completion, along with a 10% increase as the projects progress towards completion [11]. Although, the integrity of an existing offshore operating platform typically will not be jeopardized due to topsides weight growth less than 10% [12].

The integrity of structure is evaluated considering the static inplace, dynamic, seismic, and fatigue condition during engineering design phase. In those analyses, the gravity load is converted into mass and the properties of dynamic response (natural frequency, mode shapes, etc.). The mass should include platform's modelled self-weight (topsides, substructures, conductors), un-modelled weight (structures' appurtenances, equipment), marine growth (that accumulated at the jacket member), entrapped water inside the submerged substructure members, and submerged members [13]. Therefore, the accuracy of the input of design gravity load will affect the platform's natural period. Erroneous load predictions may result in under or overdesigned that causing a risk to safety and economic.

Examination on the influence of variations in model mass on structural behavior was conducted by utilizing a scaled-down offshore jacket platform, the actual dynamic responses against the predictions generated by a finite element model are assessed. The result highlighted two significant trends: an increase in the model's mass resulted in diminished foundation reactions, while heavier deck loads corresponded with reduced natural frequencies [14]. Subsequently, [15] and [16] investigated the uncertainties associated with modeling pile-supported steel jackets subjected to seismic forces. These variables are categorized into three categories for sensitivity analysis and found that mass, elastic modulus, and plastic hinge properties had a measurable—though not dominant—impact on structural performance. Hence, the change of structure mass also affects the performance of offshore structure under seismic condition.

Appropriate method of installation and excellent levelling shall be conducted to ensure the platform installed evenly. Tilting of offshore structure due to any error during installation or seabed condition (inclined seabed floor) or effect of natural geohazard (settlement, subsidence, etc.) will shift the mass and CoG location of the structure with respect to seabed elevation. This condition alters the structural behavior of the platform as reported by [17] and induces torsion effects that are influenced by the location and length of the individual legs [18].

Addition of facilities and increased of loading on the existing offshore structure are among the six (6) items to trigger up the initiation of mandatory assessment work to be performed to the existing offshore platform [13]. These two items automatically correspond with the change of the CoG location of the structure. Thus, keeping the record of platform's inspection and proper modification (such as meticulous layout study for adding new equipment or new deck extension, etc.) are example of excellent in maintaining the structural integrity of the facility.

At the decommissioning stage, the offshore platform can be disposed through partial or complete removal. For complete set removal, i.e. topsides or jacket, many single lift methods of topsides are summarized from year of 2016 till 2020 with various topsides weight [19]. Afterward, the structure will be put on the barge for transportation to onshore yard. As normal practice of lifting and transportation, the accurate data of weight of the structure including dimension and CoG is one of most important among other key parameters used for lifting and transportation analysis which commonly considering the CoG shift cases to ensure the structural integrity of the existing lifted parts and lifting points. The result will influence the selection of removal method; either through remove in one piece (single lift similar to reverse order to initial installation) or removal in parts/modular, which then influence to planning of marine spread mobilization. Therefore, the data of topsides weight shall be managed properly through weight control procedure which based on [10] as presented in **Figure 1**, the weight development graph also covering the phase of decommissioning and removal.

As the summary of paragraph described earlier, the dynamic behavior and seismic performance of the offshore structure influenced by the slenderness (stiffness) and mass of the structure. Mass's accuracy depends on the accuracy of design load input and post installation condition of the structure (i.e., straight or tilting). Hypothetically, the accuracy of mass data will correlate with the certainty of structural CoG location. Moreover, for the tripod structure that commonly slender and high natural frequency. Therefore, this research is proposed to study the sensitivity of the tripod structure on its dynamic behavior, pile foundation capacity, and structural performance during seismic condition under the variation of CoG location.

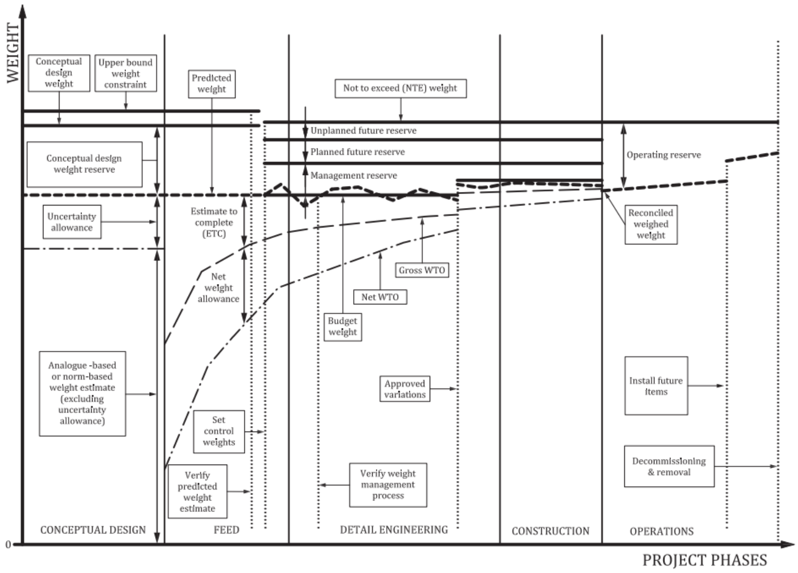

Figure. 1. Weight development graph [10]

II. Literature Review

In accordance to [13], if an existing platform experiencing any of the following conditions: increase of personnel on board (POB), additional facilities (i.e., equipment, deck extension, etc.), increased of loading on structure, decrease of air gap, and structural impairments detected scouring inspections; then assessment process should be conducted. The first two condition mentioned above will be likely to impact the shifting of structure's CoG location, however, there is no specific requirement stated in this code to include the study of CoG shifting in the offshore structure design. Meanwhile, the [10] mandate consideration of CoG shift envelope – defined in two-dimensional (2D)/planar or three-dimensional (3D) Cartesian coordinate system.

The technical specification of three oil and gas operating companies outlines the requirement related to the shift of CoG shift as follows:

a) The uncertainty for CoG shift is considered in the jacket/substructure during service condition, specifically for pile design purposes. The envelope defines the topsides CoG position within 5% of its overall dimensions and with minimum of 2mx2m. However, there is no requirement for vertical CoG shifts, nor any mandate to assess the impact of CoG uncertainty on dynamic response, nor structural behavior, or seismic performance [20].
b) The consideration to put loading in the CoG are limited to modelling of lumped masses of topside's component such as compressors, drivers, and tanks. Node at the CoG may be rigidly connected using three pin-ended struts to appropriate main deck joints to account the impact of unbalance compressor vibration forces and moments into the structural model. However, there is no requirement to assess CoG's uncertainty of entire facility and its impact to the dynamic response/behavior and seismic performance [21].
c) The requirement to consider the uncertainty of CoG of the structure is applied for pre-service condition and related to the modelling of lumped masses of topside's large masses such as compressors, drivers, and tanks. This specification highlights that from conceptual design through final platform abandonment, the accuracy of topside and substructure weights and CoG is a crucial factor in major decision-making. Recognizing the potential CoG inaccuracies, this standard acknowledges their possible impact on the response of dynamic, structural behavior, and performance of the structure during seismic condition [22].

The CoG shift envelope to be applied to the topsides for inplace analysis of the substructure. A dummy load to shift the CoG into each of the four extreme corners for the purpose to maximize the leg loads or frame load. The required shift is 1.0m between North to South direction and 1.5m east to west. However, there is no requirement to shift the CoG into vertical direction and also no explanation of impact of the uncertainty of CoG location to dynamic response/behavior and seismic performance that may result from inaccuracy structure and design load modelling [8].

The presence of an inclined seabed induces geometric irregularities in the jacket structure, primarily through variations in leg lengths and their relative positions on the seafloor. This uneven configuration results in a non-uniform mass distribution, which consequently introduces torsional effects within the structural system. For the purpose of to resist these torsional demands effectively, the implementation of stiffer tubular members in areas subjected to slope-induced asymmetry is essential. The research was conducted using finite element by ABAQUS software to analyze the structure behavior of four-legged offshore fixed jacket against four famous earthquakes. The jacket is stand on seabed with inclined 20 degrees and exposed to those different earthquake's PGA. Geometrically, the center of gravity of the structure is shifted from the original CoG with respect to global coordinate system [18].

Structural performance of an existing three-legged offshore fixed platform has examined against earthquake loading due to new equipment added onto deck that required extension. The analysis is conducted using SACS. In overall, refer to the structural analysis result, it is indicated the existing platform continues to perform without compromising structural integrity and meeting compliancy to the applicable design criteria although it susceptible to seismic loading load [23]. In this study, no sensitivity on the additional weight location to investigate the optimum design and investigate the variation of impact to the performance of the existing structure against seismic load.

As per recent study in [24], numerical model of a four-legged fixed offshore platform using OPENSEES (Open System for Earthquake Engineering Simulation) is developed to evaluate the impact of various uncertainty parameters on seismic performance. The analysis incorporated fragility and sensitivity assessments through Incremental Dynamic Analysis (IDA), with simulation efficiency enhanced using the Latin Hypercube Sampling (LHS) technique. The study considered several sources of uncertainties, including near-fault ground motions, mass, gravity load, materials (elastic modules and yield strength), and damping ratio. The results indicated that the gravity load and mass were the most significant variables, substantially affecting IDA outcomes and collapse fragility curves. A reduction in these parameters was found to improve the platform's dynamic stability and seismic resistance.

Through utilization of USFOS software to study the effects of gravity load to the dynamic behavior (P-Delta and stress softening/stiffening) and fatigue life of the offshore structures. The research model is the offshore structure with four-legged substructure. Its performance is compared for the case of with and without gravity load presence with consideration of some seastate block (different in wave significant height and wave period). It demonstrated the dynamic characteristic and fatigue performance of the structure is significantly influenced by the gravity load presence [7].

The influence of the uncertainty of seismic response parameters (i.e., plastic hinge, mass, and damping ratio) on computer modelling for the eight-legged offshore platform is investigated through sensitivity analysis on the seismic response parameters [15]. The research utilize methodology of TDA (Tornado Diagram Analysis), FOSM (First-Order Second Moment), and Non-linear dynamic analysis using SAP2000 program software, the uncertainties is studied. It has been obtained the accuracy of modelling of those seismic response parameters during design process is important to ensure the reliable performance under seismic loads. The seismic response is significantly impacted also.

A scale model experiment of the four-legged offshore jacket structure is utilized to investigate the comparison of the dynamic response and reaction at foundation with the result of analysis using FEA (Finite Element Analysis) [14]. The dimension of the scale model is obtained using Froude's Law method, meanwhile computerized model using ANSYS software as the FEA. The result demonstrated a strong agreement between the scale model and numerical analysis method result in term of the structure's dynamic response. Meanwhile the comparison of foundation reaction forces with finite element analysis results showed a reasonable match, a difference of approximately 13% was noted. The study also revealed that increase of the mass of topsides lead the reduction of structure's natural frequency and resulted in decrease of reaction forces at the foundation. In conclusion, mass distribution influenced significantly the dynamic behavior and load transfer characteristics.

As many literatures described above, the impact of variation of CoG location in any directions (including vertical direction), change of mass, and combination between these two items to the dynamic behavior and seismic performance to the offshore fixed tripod structure had not been observed specifically. Therefore, this research is proposed to study this gap through sensitivity analysis method.

Offshore Tripod Structure

The offshore tripod structure is a fixed substructure comprising three tubular legs connected by bracing system and firmly supported the topsides to the seabed using pile foundations [2]. The pile either through leg foundation type, skirt sleeve pile type, or by suction pile type.

Topsides to support the equipment and other element of facility to support the required operational or production such as vent/flare boom, telecom mast, crane, etc. The main vertical member of topsides commonly using tubular meanwhile the horizontal member to use beam shapes. The topside that supported by tripod structure might be classified based on its functionality and purposes such as wellhead platform, riser platform, flare support structure, bridge support structure, etc. Commonly it is a minimum facility considering the structural characteristic of tripod that is slender and high natural period.

