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Pushover Analysis of Ageing Offshore Jacket Platform in Shallow Water Under Extreme Storm and Mitigation Strategy for Platform's Life Extension

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***Abstract*: Many offshore jacket platforms worldwide have approached or exceeded their original design life but are still in use and productive. According to the international codes, standards, and industry best practices, structural assessments of ageing fixed offshore jacket platforms shall be conducted against relevant target values to assess whether it is fit for purpose or risk reduction measures should be considered for continuing its operation. This research examines the collapse behaviour of an ageing offshore jacket platform under extreme storm conditions. Nonlinear collapse analysis has been performed to assess fixed offshore jacket platforms' structural integrity and reliability in shallow water under extreme storm conditions. Two tripods and 4-legged jacket platforms at water depths between 30 to 80 meters, located in the Mahakam Delta, Kalimantan, Indonesia, have been selected in this research as wellhead platform models commonly installed in shallow water. Sensitivity studies examine the effects of pile-soil interaction, variations in pile depth, topside load adjustments, marine growth removal, and jacket strengthening on structural performance. From the structural integrity and reliability perspective, the findings highlight that strengthening the jacket by adding soldier piles is the most effective approach for extending the platform's lifespan, especially for a wave-dominated platform. Additionally, a cost feasibility analysis is advised for future evaluation to determine whether jacket strengthening is viable or if alternative risk reduction strategies should be further explored for the ageing offshore platform.**

Keywords—Ageing Offshore Jacket Platform; Mitigation Strategy; Probability of Failure; Structural Integrity; Structural Reliability; Platform's Life Extension.

I. INTRODUCTION

Many offshore platforms in production have already reached and passed their original design life [1],[2],[3]. Some factors behind the above phenomenon are economic potential and continued requirements to boost oil or gas output, whether from the original fields or as a base for nearby fields and subsea operations [4],[5]. Other factors encouraging operators to perform several life extension studies are enhanced production and drilling techniques, reduced profit margins due to low oil prices, uneconomical discoveries of smaller fields, and substantial remaining oil reserves in existing fields [6].

Over 50% of offshore platforms on the Norwegian Continental Shelf (NCS), the United Kingdom Continental Shelf (UKCS), and the Gulf of Mexico Shelf, and approximately 70% of offshore platforms in the Middle East, accounting for around 800 platforms, are currently operating beyond their original design lifespan of 20 to 25 years [6],[7],[8],[9],[10],[11]. In Indonesia, 54.65% of the 613 offshore platforms have surpassed 20 years of service and continue operating beyond their designed lifespan due to remaining oil or gas reserves in their operational areas [12].

Some technical concerns may arise when the ageing offshore jacket platform reaches its operational life, such as structural modifications, changes in met ocean parameters, or additional topside loads due to operation requirements or the current situations. Mitigation actions due to the above technical concerns shall be required to keep the platform safe and structurally robust under operating and extreme storm conditions for continued service.

This research concentrates on the most prevalent and well-established type of fixed jacket support platform, which is used on over 95% of the world's offshore platforms [13],[14]. The research aims to evaluate the behaviour of an ageing offshore jacket platform in shallow water under extreme storm conditions and to identify an appropriate mitigation strategy for extending the platform's lifespan in compliance with current codes, standards, and industry best practices. This research has conducted sensitivity studies on the parameters and criteria used in the structural analysis and how those parameters may affect platform integrity.

Compliance with international codes is essential when evaluating the ability of an ageing offshore jacket platform to ensure its structural integrity in severe environmental circumstances. This research is also intended to examine how structural integrity and reliability focused on the likelihood of failure of the ageing jacket platform would be crucial factors to aid in

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decision-making for the platform’s life extension, which is expected to contribute to existing scientific knowledge and further research.

II. Literature Review

The oil and gas industry encounters a major challenge in preserving the structural integrity of ageing infrastructure for prolonged operational use. Accurate degradation of platform modelling and maintenance planning is important to extend offshore structures' operating lives safely [6]. One factor that triggers the reassessment of an existing offshore platform is exceeding the structure's original design lifespan. It is also important to know the behaviour of ageing fixed offshore jacket platforms for maintenance or decommissioning [15]. Prior to assessing the ageing offshore platform's condition, information on history, current situation data, and platform inspection reports should be collected to predict its future state and observe for planning any possible life extension for the platform. Existing codes and standards suggest reassessing structures for life extension utilising linear analysis, analysis of nonlinear system strength, and analysis of structural reliability to evaluate the ultimate limit state of the structure [1].

Failure modes must be reviewed when assessing the platform’s life extension since they may be present in ageing structures [1]. When an ageing platform needs to be reassessed for requalification purposes, it must withstand the codes' ultimate strength loading criteria. Reassessment can evaluate strength performance to predict the structure's response to excessive loads and identify potential deficiencies in strength [15].

The platform's reassessment seeks to determine whether the structure remains suitable for its intended purpose and identify measures to minimise the risk of severe storms to As Low as Reasonably Practicable (ALARP). Some factors, such as the consequence of failure, the cost-effectiveness of risk reduction measures, and the expected reliability level, must be considered when deciding on risk reduction on an ageing platform.

One of the most significant hazards to structural integrity is extreme weather, which includes waves, currents, and wind [16]. A sensitivity analysis of offshore jacket-type platforms to wave loading hazards was conducted using an assessment of a jacket structure in the Arabian Gulf region. The study showed that the critical factors affecting the platform's behaviour come from the foundation and the yield stress of the bracing members [17]. Conducting non-linear analysis accurately captures the behaviour of structures, enabling the development of economical and rational structural designs [13].

The El Morgan M-100 platform, built in 1960 in the Gulf of Suez, Egypt, used the latest API 21st edition to examine the impact of variations in wave height, associated wave periods, current velocity, and marine growth on the jacket structure. An increase in the height of waves and the speed of currents reduces the factor of safety (FoS) and increases displacements and stresses. Further, marine growth will affect the platform structure by increasing forces, displacements, and bending moments on the piles, thus lowering its safety margins. [18].

Structural reliability refers to a structure's capacity to remain fit for purpose under various conditions, including operational, extreme, fatigue, and accidental scenarios, over a defined period. It evaluates the probability of failure using deterministic and probabilistic methods [15]. The term "fit-for-purpose" means that it should not be required that all existing structures always meet the absolute prescriptive standards [19]. Furthermore, it has been successfully applied under Shell's operations in the North Sea. A methodology was developed for the Reliability-Based Design and Assessment (RBDA) of an ageing fixed offshore structure. It has been successfully applied in the North Sea through Shell's operating company to facilitate a detailed reassessment focused on managing the structure's safety, conducting integrity assessment, and maintaining reliability by analysing the loads applied to the platform structure [20]. Further, the other methodology, Global Ultimate Strength Assessment (GUSA), for Malaysia jacket structures, reassess a structure's safety, integrity, and reliability by evaluating its loading as a high-end structural analysis for Risk-based Assessment (RBA) [21].

As described in the literature, Numerous reliability assessment methods for offshore jacket platforms have been developed and widely adopted within the oil and gas industry [19],[22],[23]. Another framework, Probabilistic Incremental Wave Analysis (PIWA), has been employed to evaluate the performance of jacket offshore platforms subjected to extreme wave conditions [24]. The other approach to obtaining the ultimate capacity of offshore platforms is to perform the Incremental Wave Analysis (IWA) [25]. The assessment was done to evaluate the offshore structure using nonlinear dynamic analysis, which is subjected to irregular wave forces called Endurance Wave Analysis (EWA). This EWA model gradually worsens sea conditions to assess structural integrity from elastic behaviour to collapse. The approach is suited for both new design and existing structure assessments [26]. Further, the assessment of the probabilistic method under extreme waves utilising the Modified Endurance Wave Analysis (MEWA), which is effective, has been made, i.e., for the Ressalat Platform in the Persian Gulf [27].

Evaluating the platform's performance behaviour through nonlinear structural analysis and accurate modelling of the platform plays an essential role in ensuring design safety and operational cost feasibility [28]. The behaviour of offshore jacket platforms under wave forces, influenced by the flexibility and nonlinear characteristics of the supporting piles, has been extensively analysed [29]. However, the most prevalent approach to assessing the ultimate capacity of an offshore platform is to perform a non-linear collapse or pushover analysis. This method estimates and evaluates the demands placed on structures and their elements, then compares them to the existing capacity to determine the design's reliability and acceptability [45]. Concerns related to pushover analysis often focus on the choice of load patterns, the extent of pushing, and the aspects being assessed [30].

Interaction between soil-pile and structure platform during the analysis of jacket structure should not be ignored since it would result in underestimating the vertical and lateral vertical deflections, shear forces, support reactions, bending moments in the legs, and axial forces in the beams and plan bracings of the structure [31]. The study has been carried out on the foundation by modelling it, utilising non-linear soil springs, uncoupled and distributed along the pile's length, to investigate the performance of jacket platform under environmental loads, which are significantly influenced by the pile-soil interaction [32].

A sensitivity study of the jacket-type offshore platforms considered to have a Pile-Soil-Structure Interaction (PSSI) has also been investigated. It is confirmed that soil properties are the primary source of uncertainty in jacket structures' nonlinear static and dynamic behaviours [17]. The pile and soil interaction and horizontal bracing significantly impact the determination of the platform's ultimate capacity to enhance the structural integrity and reliability of jacket platforms [33].

The other sensitivity study has been conducted considering pile structure interaction (PSI) with and without pile structure to assess the impact on reserve strength ratio (RSR) and identify parameters that significantly impact RSR. Pushover analysis with PSI results in a reduced RSR compared to analyses without PSI, which gives a difference of about 5.6% to 50.1% [34]. The axial ageing effects of pile foundations are investigated, and they are expected to improve the platform's structural integrity. The maximum improvement in RSR of the observed jackets was 11% and 27%, whereas the maximum decrement was about 11% and 17% [35]. In this research, the axial ageing effects of the pile foundation will not be considered in the analysis; instead, variation in the pile depth will be applied to compare the proposed scenarios.

When major changes in structural integrity are identified, managing and maintaining structural integrity becomes a critical aspect of platform life extension studies, and it is essential to implement appropriate strengthening, modification, and/or repair (SMR) plans [36],[37],[38],[39]. Strengthening or reinforcing the jacket platform can effectively decrease its Probability of Failure (PoF). As strengthening reaches up to 20%, the confidence index improves while the probability of failure (PoF) decreases [40].

The methodologies adopted in determining the probability of failure (PoF) due to hazards posed by extreme storms on the existing platform have been published in the literature [19],[22],[23]. The study, which reviewed and discussed the reliability of specific members of Malaysian offshore jacket platforms, confirmed that a higher reliability index corresponds to a lower probability of component failure. In addition, variation in met ocean values does not have much effect on component reliability [41].

According to API RP 2A-WSD 21st Edition, October 2007, Sec. 17.5.2; Table 17.5.2b, RSR values on the assessment criteria for other U.S. areas (still in U.S. areas) is 1.60 for manned non-evacuated or unmanned platform with a high consequence factor and 0.8 for unmanned platform with a low consequence factor. According to ISO standards, platforms are designed for a 100-year return period load but are reassessed for a 1,000-year return period or a 1×10^{-3} Probability of Failure (PoF) for unmanned platforms, and a 10,000-year return period or a 1×10^{-4} PoF for manned platforms. By considering the effects of ageing on the pile, the RSR of the existing jacket platform can be improved or reduced [42].

Pushover Analysis

Pushover analysis is a static nonlinear collapse analysis method used to determine a structure's ultimate capacity by demonstrating its instability. The analysis evaluates the structural response against lateral load by applying progressively increasing environmental loads until the structure reaches the point of collapse.

The fundamental concept of pushover analysis involves incrementally applying environmental loading to the structure using a specified load factor. Nodal displacements and element forces are evaluated at each load step, with corresponding updates to the stiffness matrix. Plasticity will be introduced when a member's stress occurs and reaches the yield stress. It reduces the structure's stiffness, redistributing subsequent load increments to adjacent members and those affected by plastic deformation. This progressive collapse of members will continue until the structure as a whole collapses. [43].

The analysis considers the gravity loads associated with the critical 100-year storm environmental load. This critical load case is stepped up with a factor until the structure collapses. Figure 1. shows that the environmental load gradually increases on the jacket platform until structural collapse.

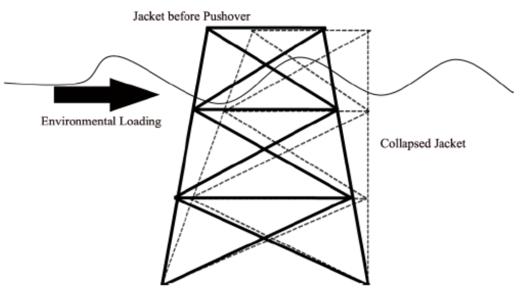

Figure. 1. Environmental loads gradually applied to the jacket structure until its collapse

The Reserve Strength Ratio (RSR) evaluates the structure's capability to resist loads exceeding the limits outlined in the platform's design criteria. This reserve strength can maintain the platform's operations beyond its intended service life.