Tripod structure modelled similarly to a simple pendulum. It serves as a representative case where gravitational force functions as a restoring mechanism. The system configuration (**Figure 2**) involves a point mass *m* sustained at the tip of a lightweight rigid bar (with length of *l*). Pivot action at the base of bar. The gravity load *m* acts as a destabilizing force on the system. At equilibrium, the gravitational forces *mg* is counterbalanced by the thrust along the bar OA. The corresponding dynamic equilibrium is derived by taking moment about point "O".

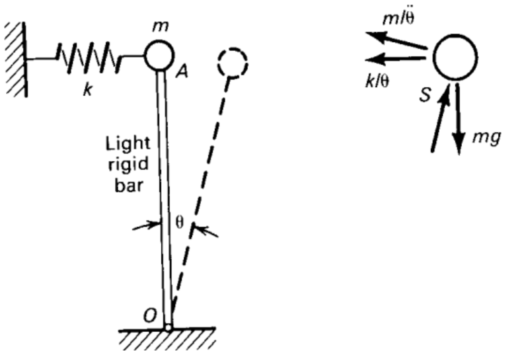

Figure. 2. Illustration of Inverted Pendulum [25]

As described in [25] the equation of motion for such dynamics system through the following non-linear equation of motion for an inverted pendulum.

$$ml^2\ddot{\theta} + kl^2\theta\ cos\theta - mglsin\ \theta = 0 \tag{1}$$

In case of the angles is relatively small ($\theta \approx 0$), the above equation changed into equation 2.2 below:

$$m\ddot{\theta} + \left(k - \frac{mg}{l}\right)\theta = 0 \tag{2}$$

This demonstrates that the gravitational load effectively reduces the effective stiffness (*k*) of the system. Furthermore, instability of the system arises when the mass *m* reaches the critical value $m = kl/g$ at which point the structure becomes unstable and undergoes buckling due to its own weight.

Dynamic Analysis

The aim of this analysis is to compute the natural periods of vibration and the associated mode shapes of the platform, and in addition, to compute the necessary dynamic amplification factor (DAF) for any suitable environmental condition. The natural period (Tn) is expressed with the following formula in [26]:

$$T_n = 2\pi\sqrt{\frac{M}{K}} \tag{3}$$

Structure system in motion dissipate energy and this damping causes the amplitude to decay from the initial value. The offshore platform is a multi-degree of freedom and the dynamic response of this structure is expressed through motion equation given below:

$$M\ddot{x} + C\dot{x} + Kx = f(t) \tag{4}$$

where M (mass matrix), C (damping matrix), K (matrix of stiffness), with $\ddot{x}$, $\dot{x}$, and x as the vector of acceleration, velocity, and displacement, respectively.

DYNPAC program module of SACS software suites is employed in the dynamic analysis to determine the structure's dynamic characteristics, natural periods (eigenvalues), natural mode shapes (eigenvectors), modal internal forces, and associated stress vectors/distribution. The module provides the essential mode shapes and corresponding modal masses required for subsequent modal dynamic analysis. All mass properties and all stiffness associated with the reduced/constrained degrees of freedom (DoF) is accounted by the eigenvalue's extraction process.

The consistent mass approach is adopted to generate the mass matrix. This method represents the kinetic energy of the distorted element by the element joint velocities, as represented by the velocities of all degrees of freedom at the joint. The kinetic energy is defined as:

$$KE = \frac{1}{2}\int M\delta^2 dx \tag{5}$$

The overall stiffness and mass matrices that correspond to all DoF of the model are assembled by the program. Subsequently, Guyan reduction technique will be used to simplify both matrices to include master or retained DoF only resulting in.

$$KE = \frac{1}{2}\{\dot{\delta}'_m\}[M'_{mm}]\{\dot{\delta}_m\} \tag{6}$$

In this dynamic analysis, the water depth is considered at mean sea level (MSL). Consequently, all members situated below this level will possess an added mass value. The legs will be flooded with water up to the top of leg, whereas all other members will be treated as non-flooded. The mass model consists of the mass of the following: structural, added water, contained, and marine growth.

The added mass is defined as the mass of water that expected to move together with the structural member during deflection. For tubular component, a mass value numerically equivalent to the mass of water displaced by the submerged member is utilized including marine growth (whenever is relevant). The contained mass represents the water that is enclosed or held by the submerged members. Eigenvalues and eigenvectors will be produced by the matrix decomposition and solution from which the natural periods are derived and the mode shapes of the structure illustrated.

The ratio of structural natural period (Tn) to wave period (T) then computed for the relevant waves. Subsequently, with considering damping factor (ζ) taken as 5% (for steel structures submerged in water) the DAF of the structure can be computed using the relationship of single DOF as follows.

$$DAF = \frac{1}{\sqrt{\left[\left(1-\frac{T_n^2}{T^2}\right)^2+\left(\frac{2\zeta T_n^2}{T^2}\right)^2\right]}} \quad (7)$$

The dynamic analysis is conducted without explicitly modeling the pile soil interaction system (non-linear) of the jacket foundation. This non-linear behavior is approximated using foundation linearization through superelement module provided by the software. Each pile foundation is represented by a 6 x 6 stiffness matrix to simulate the soil's linear elastic response.

Inplace Analysis

This analysis is aimed to verify the capacity to the designed pile foundation to withstand the design load/parameters including environmental condition of operating with minimum water depth under the variation of structural CoG location. Therefore, the non-linear pile soil interaction module included in this analysis.

The common steps of inplace analysis comprises of general steps as follows:

a) Geometry Simulation. The geometry of the platform is simulated in accordance with the design drawings and design basis. A three-dimensional (3D) rigid spaced frame model will be developed, in which all structural modeled members contribute to the overall stiffness of the systems. All the primary structural members in the jacket and deck shall be modelled. Member lengths are assumed from node to node. The corrosion allowance shall be modelled for structure in the splash zone area for strength analysis purposes. All appurtenances such as boat landing shall also be modelled including its geometry, marine growth, drag and inertia coefficients, adjusted accordingly to obtain correct wave loading. The appurtenances contribute to wave loading however do not considered in the stiffness analysis of the structure. The weight density of modelled topside's structure members is nullified.

b) CoG Shift Simulation. Simplified approach utilized in this study to avoid complexity of dummy load to shift the CoG to certain location and numerical error during dynamic analysis. Four set of dummy members (density and stiffness are nullified) is added in to the model. These members to connect the dummy joint of CoG shift location to four primaries joint at deck legs for representation of load transfer but not providing additional stiffness to entire structure.

c) Load Simulation. A simulation of the gravity loads and generate results of the analysis. Gravity loads for modelled members are automatically generated by SACS module while other non-generated dead loads are input as separate basic loads. The basic load will be considered is dead weight (selfweight of the modelled structure and other external dead load), open area live load, and environmental load. The total operating load of topsides including selfweight of topsides modelled member is applied in the dummy joint simulating each CoG shift scenario location. For large equipment, set of dummy members are modelled and load applied in the appropriate CoG of the equipment based on the manufacturer data. This load will then be transferred to primary member of topsides.

d) Foundation Simulation. This step involves modeling the foundation piles and surrounding soil based on the specifications provided in the geotechnical report. Piles located above sea level are modeled down to the mudline, aligned with the jacket legs to reflect their placement within the leg structures. No grout in the annulus between piles and legs. Shear elements (called as wishbone) are modelled to transfer shear load only between the jacket legs and the piles. Foundation below the mud line is modelled by defining the pile section properties and penetration in pile soil non-linear analysis input file. On the nonlinear foundation, the pile foundation is modelled as beam-column element. The data of pile heads geometry, submerged unit weight, and pile batter shall also be indicated in the input file. A plugged condition of the pile tip is considered to account the end bearing effects. The pile foundation behavior is characterized using pile member stiffness in conjunction with the site-specific soil response curves (P-Y, T-Z and Q-Z) as provided in the geotechnical investigation report. Comprehensive pile soil interaction analysis is conducted using PSI module of SACS software and also incorporating the effect of scour and potential soil degradation within seabed elevation (as specified in the design basis). The effect of scour is applied by nullify the P-Y value up to the specified scour depth.

e) Modal Analysis. This step to determine the dynamic properties of the platform and produce the modal masses, modal frequencies, and mode shapes.

f) Wave Response Analysis. Conduct a time series wave response analysis in conjunction with the pile soil interaction effects.

g) Combine Gravity and Wave Response. Integrate the outcome of the gravity static analysis resulting from gravity loads with wave response analysis and applying the calculated DAF as the load factor on the wave load which then combined with applicable design loads in the load combination.

h) Post processing of results. Execute post-processing of the results to derive the member forces, joint deflections, pile loads, member unity stress ratio check, etc. The summary of analysis provided in the post-processing module will subsequently be compared against the acceptance criteria.

Step a and b as above play important role to ensure the accuracy of CoG of the platform system that becomes aim of this research.

Seismic Analysis

The seismic performance of the tripod structure evaluated through spectral response based equivalent static (ESM) analysis. The natural period (T) of structure during seismic condition taken from the modal analysis using SACS software by considering water depth of mean sea level (MSL). The spectral acceleration (Sa) at natural period (Sa(T)) is obtained through interpolation with site-specific spectrum of RIE level with 5% critical damping.

The modal base shear for each direction (V_{Xi}, V_{Yi}, V_{Zi}) is calculated based on the equation below.

$$V_{Xi} = S_a(T).M_{eff-Xi} \quad (8)$$

$$V_{Yi} = S_a(T).M_{eff-Yi} \quad (9)$$

$$V_{Zi} = 0.5 \times S_a(T).M_{eff-Zi} \quad (10)$$

Vertical spectral acceleration is half of corresponding horizontal action for seismic zone 0, 1, and 2 [27].

The peak responses of all significant modes then combined to get the total peak seismic response for each direction using Complete Quadratic Combined (CQC) to capture the correlation between the closely spaced modes (i and j) of vibration based on [27] and [13]. If the structure is highly asymmetric (such as three-legged structure), or, when obtained different intensities between two components of inputs, or if two or more modes of the structure's modal response are closely spaced (i.e., frequency ratio < 1.10) [28].

$$V_X = \sqrt{\textstyle\sum_{i=1}^{n}\sum_{j=1}^{n}\rho_{ij}V_{Xi}V_{Xj}} \quad (11)$$

$$V_Y = \sqrt{\textstyle\sum_{i=1}^{n}\sum_{j=1}^{n}\rho_{ij}V_{Yi}V_{Xj}} \quad (12)$$

$$V_Z = \sqrt{\textstyle\sum_{i=1}^{n}\sum_{j=1}^{n}\rho_{ij}V_{Zi}V_{Zj}} \quad (13)$$

where, the ρ_{ij} is defined as the modal correlation coefficient between modes i and j. This element depends on the frequency spacing between modes and damping.

The cross-modal correlation coefficient for constant modal damping is calculated using the following equation [28]:

$$\rho_{ij} = \frac{8\xi^2(1+r)r^{1.5}}{(1-r^2)^2+4\xi^2 r(1+r)^2} \quad (14)$$

where: ratio of natural frequencies, $r = \omega_j/\omega_i$ and ω_j, ω_i is the natural frequencies at mode i and j modal and the damping ratio (ξ) that typically taken as 5% for structure. The modal damping ratio influences how the modes interact. As observed from the equation above, if the damping increase, the modal correlation coefficient will decrease (i.e., reducing the modal interaction). Consequently, lower total dynamic response in CQC result.

The final seismic forces (V_s) resulted from the combination of the total peak directional responses of three orthogonal directions (two horizontal and one vertical) to obtain the final seismic forces. The Vs then applied at the CoG of topsides combined with gravity load for structural analysis with PSI input. The overturning moment (MOTS) for seismic condition determined by deploying the simplified method equation below. The overturning moment is the product of base shear and effective height (H_{eff}), in this case is the vertical distance from the mudline to particular CoG position of each case.

$$M_{OTS} = V_s.H_{eff} \quad (15)$$

Further process of seismic analysis is observing the adequacy of parameters related to pile foundation capacity (axial compression load, pile member stress utilization ratio, and pile bearing capacity factor of safety) is executed through SACS with PSI and post-processing module which subsequently the result be compared against the acceptance criteria.