RSR represents the ratio between the collapse base shear and the design base shear for a 100-year return period, as shown in Equation 1:

$$RSR = \frac{BS_{ultimate}}{BS_{design}} \quad \text{……….} (1)$$

Where:

$BS_{ultimate}$ = the ultimate capacity

BS_{design} = base shear loading design on the jacket for the 100-year return period of met ocean loading.

The base shear design is calculated when the environmental load factor equals 1.0, whereas the collapse base shear represents the maximum base shear experienced at the point of collapse, as illustrated in **Figure 2.**, [44].

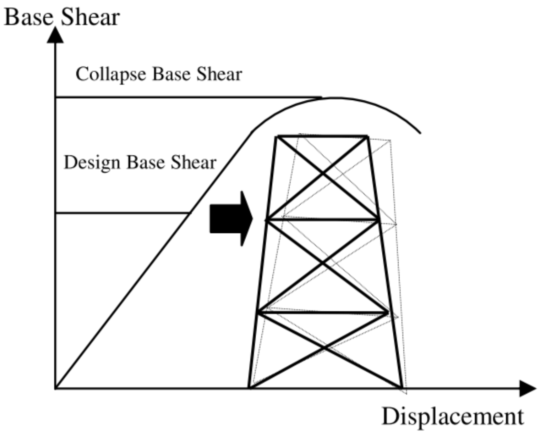

Figure. 2. Collapse base shear and design base shear for structural reliability analysis

Once the pushover analysis obtains the RSR value of the ageing platform, it is to be used further in reliability assessment to get the probability of failure (PoF) [45]. Structural reliability quantifies the system's probability of failure (PoF) due to uncertainties in the design, fabrication, and environmental conditions. It is associated with the structure's ability to fulfil its design purpose for a specified design lifetime [45].

Reliability should be evaluated using predictive models and probabilistic methods. It can be represented as the probability of failure (PoF) or the reliability index (β). The structure cannot function as intended once it exceeds a defined threshold, referred to as the limit state. The limit state represents the safety margin between a structure's resistance and the applied load.

The limit states considered for the reliability analysis are:

1) Ultimate limit states, i.e., shear failure, flexural failure, collapse
2) Serviceability limit states, i.e., durability, cracking, deflection, and vibration.

Equations 2 and 3 and **Figure 3.** show the limit state function or failure, G, and the probability of failure (PoF) [46].

$$G = R - S \quad \text{……….} (2)$$

$$PoF = \Phi\,(-\beta) = \Pr\,(R \leq S) = \Pr\,(R - S \leq 0) = \Pr\,(G(R,S) \leq 0) = \int_0^{\infty} F_R\,(r) f_S(r) dr \quad \text{……….} (3)$$

Where:

R = the resistance of the system
S = the loading of the system
Φ = the standard normal distribution function
β = reliability index
f_S (r) = the probability density function of the load
F_R (r) = the cumulative probability density function of the resistance

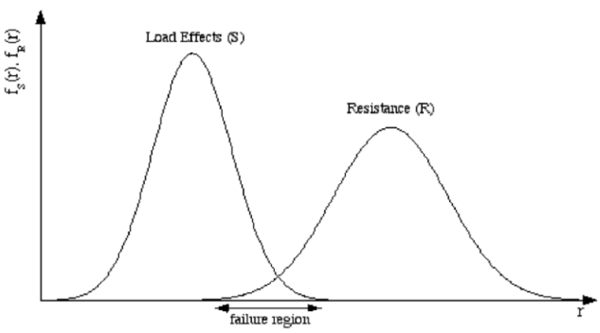

Figure. 3. Definitions of load effects and structural resistance [46].

A structural element fails when the Load model (S) exceeds the Resistance model (R) (S) exceeds the Resistance model (R), as shown in **Figure 4.**, [46].

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \quad \text{……….} (4)$$

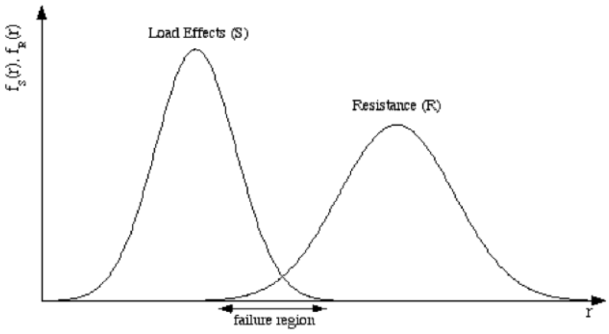

Figure. 4. Definition of the probability of failure [46].

The reliability index or safety index, β, is defined in Equation 4. It represents the distances of the mean margin of safety from the failure surface, so the more significant the reliability index value, the safer the structure.

Curves must be developed to calculate the return period and the probability of platform collapse failure. One structural assessment methodology uses a probabilistic approach, referring to AIM-ALE2021-76076 [23]. In this method, the hazard curve is developed in the log-normal chart from the higher base shear among the eight or twelve environmental directions from extreme storms for all the return periods. The hazard curves describe the platform loading variation as the return period function. The substructure loading is assumed to follow a straight line on a log scale and is essentially an exponential distribution. The curve distribution shape, or hazard curve gradient, will be calculated by evaluating the global loading for each direction for 100-yr, 1,000-yr and 10,000-yr return periods. The hazard curve and substructure capacity are used to calculate the probability of failure for each analysed direction. The inverse of the return period represents the probability of failure for each direction and vice versa.

The other methodology used to get the probability of failure is simplified reliability calculation to determine pile failure [47]. A simple safety margin calculation is carried out, in which the structures are assumed to be unable to redistribute load between their piles.

III. Research Methodology

A. General

Structural integrity and reliability assessments are essential to have beyond platforms to know for their fit-for-purpose during operation throughout, especially for the platform has approached or beyond-deploy in ageing jacket platforms to ensure their fit-for-purpose during operation, significantly when the platform has approached exceeded its design life.

Structural integrity is defined as a structure's capacity to withstand its designed loads without experiencing failure from deformation, fractures, or fatigue. Structural reliability measures the likelihood that a system may fail due to uncertainties in its design, fabrication, and environmental conditions. A structure's reliability also relates to its capability to perform its intended function throughout a specified design lifespan. Reliability assessment aims to estimate the total annual probability of failure (PoF) under extreme storm cases. This assessment requires numerical models and probabilistic approaches to estimate the likelihood of failure for a structure or system. The probability of failure is determined by accounting for uncertainties and variations in factors such as loads, material properties, dimensions, and other performance-influencing parameters. The ageing jacket platform assessment must comply with API RP 2A WSD and API RP 2SIM. Considering the corroded jacket members in the splash zone, the ageing jacket platform has been modelled by reducing a 6mm tubular member thickness. However, the member weight shall remain unchanged by overriding the section area of the respective member. The structural capacity is evaluated based on reserve strength ratio (RSR) and annual probability of failure (PoF) values to withstand specific loading conditions and identify mitigation actions as required. The PoF is derived from the long-term distribution of environmental loading on the structure and the structure's resistance or ultimate strength.