III. METHODOLOGY

A. General

This section describes the general workflow of this research, assessment phases, analysis model, design data, sensitivity method, and acceptance criteria. General workflow from the early stage up to the completion of this research is depicted in the flowchart shown in **Figure 3**.

The assessment is conducted in three major phases as follows:

A. Phase-1: Topsides operating load and origin CoG generation.
   a) Modify the origin analysis model taken from the primary data to suit with this study. The modelled topsides members set as normal steel density to account it selfweight into the combined gravity load.
   b) Run analysis static inplace without pile-soil interaction using SACS program software.
   c) Extract the total combined operating load of topsides and CoG of the topsides that automatically generated by software.

B. Phase-2: Perform structural analyses
   a) Modify the model (nullify the steel density of the topsides model, skip the topsides member from design, introduce the dummy member for CoG modelling).
   b) Develop 75 sets of analysis models for each inplace and seismic condition.
   c) Observe the behavior of mode shapes and the mass participation for the first 30 mode shapes.
   d) Extract the analysis result (natural period from modal analysis, maximum overturning moment, maximum base shear, pilehead lateral displacement, pile axial compression load, pile member UC stress ratio, and pile bearing capacity Factor of Safety – FoS –).

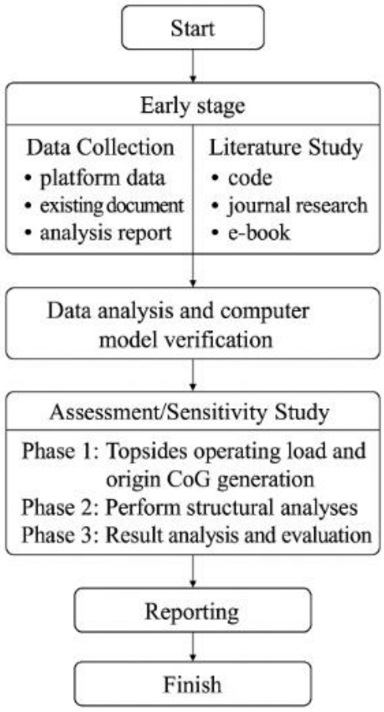

Figure. 3. Research Methodology Flowchart

C. Phase-3: Perform assessment
  a) Analyze, assess, and compare all the results.
  b) Develop charts/graphs and table as necessary to visual the result.
  c) Conclude the sensitivity and provide recommendation.

B. Analysis Model

A shallow-water tripod offshore structure in Mahakam Delta of Indonesia is utilized in this study. The platform consists of 3-legged substructure model as outlined in **Table 1** with conventional pile through leg and topsides with minimum facility wellhead type. The topside furnished with an inclined vent boom and a vertical telecom mast. The 3D model view of the tripod structure is as shown in Figure 4.

C. Metocean Data

Platform's water depth is defined as 47.30m that means the mudline elevation is -47.30m from chart datum (CD) [29]. The metocean data for this tripod used for this study with respect to 1-year operating condition is outlined in Table 2.

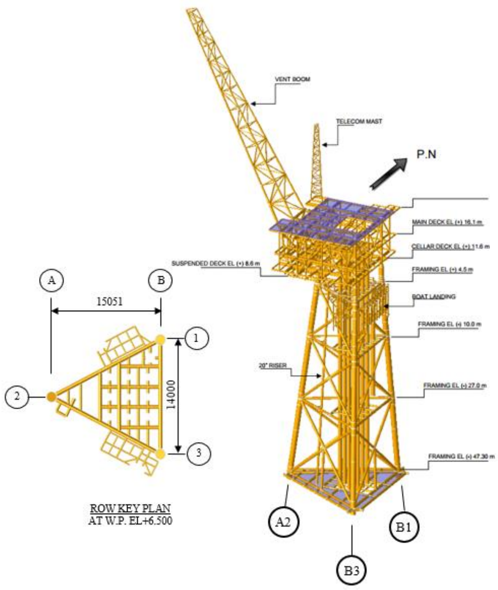

Figure. 4. 3-D View of The Platform

TABLE 1.
TRIPOD STRUCTURE MODEL

Water depth	47.30 m
Topside work point elevation	EL (+) 6.5m
Horizontal framing elevation	EL (+) 4.5m; EL (-) 10.0m; EL (-) 27.0m; EL (-) 47.3m
Main Pile	3 number of 48” dia. Piles
Conductor	6 number of 36” dia. Conductor pipes 3 number of 30” dia. Conductor pipes
Riser	1 no of 20” dia. Riser
Boat landing	1 no. at Row B
Top of steel (TOS) elevation of the deck	TOS EL. (+) 21.90m (Upper Deck) TOS EL. (+) 16.10m (Main Deck) TOS EL. (+) 11.60m (Cellar Deck) TOS EL. (+) 8.60m (Suspended Deck)

The wave loading is generated from Stokes 5th order theory based on the apparent period. It is combined with current, and wind load (suitable for substructure verification) is generated by the software to attack the platform in 12 directions. The hydrodynamic factor of [13] covering wave kinematic factor (0.95), surface roughness, and current blockage factor of 0.90 for all direction (end on, diagonal, and broadside) is used. No blockage factor for conductor is considered.

The surface roughness factor Cd (0.65 and 1.05) and Cm (1.60 and 1.20) for smooth member and rough member respectively. The smooth member considered above +2m of LAT (above marine growth zone) while the rough member counted below +2m of LAT (marine growth zone). The marine growth density for this water area is 1400 kg/m^3 in air.

TABLE 2.
SITE SPECIFIC METOCEAN DATA

Description	Value
Wave Height, H_{max} (m)	3.30
Wave Period, T_{ass} (s)	6.90
Associated Current (m/s)	
• Surface	0.90
• Mid-depth	0.65
• Mudline	0.55
Wind Speed (m/s) at 10m above LAT	
Squall 1 minute	16.10
Water Level (m)	
• Chart Datum (CD)	47.30
• Mean Sea Level (MSL)	1.10
• Lowest Astronomical Tide (LAT)	0.48
Marine Growth thickness on radius (mm)	
• Above +2.0m	0
• Between +2.0 to -15.0m	100
• Between -15.0 to -30.0m	50
• Between -30.0 to -47.3m	25

The splash zone to be considered in the analyses is from elevation (-) 2.52m until (+) 6.38m. Both elevations are with respect to chart datum elevation. Within this splash zone, the corrosion allowance (CA) to be considered on the analysis. The CA is 6mm on the thickness of jacket member and 5mm of CA to be applied on both faces of caisson. No CA to be considered for diagonal members of the jacket at the level of +4.50m.

D. Geotechnical Data

The axial T-Z, end bearing Q-Z, and lateral cyclic P-Y soil curve are taken from the Geotechnical Investigation Work report of the platform. An intact soil is considered for pile A2 whereas degraded soil is considered for piles B1 and B3 to account the effect of Jack Up Spud Can disturbance. In this study, the data of 0.5 diameter offset of the spud can is considered and scour within seabed elevation is assumed as 1.2m as per design basis for this platform.

Pile capacity assessment utilized in the geotechnical report as specified [30] (API method 2). On the non-linear foundation analysis, the P-Y data for cyclic loading is utilized. The graph is shown in **Figure 5**.

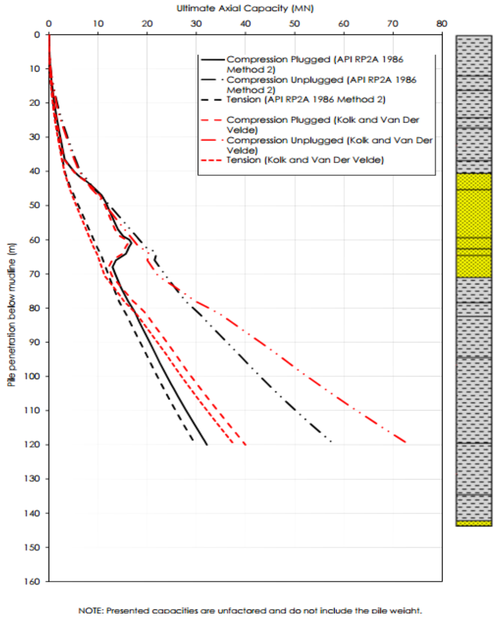

Figure. 5. Ultimate Axial Pile Capacity – Driven Pile

E. Seismology Data

The site-specific effective ground acceleration 0.120g for rare intense earthquake (RIE) is used for basis to assess and qualify the sensitivity of seismic performance of this tripod structure under CoG shift variation with critical damping of 5%. The response spectra for RIE level as presented in **Figure 6**.

F. Design Load

The design load considered covering the gravity load and environmental load. The gravity load consists of dead load of selfweight of the modelled structure, un-modelled structure items, equipment load (empty and operating), piping load (empty and operating), and live load. The corrosion allowance of the jacket member in the splash zone is not considered in the selfweight generation. For massive or large equipment, the load is applied on its CoG position that inputted based on information from the specific vendor. Meanwhile the live load applied on the open areas that not occupied by the equipment permanently.

External loading imposed to the structure arises from sea environmental factors (wave action, current, marine growth, buoyancy effects, and wind) is considered as environmental design load. In the dynamic analysis of the offshore structures, the effect of wave, impact, and ground motion, known as hydrodynamic forces, is taken into consideration.

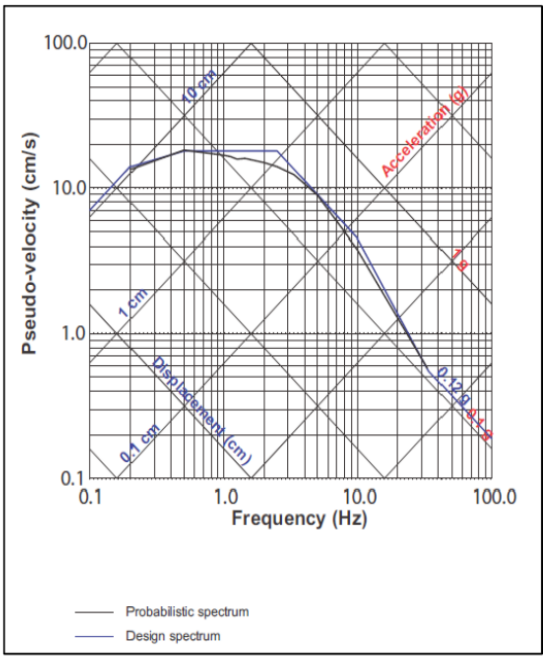

Figure. 6. Plot of Site-Specific Response Spectra for RIE Level

This force counted per unit length acting along the axis of each submerged jacket member. The environmental load generated automatically by the analysis software program considering the input of seastate parameters (as per site specific metocean data and generic as per [13]) using the Morison equation:

$$F_{wave} = F_D + F_I = C_D \frac{w}{2g} AU|U| + C_m \frac{w}{g} V \frac{\delta U}{\delta t} \quad (15)$$

Even though, the wind loads are inherently dynamic, certain structures may exhibit a predominantly static response. In the offshore fixed structure located in relatively shallow water, winds typically contribute less than 10% to the overall global load [13]. The wind drag force acting at a given Z elevation on the structure is determined using the following equation:

$$F_{wind} = \left(\frac{\rho}{2}\right) u^2 C_s A \quad (16)$$

The shape coefficient (*Cs*) is evaluated based on wind incidence at perpendicular angles relative to the projected areas of the structural components. The recommended value of *Cs* is referred to [13].

The vector summation of local drags and inertia forces induced by wave and current loading, dynamic amplification effects associated with wave and current loading, and wind-induced forces will be computed by the software to obtain the total base shear and overturning moment at the mudline level resulting from the environmental loads.

G. *Sensitivity Method*

The sensitivity of the structure examined through cases that mapped into four quadrants as presented in **Table 3**. Artificial structural CoG shift envelopes due to topsides operating weight including case of weight increment. The CoG envelope, in accordance with [10], may be in two-dimensional (2D) or three-dimensional (3D) depending on the facility or assembly.

TABLE 3.
FOUR QUADRANTS MAP OF SENSITIVITY ANALYSIS CONCEPT

II CoG is shifted but no additional/variation of weight	**I** Origin case (no CoG shifting and no increment of weight)
III CoG in origin location but weight increased	**IV** CoG is shifted and weight increased

The CoG shift envelope, illustrated in **Figure 7**, is computed based on the concept of as follows:

a) X $\pm$ 1.00m; Y $\pm$ 1.00m (corner A1, B1, C1, D1)
b) X $\pm$ 2.00m; Y $\pm$ 2.00m (corner A2, B2, C,2, D2)
c) X $\pm$ 1.00m; Y $\pm$ 1.00m; Z $\pm$ 1.00m (corner A3, B3, C3, D3, A4, B4, C4, D4)
d) X $\pm$ 2.00m; Y $\pm$ 2.00m; Z $\pm$ 1.00m (corner A5, B5, C5, D5, A6, B6, C6, D6)

As addition to CoG shift case location, the load increment is set as 10% and 20% with respect to the total of original topside operating weight to enrich the sensitivity cases. By considering the concept as above, the total number of cases for this sensitivity study up to total of 75 cases.

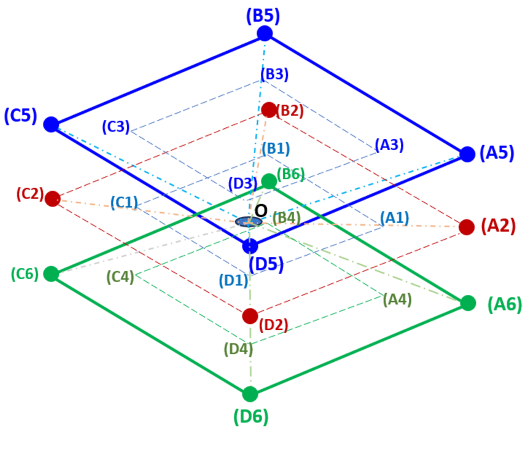

Figure. 7. CoG Shift Concept Illustration

The factual definition of quadrant mapping is detailed as follows:

a) Quadrant I (Origin Case, No CoG shift and no increment)
This is the baseline case and an ideal condition. That is no deviation between the estimated weight cum CoG with actual condition (post weighing at fabrication and no change on the topsides along the facility operation).
b) Quadrant II (CoG shifted, but no weight increment)
The weight is well managed and no variation to weighing result, however, the arrangement of facilities after construction differs from the design (i.e., modelling of top of structural steel of primary beam, piping assembly, equipment, tilting after installation, subsea subsidence, etc.). This condition might be occurred due to change during design or error in construction/installation.
c) Quadrant III (CoG in origin location but weight increased)
The location or arrangement of facilities is as per design, however the actual weight of equipment/pumps/major turbine generator, piping assembly, etc., post fabrication or along facility operation found heavier than the design assumption (error in assumption).
d) Quadrant IV (CoG is shifted and weight increased)
Modifications to the facility—such as the installation of new equipment requiring deck extension, structural strengthening, elevating the topsides that requiring additional of new legs, and integration of new topside modules—led to a deliberate consequential shift of CoG associated with increase in overall weight (intentional weight increase).

H. Acceptance Criteria

The study is focusing on the dynamic behavior of the tripod structure and seismic performance. No redesign option to be observed. Hence the following criteria is used for basis of recommendation:

a) Natural period to remain below 3.5second to avoid fatigue concern.
b) Pile member UC stress ratio shall be less than 1.0.
c) Pile bearing capacity FoS shall be less than 2.0 for inplace operating condition and 1.0 for seismic condition.

IV. RESULT AND DISCUSSION

Refer to Phase 1, the total weight of design loading combination for topsides operating condition is 12788 kN with origin CoG coordinate (X-Y-Z) determined as -6.83, 0.20, 18.27 (meter) with respect to the global coordinate of the platform used in the SACS analysis model. The plot of origin CoG location on the platform is presented in **Figure 8**.

Series of structural analyses (dynamic, inplace with pile-soil interaction) and spectral response based equivalent static analysis and comparative analysis methods for seismic condition had been performed. The dynamic behavior parameters (natural periods, overturning moment, base shear, pilehead lateral displacement, axial compression load distribution), mode shapes, and performance (focusing on pile member UC stress ratio) are extracted and analyzed.

Observed the from the mode shape of tripod structure as follows:

a) Normal deformation pattern in translational direction (X and Y) and torsional (about Z) is shown for each case especially for the first three mode shape.
b) The minimum of 90% structural mass participation in each horizontal (X and Y) direction for modal analysis as required by codes and standard practices ([27], [29], [31]) is achieved for all cases.
c) Obtained a consistency of total added mass for all CoG shift cases regime.

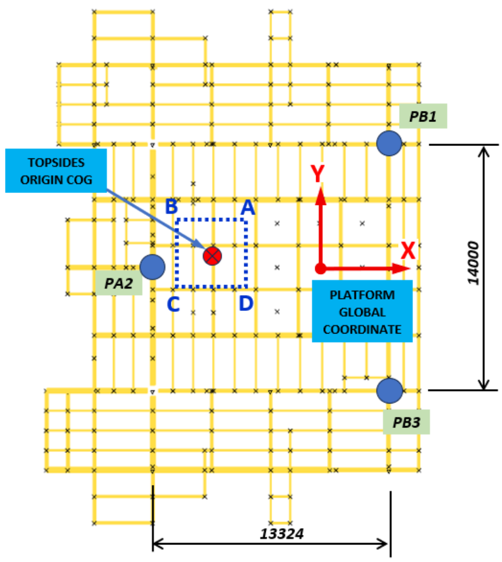

Figure. 8. CoG Shift Envelop on Platform

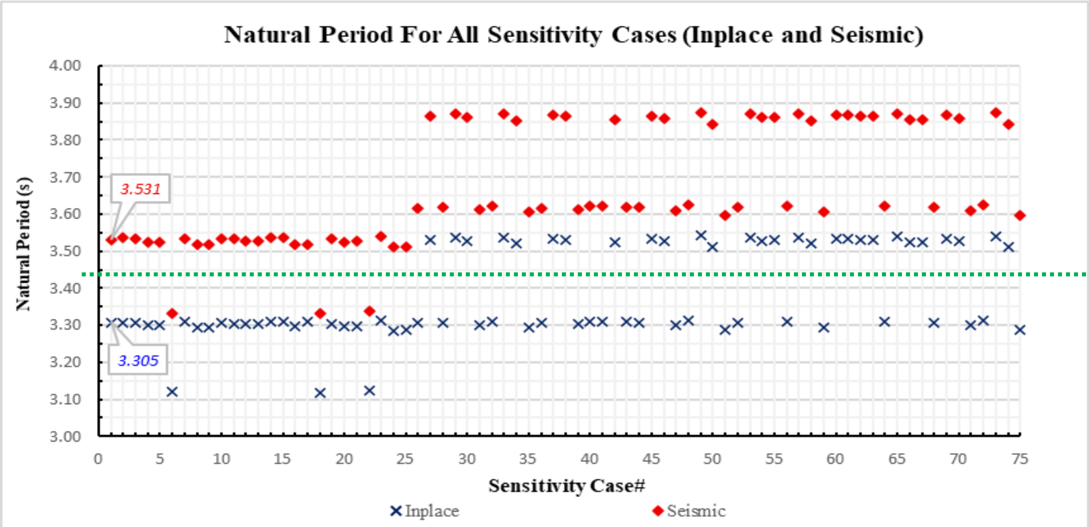

Figure. 9. Natural Periods of Structure for All Cases (Inplace and Seismic)

A. Natural Periods

The natural period of tripod based on dynamic inplace operating and seismic condition at origin topsides CoG determined as 3.305s and 3.443s respectively. The range of natural period for all cases of CoG shift resulted from dynamic analysis at inplace condition is 3.116~3.542 second while range of 3.331~3.876 second for seismic condition. The trend of natural period's range is presented in **Figure 9**.

As observed from the graph and analysis result the following sensitivity:

1) The natural period of the observed offshore tripod structure is determined varies for each CoG shift case compared to the baseline case (CoG at origin location). It means the variation of CoG location influenced the dynamic response of offshore tripod structure regardless the magnitude of variation.
2) The natural period of structure becomes longer than the threshold (3.5second) for inplace conditions is majority occurred starting from case number 27.
3) Anomalies from the above point is the sharp drop for cases number 6, 18, and 22. At these cases, the natural period is significantly shorter than the baseline case (period shortening range of 5.49% to 5.72% for inplace and 5.42% to 5.65% for seismic condition). At these cases, the CoG are shifted away toward "Corner D" (further nearest to leg PB3 and shifted away from the vent boom and telecom mast)
4) The natural period of the tripod structure for case CoG shifting within the same elevation (2D planar) and Z direction $\pm$1m is marginally longer than origin position (no shift) without weight increment. It is about 0.23% for inplace operating condition and 0.30% for seismic condition. The natural period is still very marginally longer than the origin although the weight increased 10% if the CoG of topsides during operating condition is remained not shifted
5) Introducing weight increase of 20%, mostly the natural period of both inplace and seismic condition become longer than baseline case up to 7.15% for inplace and 9.75% for seismic condition. As the exception, there are three cases of shortening occurred at inplace condition for CoG shifted 2m (2D and 3D) toward Corner D. In these cases, the natural period is 0.20% to 0.57% shorter than baseline case. This corner geometrically located away from vent boom and telecom mast.

B. Maximum Overturning Moment and Base Shear

The scattered value of maximum overturning moment and base shear during both inplace and seismic condition is presented in **Figure 10**, **Figure 11**, **Figure 12**, and **Figure 13**. At inplace condition, the maximum overturning moment is 83.67 MN-m and for seismic condition is determined as 32.09 MN-m. The range of scattered value around 68.95 MN-m to peak value of 133.46 MN-m for inplace condition, although majority of the cases are relatively close to or slightly above/below the baseline. Meanwhile, lower scattered value with range of 27.46 MN-m to peak value of 34.70 MN-m for seismic condition. Most cases in seismic condition are closely clustered around the baseline case, with fewer outliers and smaller overall deviation compared to inplace condition.

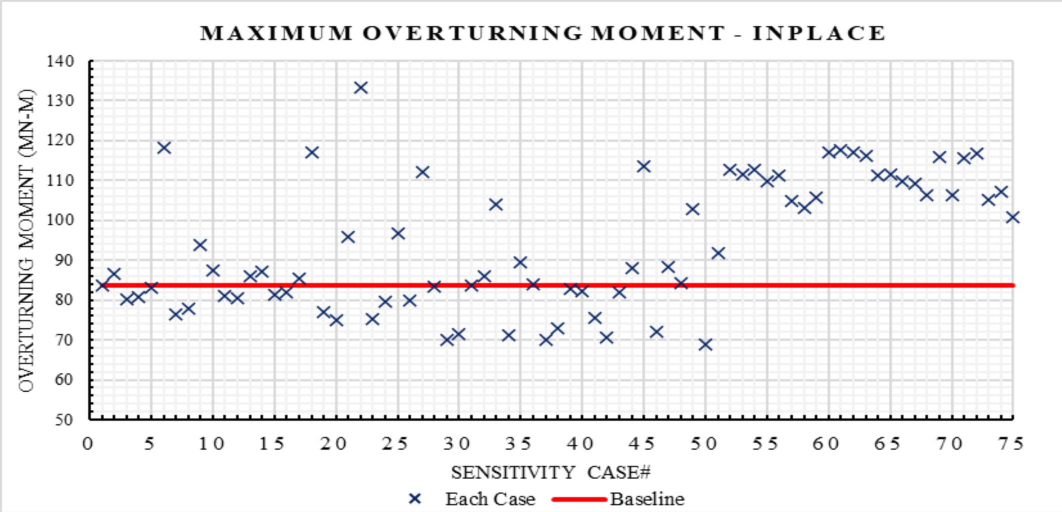

Figure. 10. Maximum Overturning for All Cases (Inplace)