The RSR of a jacket platform measures its capacity to resist the maximum loads it may experience during its design life. It is described as the ratio of the platform's ultimate strength to the maximum load intended to support. As defined by API RP 2A, the RSR is calculated as the ratio of the base shear at collapse (i.e., Ultimate Limit State, ULS) to the 100-year storm base shear (i.e., F design) with the application of load factor approach on the environmental loads as depicted on **Figure 5.**, and **Figure 6**. It is to be noted that the wave height is not increased. The base shears include wave, current and wind loads, the main factors causing ductile collapse.

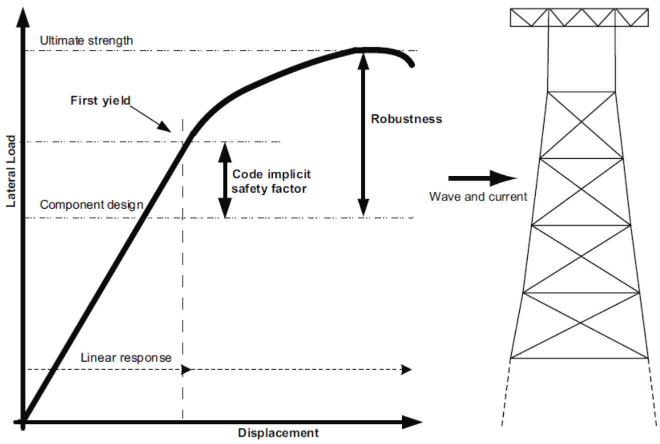

Figure. 5 Schematic Load vs Deformation Diagram of an X-braced Platform (Taken from API RP 2SIM – Figure A.2)

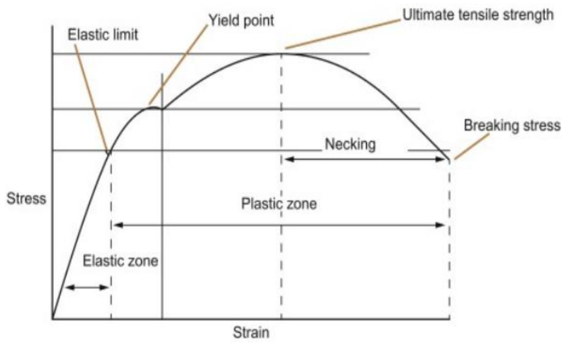

Figure. 6. Stress and Strain Relationship

B. Platform Assessment Method

The platform assessment is performed by running a pushover or non-linear collapse analysis using the SACS Collapse module for the predominant environmental direction. The models of tripods and 4-legged platform taken from the Structural Design Basis [48] are created using the SACS Precede module, as outlined in **Table 1**.

TABLE 1.
OVERVIEW OF PLATFORM MODELS

Description	Tripod-1	Tripod-2	4-Legged Platform
Water Depth with Reference to CD	47.30 m	66.60 m	71.70 m
Mudline Elevation with Reference to CD	(-) 47.30 m	(-) 66.60 m	(-) 71.70 m
Jacket Work Point Elevation	(+) 6.50 m	(+) 6.50 m	(+) 6.70 m
Platform North Orientation from the True North	Same as True North	Same as True North	Same as True North
Deck Configuration with respect to Chart Datum	Upper Deck EL (+) 21.90 m Main Deck EL (+) 16.10 m Cellar Deck EL (+) 11.6 m Suspended Deck EL (+) 8.60 m	Upper Deck EL (+) 21.90 m Main Deck EL (+) 16.10 m Cellar Deck EL (+) 11.6 m Suspended Deck EL (+) 8.60 m	Weather Deck EL (+) 20.77 m Mezzanine Deck EL (+) 16.00 m Lower Deck EL (+) 12.6 m Sump/ SDV Deck EL (+) 8.55 m
Platform Brace Type	X-Braced Tripod Jacket	X-Braced Tripod Jacket	X-braced and vertical diagonal 4-leg Jacket

Description	Tripod-1	Tripod-2	4-Legged Platform
Number of Leg	3 nos.	3 nos.	4 nos.
Number of Pile	3 nos. of 48" dia piles	3 nos. of 48" dia piles	4 nos. of 60" dia piles
Number of Conductor	6 nos. of 36" dia conductor pipes 3 nos. of 30" dia conductor pipes	3 nos. of 36" dia conductor pipes 4 nos. of 30" dia conductor pipes	2 nos. of 36" dia conductor pipes 8 nos of 30" dia conductor pipes
Number of Riser	1 no. of 20" dia riser	1 no. of 16" dia riser	1 no. of 16" dia riser
Number of Boat landing	1 no. at Row B	1 no. at Row B	1 no. at Row 2
Number of Vent Boom	1 no. at Row A	1 no. at Row A	1 no. at Row A
Number of V-shape	2 nos. located between Leg A2 & B1 and at Leg B3	2 nos. located between Leg A2 & B1 and at Leg B3	1 no. at Leg B1

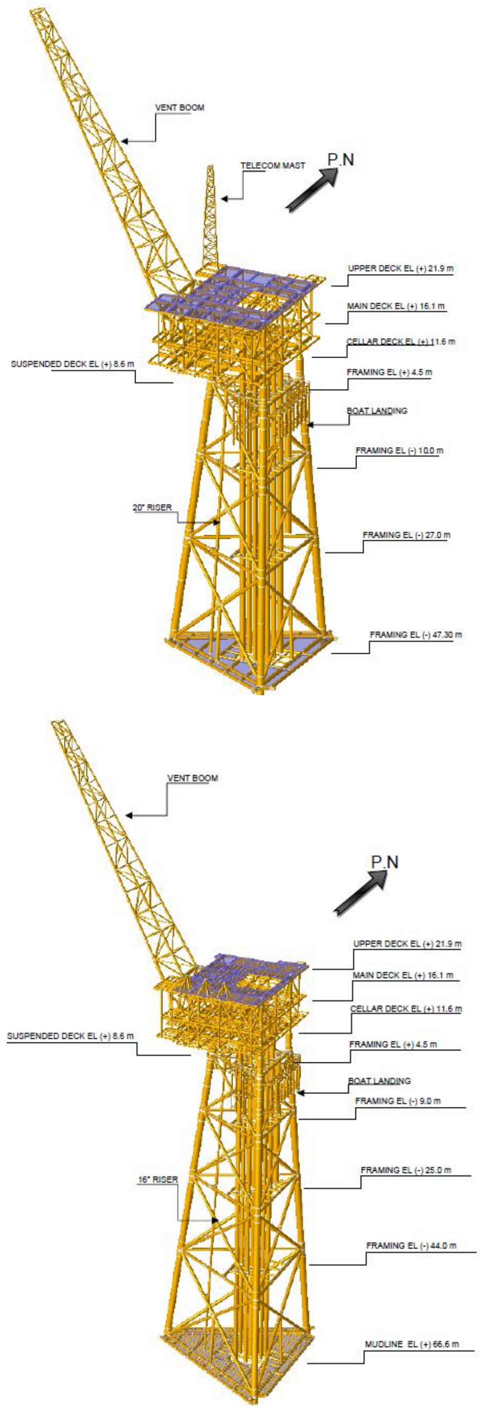

Figure. 7. Tripod models (Taken from SWP-J and WPN-3 Platforms)

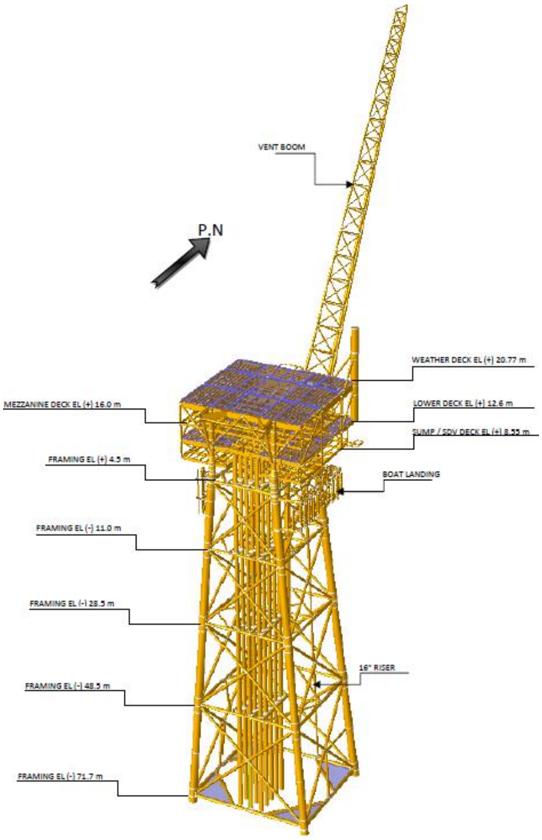

Figure. 8. 4-Legged jacket model (Taken from WPS-2 Platform)

The above models, shown in **Figures 7. and 8.**, study how gravitational and environmental loads (waves, current, and wind) affect the collapse behaviours of ductile structures.

The results of pushover analysis are influenced by several key parameters such as material properties (the modulus of elasticity of material, material yield strength & material ultimate strength), load patterns (the direction and magnitude of the lateral loads applied to a platform), boundary conditions (the foundation stiffness and the degree of fixity can affect the load's distribution and deformations), geometric properties (the structure height, deck area & the number of decks) and damping in the system.

The analysis included two types of loads: functional and environmental. Functional loads include dead weights, live loads, buoyancy, and other gravitational loads. Environmental loads include wave, wind, and current loads. The platforms' models in this research have considered wall thickness surveys due to corrosion, taken from the Annual Platform Inspection Report and Risk-Based Underwater Inspection Reports [49],[50],[51]. In addition to the above, the following assumptions are also considered in the analysis:

1) The structural components did not exhibit significant anomalies, such as excessive pile settlement, cracks, heavy corrosion, etc.
2) No wave-in-deck occurred on the platform.
3) No seabed subsidence surrounding the platform.
4) No flooded jacket braces were found on the platform.
5) Stokes 5th order wave theory is utilised to compute wave loads during Pushover analysis.
6) Scouring is considered in the analysis.
7) All the observed platforms are categorised as API consequence Class L-1.
8) Non-ageing soil is considered in the assessment.
9) No structural damage was found on the jacket platform due to the vessel collision.
10) No cost feasibility analysis is performed, as this research is limited to the structural assessment scope.

The factors used in the application of each load step are as follows:

Gravitational loads = 1.0

Environmental loads (wave, current & wind loads) = incremented until collapse

All the deck members, the appurtenances (boat landing, riser, etc.) and any supporting elements are considered elastic members. Joint flexibility effects are taken into account when performing a pushover analysis. The joint strength check option refers to API LRFD. The local buckling method refers to API Bulletin 2U. Maximum ductility allowed is considered to be 15% for mild steel, with the strain hardening ratio set to 0.002. The strain hardening ratio is defined as the ratio of the slope of the stress-strain curve's plastic portion to the elastic portion's slope.

It is noted that in shallow water where the water depth is smaller than 1,000ft or 305 m as per the DNV code, the most significant factor affecting the structural integrity of the offshore platform is usually derived from wave loading. Engineers, operators and concerned parties must also closely monitor the potential influence of environmental loads, including wave height and gravity, along with material deterioration factors like corrosion allowance, on the offshore structures [52].

The directions considered in the structural analysis are outlined in the API: 8 directions for a rectangular platform and 12 directions for a tripod. The collapse assessment must consider all the directions to generate the hazard curves. The annual probability of platform collapse is calculated from the sum of the directional probabilities of platform collapse.

The loadings are calculated to obtain the maximum base shear to determine the lowest substructure capacity. The wave loading is calculated using the appropriate wave theory, categorised as stream function wave theory; refer to the API-RP-2AWSD. The assessment incorporates wave kinematics and current blockage factors as specified in API-RP-2A WSD. Wind and current loadings are applied in the platforms aligned with the wave direction.

TABLE 2.
100-YEAR RETURN PERIOD (RP) MET OCEAN DATA

Description	Tripod-1 (SWP-J)	Tripod-2 (WPN-3)	4-Legged Platform (WPS-2)
Wave Height (m)			
H_{max}	6.20	6.60	6.60
Wave Period (s)			
T_{ass}	8.60	8.70	7.70
Associated Current (m/s)			
Surface	1.35	1.65	1.65
Mid-depth	0.90	1.50	1.50
Mudline	0.75	1.00	1.00
Wind Speed (m/s)			
Squall 1 minute	26.30	26.30	20.8
Water Level (m)			
Admiralty Chart Datum, ACD	47.30	66.60	71.90
Mean Sea Level (MSL)	1.10	1.10	1.10
Lowest Astronomical Tide, LAT	0.48	0.48	0.48
Surge Height for 100-year RP (m)	0.50	0.50	0.50

The jacket platform collapse capacity is calculated where the 100-year conditions have been used to estimate the directional collapse capacities and the corresponding probability of failure in the given direction. The pushover or non-linear collapse analysis results will provide the platform capacity in terms of reserve strength ratio (RSR) and collapse base shear of the platform for the corresponding met ocean data of a 100-year return period taken from the Structural Design Basis [48] as tabulated in **Table 2.**, above.