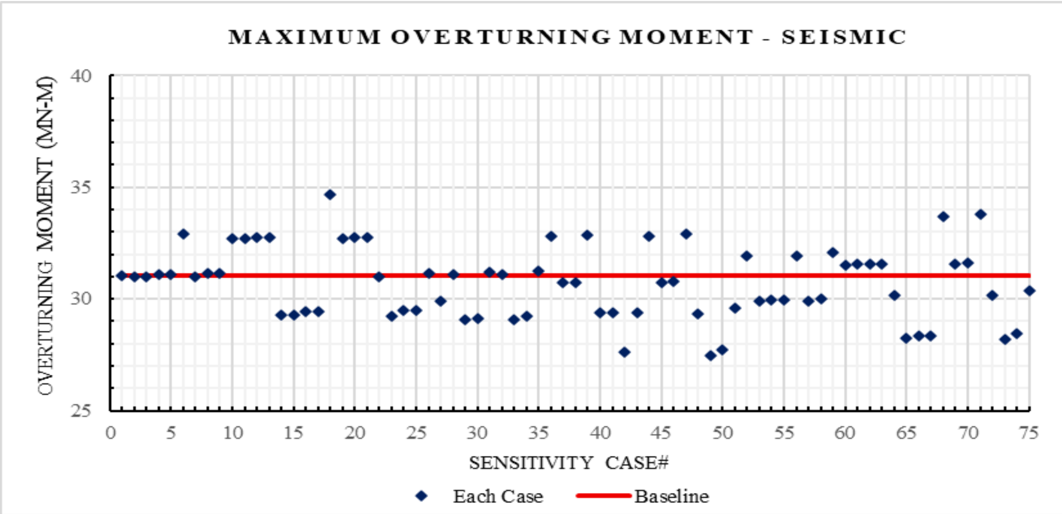

Figure. 11. Maximum Overturning for All Cases (Seismic)

The maximum base shear at baseline case during inplace condition and seismic condition is determined as 2049.31 kN and 1756.37 kN respectively. The base shear during inplace condition is much scattered above the baseline (2049.31 kN) with the peak value of 3192.06 kN. Meanwhile for seismic condition, the baseline is 1756.37 kN with peak value of 1800.50 kN. Similar to overturning moment, the base shear for seismic condition is much more clustered around the baseline compared to the inplace condition.

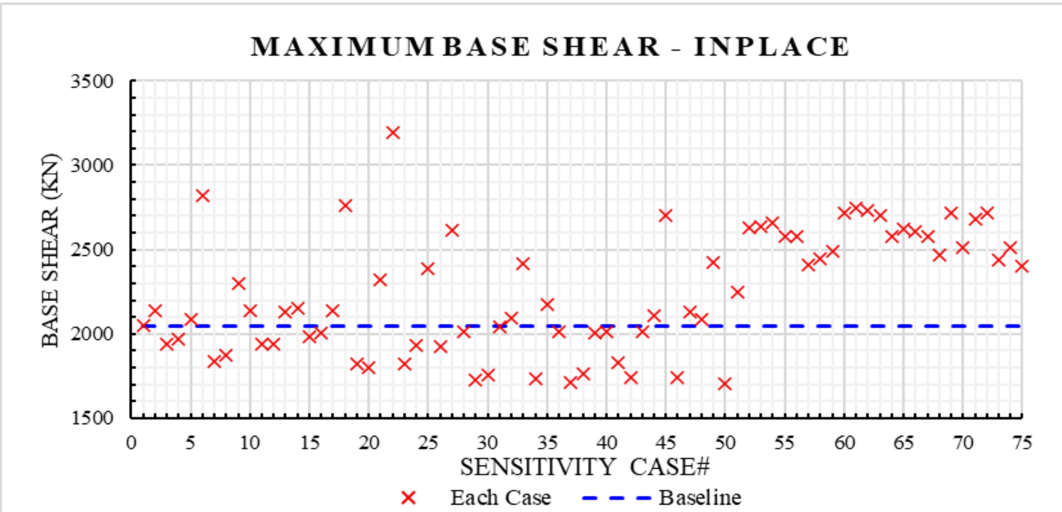

Figure. 12. Maximum Base Shear for All Cases (Inplace)

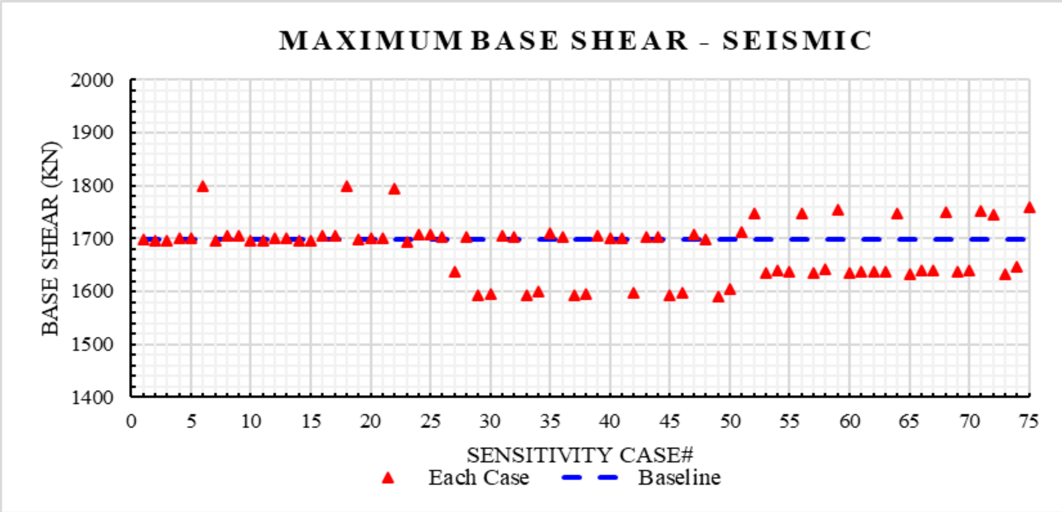

Figure. 13. Maximum Base Shear for All Cases (Seismic)

The sensitivity observation for both parameters are as follows:

1) CoG shift variation is sensitively influenced the magnitude of overturning moment and base shear for inplace condition. For seismic, the sensitivity of overturning moment slightly lower than inplace condition. While for base shear, the sensitivity in seismic condition is low to moderate sensitive.
2) The most sensitive case of overturning moment to baseline is occurred due to case 22 (shifting 3m downward to corner A6) without involvement of weight increment. For shifting with weight increment, the sensitivity determined as slightly more moderate than the above case.
3) For base shear, wide range of variation observed and the CoG shift 3D-Downward toward corner A6 without weight increment is notably significantly elevating base shear during inplace condition. However, for seismic condition, the sensitivity for base shear is significantly lower with narrow range of variation that could benefit in seismic structural performance.

C. Pilehead Lateral Displacement

The pilehead lateral displacement value for both inplace and seismic condition for all 75 sensitivity cases is presented **Figure 14** and **Figure 15**. In inplace condition, the baseline value of pilehead lateral displacement is determined as 14.00cm for PA2, 14.24cm for PB1 and 13.08cm for PB3. The baseline of pilehead lateral for pile PA2 is 8.93cm, PB1 is 9.23cm, and 9.01cm for PB3 for seismic condition.

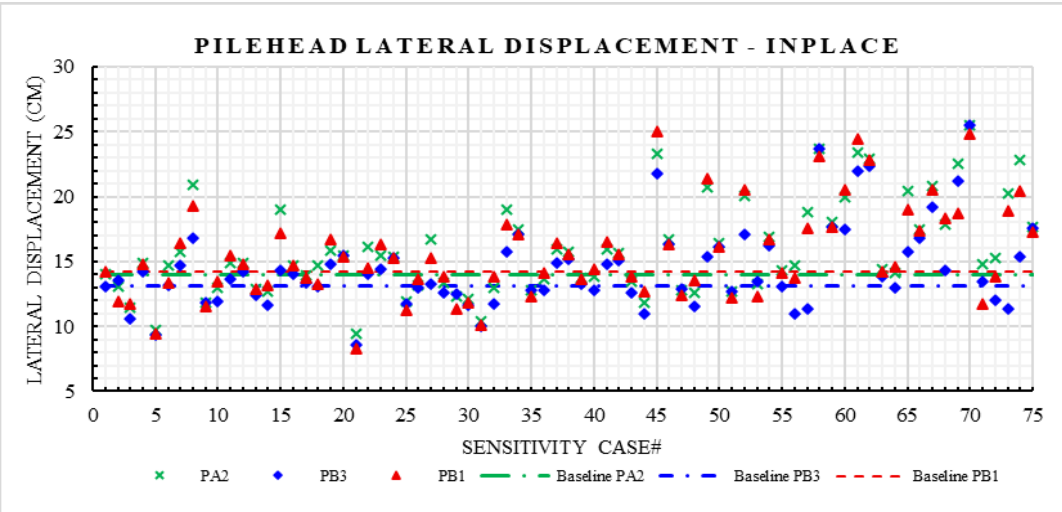

Figure. 14. Pilehead Lateral Displacement for All Cases (Inplace)

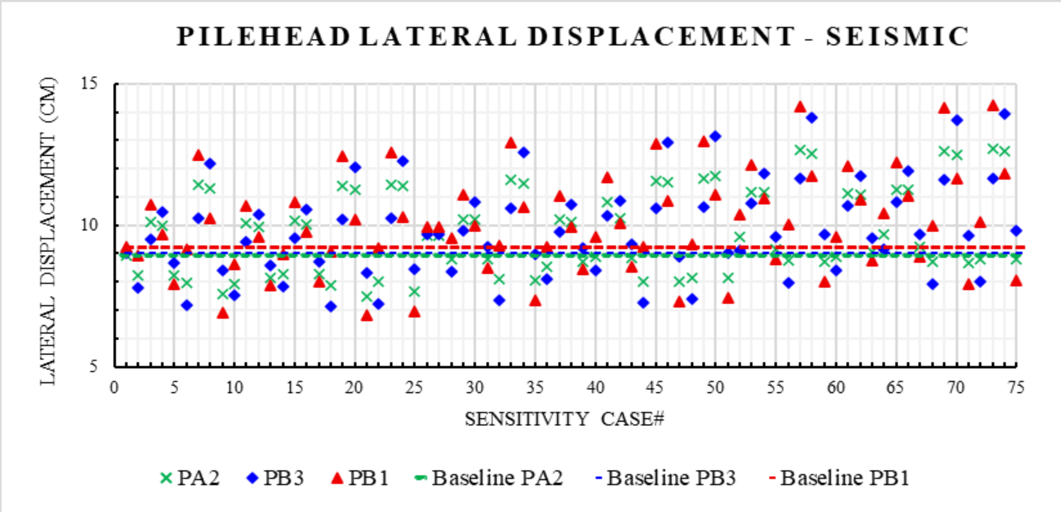

Figure. 15. Pilehead Lateral Displacement for All Cases (Seismic)

The range of lateral displacement generated from the analysis is obtained within the range of 8.33cm~25.48cm due to inplace and 6.83cm~14.26cm for seismic condition. The largest displacement occurred at pilehead PA2 and PB3 during inplace condition due to case of CoG shift to corner C5 associated with weight increment 20%, while for seismic condition is occurred at pilehead PB1 due to case of CoG shift to corner B6 with combination of 20% weight increment. The maximum displacement for both condition is still lower than the threshold of acceptance criteria for this research.

The sensitivity analysis determined as follows:

1) Even though the magnitude of maximum lateral displacement at all cases for both inplace and seismic conditions is still within the acceptance criteria for this research, however, all pilehead ID shows highly sensitive to CoG shift variation.
2) For inplace condition, pilehead PB3 obtained to have most sensitive compared to PA2 and PB1. The displacement at PB3 reach 94.80% to the baseline case value. The sensitivity for PB3 occurred mostly due to shift to corner C in combination with increment of weight.
3) For seismic condition, pilehead PB1 and PB3 is in highest sensitivity compared to PA2 mostly due to shift case toward corner B and C.