The assessments of the observed platforms are performed in two phases as follows:

A. Phase-1: Model and run pushover analysis of the existing platforms to get the collapse capacities.
   1. Model 2 (two) tripod platforms with a water depth of 47.30 and 66.60 meters, respectively, and apply 6.20 and 6.60-meter wave height under a 100-year storm to 12 directions.
   2. Model a 4-legged platform with a water depth of 71.90 meters and apply a 6.60-meter wave height under a 100-year storm in 8 directions.
   3. Run pushover analyses for the platform models mentioned in points 1 and 2 to get the critical base shear collapse from the most critical direction of environmental loads.
   4. Run pushover analyses for the platform models mentioned in points 1 and 2, using the most critical direction of environmental loads with varying pile depth below the mudline starting from 75% until 100% of the original pile depth (for sensitivity study purposes), with the following scenarios:
      a. As-is condition (doing nothing);
      b. Reducing or adding the loads on the topside described as Load Intensity (LI = (Topside Weight / Water-Depth^2). The live loads on the topside are to be reduced and increased by 50% and 75% from the original design weight;
      c. Removing marine growth (Marine growth to be removed from MSL down to 50% and 75% of the water depth);
      d. Strengthen the jacket by applying a soldier pile attached to the critical jacket leg to increase the capacity of the platform foundation.
   5. Determine platform base shear for each scenario.
   6. Determine collapse base shear and RSR for each scenario.

B. Phase-2: Performing simple reliability assessment related to pile compression failure.
   1. Perform a simple reliability assessment by implementing a simple safety margin formulation on the conservative assumption that structures are statically determinate or cannot redistribute load between piles.
   2. Carry out the integration of probabilities over all possible wave heights to obtain the overall cumulative probability of failure.
   3. Justify a suitable approach to extending the platform life based on the RSR and PoF associated with the above scenarios, as described in point A.4 of Phase-1.

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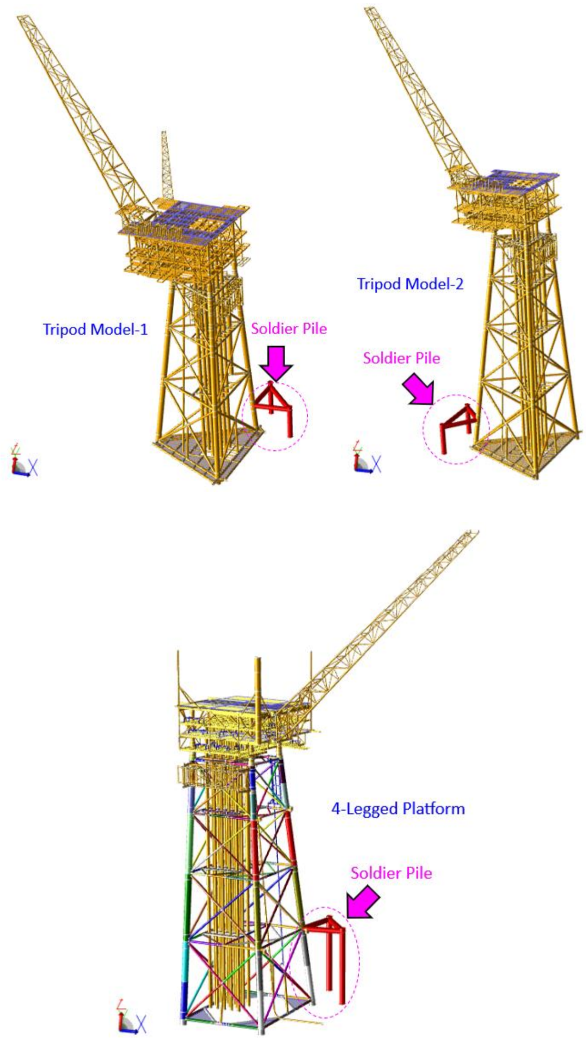

Figure. 9. Jacket strengthening for the tripods and 4-leg platform models

It is noted that the location of new piles (soldier piles) being installed at the jackets is to counter the maximum base shear and overturning resulting from the critical environmental loads, which are dominant among the other directions.

C. Acceptance Criteria

The maximum annual failure probability criteria used in the study for the acceptance criteria based on exposure category on the ageing jacket platforms as per the API RP 2SIM 1st Edition, Nov 2014 for API RP2A-WSD 22nd Design Edition and Later are described in **Table 3.** as follows:

TABLE 3.
ACCEPTANCE CRITERIA BASED ON EXPOSURE CATEGORY

Exposure Category	Ultimate Strength Metocean Criteria
L-1 • Manned evacuated with high consequence • Unmanned evacuated with high consequence	PoF $\leq 1 \times 10^{-3}$ (RP $\geq$ 1000 years)
S-1 Manned evacuated	PoF $\leq 2 \times 10^{-3}$ (RP $\geq$ 500 years)
C-2 Medium consequence	PoF $\leq 2 \times 10^{-3}$ (RP $\geq$ 500 years)
L-3 Unmanned – Low consequence	PoF $\leq 1 \times 10^{-2}$ (RP $\geq$ 100 years)

The observed platforms for tripods and 4-leg jacket platforms are in the L-1 exposure category, unmanned evacuated with high consequence; therefore, the target reliability level is assumed to be 1×10^{-3}/year, equivalent to a return period of 1000 years. The pushover failure criteria used in this research are described in **Table 4**. Once any criteria are exceeded, the pushover analysis will be terminated, and base shear collapse will be taken as the base shear.

TABLE 4.
FAILURE CRITERIA

Item	Criteria
Rupture strain	> 15% strain (S355 steel) > 20% strain (S235 steel)
Soil failure	> 200mm vertical pile slippage

The estimated PoF due to hazards posed by extreme storms is calculated using simplified reliability calculation to determine pile failure adopted from SPE 138712 [47]. This exercise involves implementing a simple safety margin formulation, in which the structures are assumed to be unable to redistribute load between their piles. The calculated probability of failure can then be compared directly against the target reliability.

Identifying and evaluating potential mitigation actions for extending the life of the ageing offshore jacket platform is the most effective way to make valuable decisions. **Figure 10** describes a flow chart of the structural analysis process.