D. *Axial Compression Load on Pilehead*

The chart of axial compression load magnitude on pilehead for inplace and seismic condition due to each sensitivity case are presented in as plotted in **Figure 16** and **Figure 17**. From both charts, observed for both inplace and seismic condition, the overall axial compression load distributed dominantly to pile PA2 (14215 kN at baseline for inplace condition and 12436.8 kN for seismic condition). This due to the CoG at origin case is very near to topsides leg that directly connected to pile PA2.

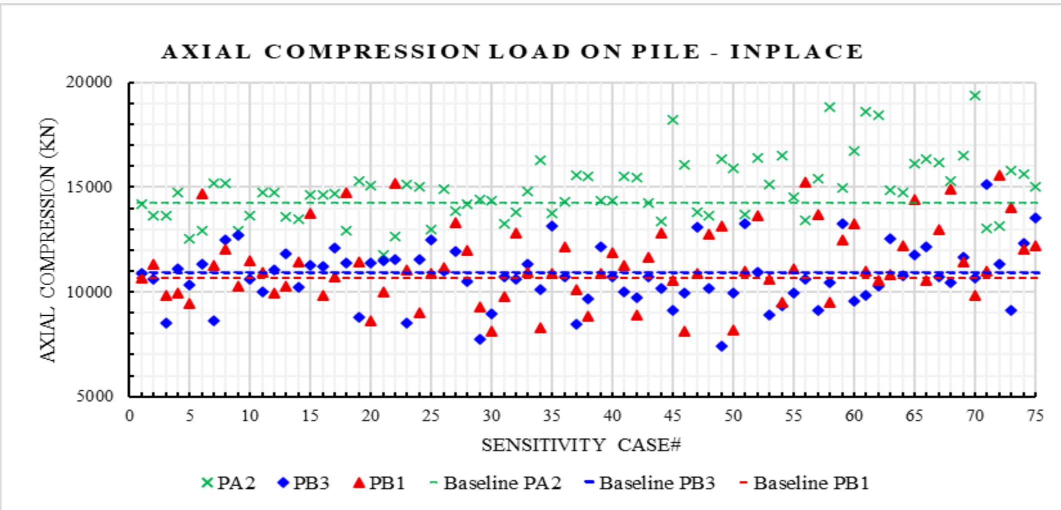

Figure. 16. Axial Compression Load on Pilehead for All Cases (Inplace)

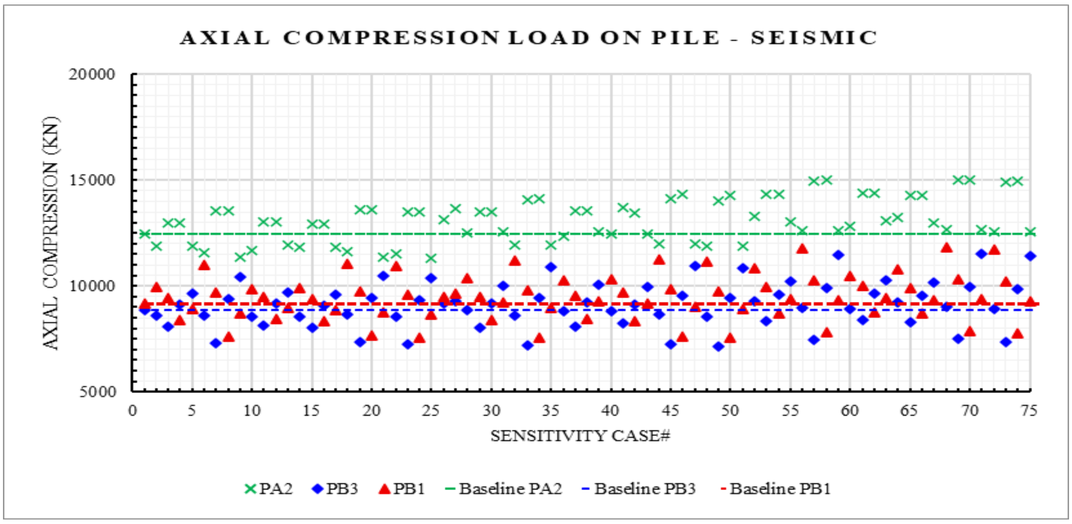

Figure. 17. Axial Compression Load on Pilehead for All Cases (Seismic)

The predominant of axial load compression load is toward pile PA2 for almost of the cases (15299.7 kN without increment of weight, 18234.2 kN for case of 10% weight increased, and 19361 kN for the 20% weight increase). However, the most variance is occurred at pile PB1. It also shows wide variation of axial compression due to CoG shift cases compared to the CoG on origin location while at seismic condition the structure performance is more centralized and rebalanced.

The sensitivity analysis with respect to axial compression load on pile determined as follows:

1) Distribution of axial compression load on each pilehead also changed along with sensitivity case in this research regardless the magnitude of value from its baseline.
2) The pilehead PB1 leads the sensitivity among other, although the heaviest axial compression load occurred at pilehead PA2. For inplace condition, it is found increased about 42.39% and 46.11% increased from its baseline value for inplace condition due to case of without weight increment and 20% increment. While, for CoG shift associated with 10% weight increment, the axial compression load on pilehead PA2 become 28.27% greater from its baseline case.

E. *Pile Bearing Capacity*

Refer to chart of analysis result for pile bearing capacity factor of safety (FoS) in Figure 18 and Figure **19** shows that the lowest FoS occurred at pilehead PA2. This finding is within the agreement with the magnitude of maximum axial compression load analysis result. Therefore, with the same pile size (OD 48”) the factor of safety (FoS) of pile bearing capacity at pile PA2 is the lowest among others.

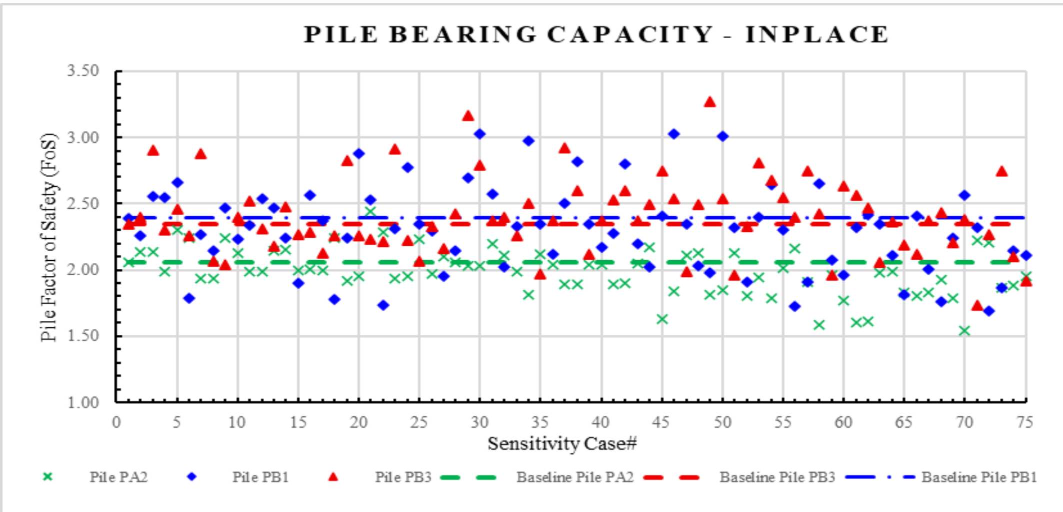

Figure. 18. Pile Bearing Capacity for All Cases (Inplace)

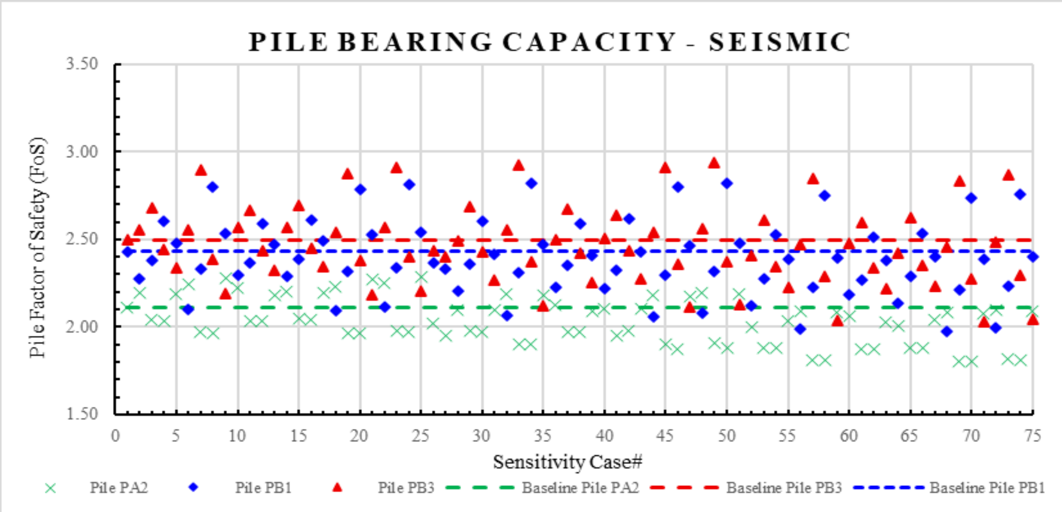

Figure. 19. Pile Bearing Capacity for All Cases (Seismic)

The acceptance limit of pile bearing capacity FoS is set as 2.0 for inplace operating condition and 1.0 for seismic RIE level. Since, the axial bearing capacity of PA2 is 32 MN (120m penetration depth) and 28.5 MN for pile PB1 and PB3 (110m penetration depth) respectively, hence, the baseline of FoS is as follows:

- for PA2 is 2.055 in inplace condition and 2.320 in seismic condition
- for PB1 is 2.390 in inplace condition and 2.726 in seismic condition
- for PB3 is 2.347 in inplace condition and 2.813 in seismic condition

Analysis of sensitivity for pile bearing capacity is as follows:

1) Obtained different value of pile bearing capacity FoS that for each case analyzed in this research. It means CoG shift influenced the performance of pile foundation.
2) The FoS for seismic condition is determined within the acceptable limit, while high vulnerability for inplace since many FoS is lower than the acceptable limit.
3) In spite of slightly less sensitive compared to both piles, the pile PA2 is determined with smallest pile bearing capacity FoS (1.544 during inplace and 1.952 during seismic) among all sensitivity cases.
4) For pile bearing capacity FoS, the CoG shift toward outer corner 3D-Downward (corner A6) influenced the highest sensitivity for inplace condition, while 3D-Upward for seismic condition.

F. Pile Member Stress UC Ratio

The magnitude of pile member stress UC ratio on pile are presented in Figure 20 and Figure 21. The member stress UC ratio during inplace condition at baseline case determined as 0.563 for pile PA2, 0.594 for pile PB1, and 0.538 for pile PB3. Meanwhile, for seismic condition, the UC ratio is 0.291 for pile PA2, 0.293 for pile PB1, and 0.287 for pile PB3.

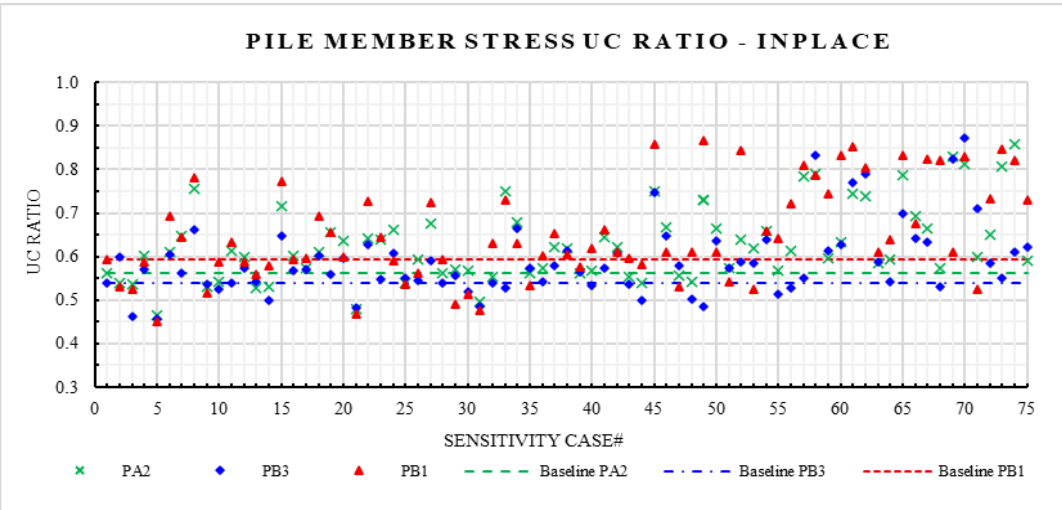

Figure. 20. Pile Member Stress UC Ratio for All Cases (Inplace)

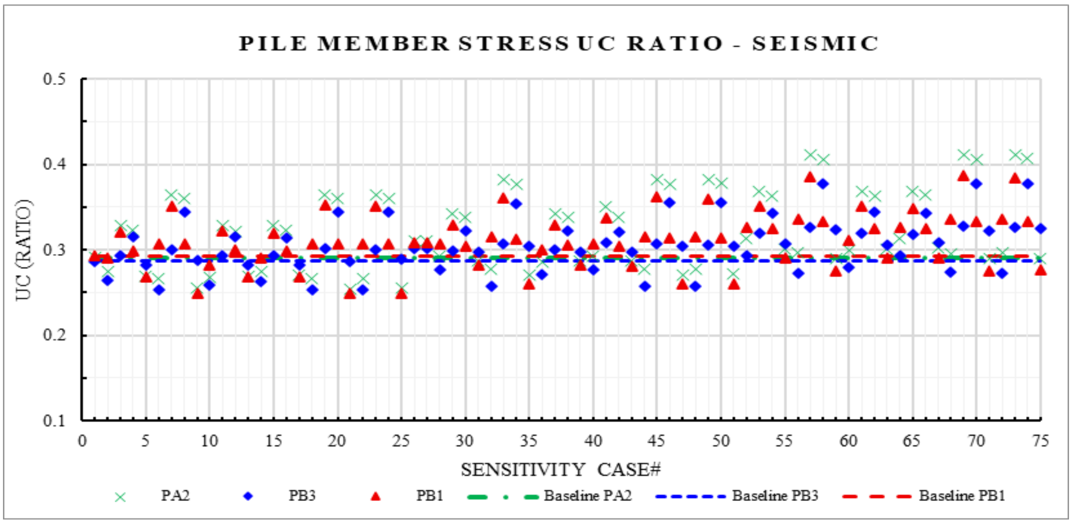

Figure. 21. Pile Member Stress UC Ratio for All Cases (Seismic)

For case of without weight increment, pile PA2 member stress UC ratio is increased 34.10% in inplace condition and 25.43% increase for seismic condition. While for case of CoG shift associated with 10% weight increment, the highest UC ratio increment is occurred at pile PB1 (45.79%) in inplace condition and 31.27% increase at pile PA2 for seismic condition. For case of CoG shift combined with 20% weight increment, the highest UC ratio increment up to 62.08% occurred at pile PB3 for inplace condition and up to 41.58% for seismic condition occurred at pile PA2.

The discussion of sensitivity obtained the following:

1) The pile member stress UC ratio determined moderate to highly sensitive for CoG shift cases with the degree is elevated along with magnitude of weight increment.
2) Pile PB3 is most sensitive (62.08%) due to case 70 (3D-Upward shift toward corner C5, 3m shift arm). Pile PB1 is less sensitive compared to PB3 for inplace condition. Meanwhile, for seismic condition, the most sensitive case occurred at pile PA2 for shift toward corner B2, B5, and B6 associated with weight increment of 20%.
3) The stress UC ratio variations of increment for seismic condition generally smaller magnitude compared to inplace. This indicates the behavior of tripod structure more stable under CoG shift variation during seismic condition compare to inplace condition.

G. *Maximum Value of Each Parameter*

For the purpose of to summarize the maximum value for each parameter with respect to inplace and seismic condition, Table 4 and Table 5 is presented. It is observed clear trend of marginal sensitivity in natural period due to CoG shift in 2D and 3D case only for both inplace and seismic conditions. However, the period significantly varies if the CoG shift is combined with weight increment resulted the period elevate longer than 3.50 second. It means, the weight increment is the main driver for longer period of the offshore tripod structure.

TABLE 4
MAXIMUM VALUE OF RESPECTIVE PARAMETERS FOR INPLACE CONDITION

Parameters		CoG Location & Cases											
		Without Weight Increase				Weight Increase 10%				Weight Increase 20%			
		Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down	Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down	Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down
Natural Period (s)		3.305	3.308	3.305	3.313	3.305	3.537	3.533	3.542	3.532	3.537	3.533	3.541
Max. Overturning Moment (MN-m)		83.67	118.33	117.08	133.46	79.801	103.917	113.492	102.756	112.249	112.798	117.572	116.686
Max. Base Shear (MN)		2.049	2.817	2.762	3.192	1.925	2.415	2.700	2.421	2.617	2.657	2.744	2.717
Pilehead lateral displacement (cm)	PA2	14.00	20.95	15.88	18.97	13.13	19.02	23.35	20.75	16.72	23.71	25.48	22.86
	PB3	13.08	16.80	15.48	15.48	12.94	17.08	21.82	16.14	13.23	23.71	25.48	19.20
	PB1	14.24	19.27	16.73	17.19	13.65	17.82	25.04	21.35	15.28	23.09	24.88	20.56
Axial compression load to pile (MN)	PA2	14.215	15.205	15.300	15.144	14.915	16.267	18.234	16.324	13.876	18.816	19.361	16.354
	PB3	10.867	12.711	11.813	12.510	10.988	13.167	13.089	13.284	11.924	13.238	15.135	13.552
	PB1	10.649	14.710	14.734	15.164	11.175	12.813	12.790	13.136	13.307	15.262	14.915	15.560
Maximum pile member stress UC ratio	PA2	0.563	0.755	0.655	0.716	0.592	0.749	0.749	0.729	0.675	0.790	0.831	0.859
	PB3	0.538	0.661	0.602	0.647	0.546	0.664	0.748	0.636	0.590	0.832	0.872	0.698
	PB1	0.594	0.780	0.692	0.773	0.563	0.731	0.858	0.866	0.723	0.845	0.853	0.846
Minimum pile bearing capacity FoS	PA2	2.055	1.932	1.921	1.939	2.966	1.816	1.633	1.810	2.101	1.586	1.605	1.544
	PB3	2.813	2.327	2.424	2.443	2.730	2.341	2.332	2.350	2.695	2.239	2.465	2.230
	PB1	2.390	1.783	1.780	1.733	2.289	2.023	2.026	1.977	1.954	1.723	1.814	1.693

TABLE 5.
MAXIMUM VALUE OF RESPECTIVE PARAMETERS FOR SEISMIC CONDITION

Parameters		CoG Location & Cases											
		Without Weight Increase				Weight Increase 10%				Weight Increase 20%			
		Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down	Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down	Origin	Shift 2D	Shift 3D-Up	Shift 3D-Down
Natural Period (s)		3.531	3.535	3.533	3.541	3.617	3.871	3.866	3.876	3.865	3.870	3.867	3.875
Max. Overturning Moment (MN-m)		32.089	32.883	34.696	32.770	31.125	31.237	32.894	29.598	29.917	32.062	33.776	30.380
Max. Base Shear (MN)		1.756	1.800	1.801	1.796	1.704	1.710	1.707	1.714	1.637	1.755	1.753	1.759
Pilehead lateral displacement (cm)	PA2	8.93	11.42	11.39	11.45	9.63	11.61	11.58	11.75	9.63	12.67	12.63	12.71
	PB3	9.01	12.17	12.07	12.26	9.69	12.59	12.92	13.14	9.69	13.82	13.73	13.92
	PB1	9.23	12.50	12.44	12.56	9.92	12.94	12.88	12.99	9.92	14.20	14.14	14.26
Axial compression load (MN)	PA2	12.437	13.553	13.592	13.592	13.138	14.103	14.350	14.285	13.677	14.991	15.031	14.955
	PB3	8.853	10.437	10.481	10.481	9.162	10.896	10.943	10.852	9.300	11.452	11.502	11.407
	PB1	9.177	10.994	11.052	10.927	9.506	11.222	11.279	11.162	9.663	11.798	11.858	11.738
Maximum pile member stress UC ratio	PA2	0.291	0.365	0.365	0.365	0.311	0.382	0.382	0.382	0.311	0.412	0.412	0.412
	PB3	0.287	0.345	0.345	0.345	0.302	0.354	0.356	0.355	0.302	0.378	0.378	0.378
	PB1	0.293	0.352	0.353	0.351	0.309	0.361	0.362	0.360	0.309	0.386	0.387	0.385
Minimum pile bearing capacity FoS	PA2	2.320	2.146	2.140	2.152	2.207	2.070	2.037	2.046	2.281	1.957	2.033	1.952
	PB3	2.813	2.433	2.424	2.443	2.730	2.341	2.332	2.350	2.695	2.239	2.465	2.230
	PB1	2.726	2.323	2.312	2.335	2.643	2.280	2.270	2.291	2.605	2.180	2.359	2.170

The significant variation in parameters (overturning moment, base shear, pile performances) across different CoG shift scenarios (2D, 3D-upward, 3D-downward), especially due to shift in 3D (upward and downward). Shifting of CoG often induce eccentric of loads (especially for combination of unfavorable CoG shift direction and increased mass), leading significantly increased of overturning moments, axial loads, lateral displacements, and higher stress utilization, while simultaneously reducing FoS of the pile foundation bearing capacity. Pile PA2 is the most severe in term of FoS, however pile PB1 and PB3 are the more sensitive than PA2.

Although, for case of shift 2D generally represent less severe cases compared to the 3D shifts, it still giving highlights about the sensitivity of the system's behavior to the precise location of the center of gravity. This implies that accurate determination of CoG is vital for design. In overall, the dynamic response of the offshore tripod structure at inplace condition is much more sensitive compared to seismic condition. The offshore tripod structure is still robust for its performance during seismic condition under variation of CoG location.

V. CONCLUSION

This study evaluated how CoG variation—both in 2D (X–Y) and 3D (X–Y–Z) with added topside mass—affects the structural response of a fixed offshore tripod platform. A total of 75 cases were analyzed to assess the influence on dynamic and seismic performance with the following conclusion:

1) The natural period is shown marginally vary under the 2D CoG and 3D CoG shifts without involvement of weight increase. However, combination with weight increment to lead the natural period tend becomes longer up to 7.15% for inplace condition and 9.77% for seismic condition. The natural period more sensitive to the case of CoG shift associated with weight increment toward corner B and C (2D shift, 3D-Upward and 3D-Downward).
2) Overturning moment and base shear were notably sensitive to CoG shifts and increased mass. For inplace condition, it is consistently increased for all cases due to combination with 20% weight increment. Irregular response for 2D and 3D shift without weight increment and 10% increment. For seismic condition, the response remained irregular for all sensitivity cases. For overturning moment and base shear parameter, the modeled offshore tripod structure more sensitive for case of CoG shift in 3D-Downward to corner A (arm distance of 3m).
3) Pilehead lateral displacement remained moderate for 2D shifts but increased by 94.80% in combination of 3D shifts and mass increment cases. Similarly, axial compression loads redistributed unequally, concentrating on outer piles under 3D shifts with added weight.
4) The performance of pile bearing capacity FoS is compromised for inplace condition and remained safe for seismic condition especially for pile bearing capacity FoS. It is due to the origin CoG already adjacent to pile PA2 that caused dominant weight distributed onto it and the pile FoS lesser than the acceptance limit. The case of CoG shift to corner A6 (3D-Upward) is resulted the most sensitive for pile bearing capacity FoS.
5) Pile member stress UC ratios remained below 0.75 for most 2D cases nonetheless escalate up near the 0.90 threshold under more extreme 3D and weight scenarios, indicating elevated local demand.

In overall performance, the tripod structure remained stable under the analyzed 2D CoG envelope. However, 3D CoG shifts—especially with mass increases—significantly impacted dynamic and structural performance. This highlights the importance of managing CoG within defined limits to maintain safe and efficient behavior of tripod offshore platforms. These findings are also relevant for decommissioning planning, where temporary CoG shifts during lift or removal could challenge structural capacity. Managing CoG within safe bounds is therefore essential for both operational integrity and end-of-life strategies.