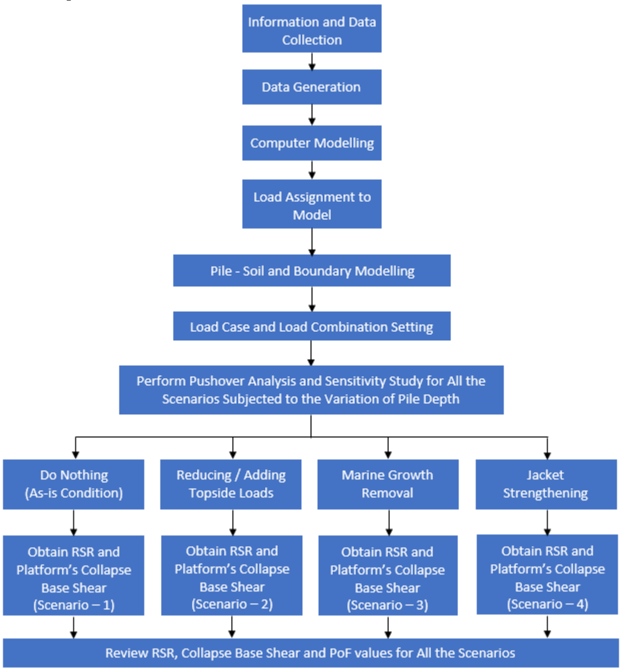

Figure. 10. Flow chart of the structural analyses process

For sensitivity analysis, several variables represented in the mitigation strategies, which are subjected to the variation of pile depths described in **Figure 10**, are then compared and selected as the most sensible and effective practical way to improve the RSR value.

IV. RESULTS AND DISCUSSION

The performed collapse analysis has obtained the outputs of collapse base shear collapse RSR values for all the cases occurring on the observed platforms with a variation of pile depth percentage from their original design, as shown in **Tables 5. to 16.** and **Figures 11. to 19**.

TABLE 5.
DO NOTHING (AS-IS CONDITION)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Minimum Criteria as per the API RP 2A Requirement		Inplace Analysis	Do Nothing (As-is Condition)				
			Hw m	Tp Sec.		FoS	RSR	Min Pile FoS	RSR	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	1.50	1.60	1.01	1.90	6,612	674	-	0.93
					80			1.13	1.80	6,488	661	-	
					85			1.25	1.95	7,207	735	-	
					90			1.37	1.90	7,055	719	-	
					95			1.49	1.80	6,449	657	-	
					100			1.56	1.80	6,477	660	-	
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	1.50	1.60	1.03	0.80	4,693	478	-	0.27
					80			1.11	1.00	4,985	508	-	
					85			1.17	1.10	6,465	659	-	
					90			1.34	1.20	7,064	720	-	
					95			1.46	1.30	7,655	780	-	
					100			1.54	1.50	8,720	889	-	
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	1.50	1.60	1.10	2.00	8,613	878	-	0.78
					80			1.19	2.40	10,398	1,060	-	
					85			1.31	2.80	12,132	1,237	-	
					90			1.50	3.30	14,148	1,442	-	
					95			1.61	3.80	16,429	1,675	-	
					100			1.72	3.90	16,755	1,708	-	

TABLE 6.
DO NOTHING (AS-IS CONDITION) AND EVENTS

Type of Structure	Topside Weight MT	Water Depth m	100-year wave Hw m	100-year wave Tp Sec.	Percentage from the original Pile Depth	As-is Condition: Event
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	Local buckling on jacket leg (1120-2120), pile nearing punch thru and pile plastic at Piles (1120 and 11
					80	Local buckling on jacket leg (1120-2120) and pile plastic at Pile (1190)
					85	Local buckling on jacket leg (1120-2120) and pile plastic at Pile (1190)
					90	Local buckling on jacket leg (1120-2120) and pile plastic at Piles (1120 and 1190)
					95	Local buckling on jacket leg (1120-2120) and pile plastic at Pile (1120)
					100	Local buckling on jacket leg (1120-2120) and pile plastic at Piles (1120 and 1190)
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	Pile nearing punch thru and pile plastic at Pile (1190)
					80	Pile nearing punch thru and pile plastic at Pile (1190)
					85	Pile nearing punch thru and pile plastic at Pile (1190)
					90	Pile nearing punch thru and pile plastic at Pile (1190)
					95	Pile nearing punch thru and pile plastic at Pile (1190)
					100	Pile nearing punch thru and pile plastic at Pile (1190)
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	Pile nearing punch thru at Piles (111P and 112P)
					80	Local buckling and hinged at Pile (112P-212P) and pile nearing punch thru at Pile (111P)
					85	Pile nearing punch thru and pile plastic at Piles (111P and 112P)
					90	Pile nearing punch thru and pile plastic at Piles (111P and 112P)
					95	Pile nearing punch thru at Piles (111P and 112P)
					100	Local buckling and hinged at Pile (112P-212P) and pile nearing punch thru at pile (111P)

For the Do-Nothing or As-is condition shown in **Table 6.**, most events happened until the platform structures collapsed because the pile was nearing the punch trough.

TABLE 7.
TOPSIDE WEIGHT REDUCTION (50% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action: Topside Weight Reduction (50% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Topside Weight Reduction MT	RSR	% Increase	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Average % Increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	66	1.85	-2.63%	6,590	672	-0.34%	2.98%	0.91
					80	66	1.90	5.56%	7,087	722	9.24%		
					85	66	1.90	-2.56%	7,121	726	-1.19%		
					90	66	1.90	0.00%	6,883	702	-2.44%		
					95	66	1.90	5.56%	6,973	711	8.12%		
					100	66	1.90	5.56%	6,766	690	4.47%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	302	1.00	25.00%	5,082	518	8.29%	7.33%	0.20
					80	302	1.00	0.00%	5,632	574	12.98%		
					85	302	1.10	0.00%	6,477	660	0.18%		
					90	302	1.30	8.33%	8,313	847	17.69%		
					95	302	1.35	3.85%	7,947	810	3.81%		
					100	302	1.50	0.00%	8,810	898	1.04%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	74	1.95	-2.50%	8,316	848	-3.45%	-2.01%	0.77
					80	74	2.35	-2.08%	10,172	1,037	-2.17%		
					85	74	2.75	-1.79%	11,913	1,214	-1.80%		
					90	74	3.20	-3.03%	13,862	1,413	-2.03%		
					95	74	3.75	-1.32%	16,072	1,638	-2.17%		
					100	74	3.90	0.00%	16,679	1,700	-0.45%		