Based the above conclusions, the following is recommended for future action:

1) CoG Control: Efforts should be made to keep the center of gravity within a safe operational zone. Cases beyond the mid-range of the sensitivity spectrum demonstrated disproportionate increases in demand and should be treated with caution during design or modification stages. Although 2D shift of + 2m and 3D (X-Y + 2m with Z + 1m) shown the tripod structure remained stable however the pile performance is compromised. In order to control the natural period elongation not further more than 3.5s, then it is proposed to limit the variation of CoG up to +1.5m in X-Y direction and + 1.0m in vertical Z direction if without weight increment. Any involvement of weight increment, the CoG shift envelope shall be limited to +1m in X-Y direction and maximum 10% weight increment during operating condition. In case, the CoG shift as per actual weight management report to be shifted more than the above, further non-linear analysis to be performed to check influence to fatigue.
2) Displacement Criteria: A more realistic operational limit shall be researched to avoid overly conservativeness to the structure. Therefore, further study to obtain the range of acceptable lateral displacement based on soil type can be proposed.
3) Structural Capacity Checks: Fatigue life, weld detailing, and soil-pile interaction effects should be validated especially for extreme CoG shift cases.
4) Operational Monitoring: During loadout or future modifications (such as retrofits or module installation), CoG position should be monitored closely. Momentary shifts in mass might leading to push the structure into a less favorable regime if not controlled. The monitoring of substructure's post installation also plays important role to verify the straightness of the structure and agreement of working point (interface between topsides and substructure). Any tilting occurred, continuous monitoring through engineering and operational actions are required to ensure the necessity of physical modification to maintain the integrity of the structure.
5) Future Studies: Extend this work into nonlinear soil-structure interaction and time-history seismic simulations to capture potential performance including to examine the proposed limit of CoG shift and weight increase to the existing tripod structure. This would be especially valuable for load cases that approach peak displacement or stress utilization thresholds.

Acknowledgements

The authors wish to extend their gratitude to Institut Teknologi Sepuluh Nopember (ITS) Surabaya, PT. Pertamina Hulu Mahakam (PT. PHM), and Upstream Oil and Gas Regulatory Special Task Force (SKK Migas) for their invaluable support, contributions, and collaboration, which were instrumental in the successful completion of this research.

References

[1] B. Lepic, "Indonesia set for massive offshore decommissioning push," Offshore Energy, 2021. [Online]. Available: https://www.offshore-energy.biz/indonesia-set-for-massive-offshore-decommissioning-push
[2] S. K. Chakrabarti, Handbook of Offshore Engineering, Amsterdam, The Netherlands: Elsevier, 2005.
[3] S. Chandrasekaran and A. Jain, Ocean Structures: Construction, Materials, and Operations, Boca Raton, FL, USA: CRC Press, 2017.
[4] S. Z. A. Syed Ahmad et al., "Offshore Structural Reliability Assessment by Probabilistic Procedures – A Review," J. Mar. Sci. Eng., vol. 9, no. 998, 2021. doi: 10.3390/jmse9090998
[5] M. S. Mia, Offshore Structures and Loads & Strength of Offshore Structure, 2024.
[6] Z. Henry, I. Jusoh, and A. B. Ayob, "Structural Integrity Analysis of Fixed Offshore Jacket Structures," J. Mekanikal, vol. 40, pp. 23–36, Dec. 2017.
[7] J. Jia, "The effect of gravity on the dynamic characteristics and fatigue life assessment of offshore structures," J. Constr. Steel Res., vol. 118, pp. 1–21, 2016. doi: 10.1016/j.jcsr.2015.09.013
[8] S. Nallayarasu, Offshore Structures Analysis and Design, Madras: IIT Madras, 2012.
[9] N. D. Alias et al., "Digital Transformation of Offshore Structure Weight Control Management into Digitally Integrated and Intelligent Analytical Tool," SPE Asia Pacific Oil & Gas Conf. and Exhibition, 2022. doi: 10.2118/210712-MS
[10] ISO 19901-5:2021, Petroleum and Natural Gas Industries — Specific Requirements for Offshore Structures — Part 5: Weight Management, Geneva, Switzerland: ISO, 2022.
[11] R. Rui and J. Walker, "Upstream Offshore-Facility Weight-Growth Study," Oil Gas Facil., vol. 4, no. 2, pp. 107–112, 2015. doi: 10.2118/170696-PA
[12] S. M. Ng et al., "A technical limits weight control tool for integrity management of aging offshore structures," presented at Offshore Technology Conf., 2022. doi: 10.4043/31655-MS
[13] API RP 2A-WSD, Recommended Practice for Planning, Designing and Construction of Fixed Offshore Platforms. Working Stress Design, 21st ed., Aug. 2007.
[14] A. A. Elshafey, M. Haddara, and H. Marzouk, "Dynamic response of offshore jacket structures under random loads," Marine Structures, vol. 22, no. 3, pp. 504–521, 2009. doi: 10.1016/j.oceaneng.2014.04.008
[15] M. Noureldin and J. Kim, "Sensitivity analysis of pile-founded fixed steel jacket platforms subjected to seismic loads," Ocean Eng., vol. 85, pp. 1–11, 2014. doi: 10.1016/j.oceaneng.2014.04.008
[16] M. Noureldin and J. Kim, "Seismic performance evaluation and retrofit of fixed jacket offshore platform structures," J. Perform. Constr. Facil., vol. 29, no. 4, p. 04014099, 2015. doi: 10.1061/(ASCE)CF.1943-5509.0000576
[17] H. R. Karimi and N. Shabakhty, "A comparative assessment of the impact of tilting on structural safety and behavior of fixed offshore platforms," Appl. Ocean Res., vol. 136, 103589, 2023. doi: 10.1016/j.apor.2023.103589
[18] A. Rezaeian, F. Hosseinlou, and A. Shnewer, "Dynamic response of offshore platforms installed on sloping seabed under actual seismic loading," J. Hydraul. Struct., vol. 9, no. 2, pp. 14–31, 2023. doi: 10.22055/jhs.2023.43809.1253
[19] M. S. Pinheiro and P. R. D. Monteiro, "Decommissioning Plans for Fixed Offshore Platforms: A Brief Revision," Asian J. Eng. Technol., vol. 9, no. 3, pp. 62–68, Jun. 2021. Available: www.ajouronline.com
[20] TOTAL ENERGIES, GS EP STR 101 – Design of Fixed Steel Offshore Structure, 2022.
[21] SHELL SES 10.1 Rev. 4, Design of Fixed Offshore Structure, Sarawak Shell Berhad/Sabah Shell Petroleum Co. Ltd., 2005.
[22] PTS 34.19.10.30 Rev. 7, Design of Fixed Offshore Structure (Working Stress Design), PETRONAS TECHNICAL STANDARD, 2012.
[23] M. H. Bukhari et al., "Reassessment of Minimum Facilities Wellhead Platform due to Installation of Extension Compressors against Earthquake Loads," IOP Conf. Ser.: Earth Environ. Sci., vol. 1198, p. 012012, 2023. doi: 10.1088/1755-

1315/1198/1/012012
[24] M. Zarrin, B. Asgarian, and M. Abyani, "Probabilistic Seismic Collapse Analysis of Jacket Offshore Platforms," J. Offshore Mech. Arctic Eng., vol. 140, no. 3, p. 031601, Jun. 2018. doi: 10.1115/1.4038581
[25] J. L. Humar, Dynamics of Structures, 2nd ed., Rotterdam, Netherlands: Balkema, 2002.
[26] N. D. P. Barltrop and A. J. Adams, Dynamics of Fixed Marine Structures, 3rd ed., Oxford, U.K.: Butterworth-Heinemann, 1991.
[27] ISO 19901-2: Petroleum and Natural Gas Industries — Specific Requirements for Offshore Structures — Part 2: Seismic Design Procedures and Criteria, 1st ed., Geneva, Switzerland: ISO, 2004.
[28] Wilson, E. L., Der Kiureghian, A., & Bayo, E., “A replacement for the srss method in seismic analysis. Earthquake Engineering & Structural Dynamics”, 9(2), 187–192., 1981, https://doi.org/10.1002/EQE.4290090207
[29] Former Operator of Mahakam Block, “Structural Design Basis of Sisi Nubi 2B and Peciko 7B,” 2012.
[30] API RP 2A 1986: API Recommended Practice 2A (RP 2A): Planning, Designing and Constructing Fixed Offshore Platforms, 18th ed., Washington, DC, USA: API, 1986.
[31] API RP 2EQ, Seismic Design Procedures and Criteria for Offshore Structures, 1st ed., Washington, DC, USA: API, Nov. 2014. [Online]. Available: https://standards.globalspec.com/std/14362608/api-rp-2eq
[32] AISC, Manual of Steel Construction, Allowable Stress Design, 9th ed. Chicago, IL, USA: AISC, 1989.
[33] API RP 2SIM, Structural Integrity Management of Fixed Offshore Structures, 1st ed., Nov. 2014.
[34] ASCE, Seismic Evaluation and Retrofit of Existing Buildings, ASCE/SEI 41-17, 2017.
[35] ISO 23469:2005, Bases for Design of Structures — Seismic Actions for Designing Geotechnical Works, Geneva, Switzerland: ISO, 2005.
[36] DNV GL, DNVGL-OS-C201: Structural Design of Offshore Units – WSD Method, Offshore Standard, July 2015. [Online]. Available: https://standards.globalspec.com/std/10164835/dnvgl-os-c201
[37] A. K. Chopra, “Dynamics of Structures: Theory and Applications to Earthquake Engineering”, 4th ed. Upper Saddle River, NJ, USA: Prentice Hall, 2012.
[38] Bentley Systems, SACS IV Manual, Structural Analysis Computer System, 2016.
[39] A. E. Mityaev, I. V. Kudryavtsev, V. A. Ivanov, M. P. Khomutov, and A. V. Kolotov, "Influence of a center of mass position on beam natural frequencies of vibration," J. Phys.: Conf. Ser., vol. 1889, no. 2, p. 022119, 2021. doi: 10.1088/1742-6596/1889/2/022119
[40] F. Hosseinlou, H. Hokmabady, A. Mojtahedi, and S. Mohammadyzadeh, "Seismic analysis of an offshore structure in Persian Gulf utilizing a physical model," Int. J. Maritime Technol., no. 11, pp. 21–31, 2019. doi: 10.29252/ijmt.11.21
[41] Gurnita, O., & Nindyanto, A. S., “Lima field seabed subsidence: Synchronized deck raising project: How it was done.” PM World Journal, 3(8), August, 2014, https://www.pmworldjournal.net
[42] L. Virgin, S. Santillan, and D. Holland, "Effect of gravity on the vibration of vertical cantilevers," Mech. Res. Commun., vol. 34, no. 3, pp. 312–317, 2007. doi: 10.1016/j.mechrescom.2006.12.006
[43] M. Suarjana and W. Kiesin, "Sensitivity Analysis of Offshore Platform Structures Under Varying Scour Depths," J. Tek. Sipil, vol. 20, no. 1, pp. 157–174, 2024. doi: 10.28932/jts.v20i1.6788
[44] S. E. A. Raheem et al., "In-place analysis for design-level assessment of the fixed offshore platform," Ships Offshore Struct., vol. 16, no. 8, pp. 892–903, 2021. doi: 10.1080/17445302.2020.1787931
[45] Y. Al-Radhi and F. Hejazi, "Assessment of Aged Offshore Jacket Type Platforms Considering Environmental Loads and Degradation Parameters," SDHM Struct. Durability and Health Monit., vol. 17, no. 2, pp. 89–113, 2023. doi: 10.32604/sdhm.2023.011439
[46] Z. Omrani, R. Amirabadi, and M. Sharifi, "Effect of uncertainty on the seismic behaviour of fixed pile-founded offshore platforms," Ships Offshore Struct., vol. 18, no. 8, pp. 1181–1191, 2023. doi: 10.1080/17445302.2022.2109353