TABLE 8.
TOPSIDE WEIGHT REDUCTION (75% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action: Topside Weight Reduction (75% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Topside Weight Reduction MT	RSR	% Increase	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Average % Increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	99	1.85	-2.63%	6,439	656	-2.62%	0.22%	0.89
					80	99	1.90	5.56%	6,979	711	7.57%		
					85	99	1.90	-2.56%	7,015	715	-2.66%		
					90	99	1.90	0.00%	6,901	703	-2.18%		
					95	99	1.80	0.00%	6,503	663	0.83%		
					100	99	1.80	0.00%	6,500	663	0.36%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	452	0.80	0.00%	4,711	480	0.38%	3.47%	0.17
					80	452	1.00	0.00%	5,537	564	11.09%		
					85	452	1.10	0.00%	6,479	660	0.22%		
					90	452	1.25	4.17%	7,347	749	4.01%		
					95	452	1.35	3.85%	7,952	811	3.87%		
					100	452	1.50	0.00%	8,827	900	1.23%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	111	1.95	-2.50%	8,431	859	-2.11%	-1.19%	0.76
					80	111	2.35	-2.08%	10,185	1,038	-2.05%		
					85	111	2.80	0.00%	12,038	1,227	-0.77%		
					90	111	3.25	-1.52%	14,065	1,434	-0.59%		
					95	111	3.75	-1.32%	16,236	1,655	-1.17%		
					100	111	3.90	0.00%	16,686	1,701	-0.41%		

TABLE 9.
ADDITIONAL TOPSIDE WEIGHT (50% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action – Addition of Topside Weight (50% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Addition of Topside Weight MT	RSR	% Increase	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Average % Increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	66	1.80	-5.26%	6,605	673	-0.10%	-1.73%	0.96
					80	66	1.80	0.00%	6,420	654	-1.05%		
					85	66	1.80	-7.69%	6,463	659	-10.33%		
					90	66	1.85	-2.63%	7,127	726	1.02%		
					95	66	1.80	0.00%	6,514	664	1.02%		
					100	66	1.80	0.00%	6,414	654	-0.97%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	302	0.80	0.00%	4,250	433	-9.45%	-3.78%	0.34
					80	302	0.80	-20.00%	4,704	480	-5.63%		
					85	302	1.05	-4.55%	6,179	630	-4.43%		
					90	302	1.20	0.00%	6,997	713	-0.95%		
					95	302	1.30	0.00%	7,650	780	-0.07%		
					100	302	1.45	-3.33%	8,531	870	-2.17%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	74	1.85	-7.50%	7,973	813	-7.43%	-4.60%	0.80
					80	74	2.25	-6.25%	9,743	993	-6.29%		
					85	74	2.65	-5.36%	11,476	1,170	-5.41%		
					90	74	3.15	-4.55%	13,569	1,383	-4.10%		
					95	74	3.65	-3.95%	15,794	1,610	-3.87%		
					100	74	3.85	-1.28%	16,671	1,699	-0.50%		

TABLE 10.
TOPSIDE WEIGHT REDUCTION (75% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action – Topside Weight Reduction (75% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Topside Weight Reduction MT	RSR	% Increase	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Average % Increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	99	1.85	-2.63%	6,439	656	-2.62%	0.22%	0.89
					80	99	1.90	5.56%	6,979	711	7.57%		
					85	99	1.90	-2.56%	7,015	715	-2.66%		
					90	99	1.90	0.00%	6,901	703	-2.18%		
					95	99	1.80	0.00%	6,503	663	0.83%		
					100	99	1.80	0.00%	6,500	663	0.36%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	452	0.80	0.00%	4,711	480	0.38%	3.47%	0.17
					80	452	1.00	0.00%	5,537	564	11.09%		
					85	452	1.10	0.00%	6,479	660	0.22%		
					90	452	1.25	4.17%	7,347	749	4.01%		
					95	452	1.35	3.85%	7,952	811	3.87%		
					100	452	1.50	0.00%	8,827	900	1.23%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	111	1.95	-2.50%	8,431	859	-2.11%	-1.19%	0.76
					80	111	2.35	-2.08%	10,185	1,038	-2.05%		
					85	111	2.80	0.00%	12,038	1,227	-0.77%		
					90	111	3.25	-1.52%	14,065	1,434	-0.59%		
					95	111	3.75	-1.32%	16,236	1,655	-1.17%		
					100	111	3.90	0.00%	16,686	1,701	-0.41%		

TABLE 11.
ADDITIONAL TOPSIDE WEIGHT (50% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action – Addition of Topside Weight (50% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Addition of Topside Weight MT	RSR	% Increase	Base Shear Collapse kN	Base Shear Collapse MT	% Increase	Average % Increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	66	1.80	-5.26%	6,605	673	-0.10%	-1.73%	0.96
					80	66	1.80	0.00%	6,420	654	-1.05%		
					85	66	1.80	-7.69%	6,463	659	-10.33%		
					90	66	1.85	-2.63%	7,127	726	1.02%		
					95	66	1.80	0.00%	6,514	664	1.02%		
					100	66	1.80	0.00%	6,414	654	-0.97%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	302	0.80	0.00%	4,250	433	-9.45%	-3.78%	0.34
					80	302	0.80	-20.00%	4,704	480	-5.63%		
					85	302	1.05	-4.55%	6,179	630	-4.43%		
					90	302	1.20	0.00%	6,997	713	-0.95%		
					95	302	1.30	0.00%	7,650	780	-0.07%		
					100	302	1.45	-3.33%	8,531	870	-2.17%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	74	1.85	-7.50%	7,973	813	-7.43%	-4.60%	0.80
					80	74	2.25	-6.25%	9,743	993	-6.29%		
					85	74	2.65	-5.36%	11,476	1,170	-5.41%		
					90	74	3.15	-4.55%	13,569	1,383	-4.10%		
					95	74	3.65	-3.95%	15,794	1,610	-3.87%		
					100	74	3.85	-1.28%	16,671	1,699	-0.50%		

As shown in **Tables 5. and 7., 8., until Table 15.,** the load intensity for Tripod-2 is less than that for the other platform, which can be categorised as a wave-dominated platform structure. The probability of failure for this platform will then be checked using a simple reliability assessment related to pile compression failure.

TABLE 12.
ADDITIONAL TOPSIDE WEIGHT (75% FROM ORIGINAL DESIGN)

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action							
						Addition of Topside Weight (75% from Original Design of Live Load on Topside)							
			Hw m	Tp Sec.		Additional Topside Weight MT	RSR	% increase	Base Shear Collapse kN	Base Shear Collapse MT	% increase	Average % increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	99	1.80	-5.26%	6,573	670	-0.60%	-3.49%	0.98
					80	99	1.80	0.00%	6,442	657	-0.70%		
					85	99	1.80	-7.69%	6,434	656	-10.72%		
					90	99	1.80	-5.26%	6,447	657	-8.62%		
					95	99	1.80	0.00%	6,399	652	-0.77%		
					100	99	1.80	0.00%	6,508	663	0.48%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	452.00	0.80	0.00%	4,270	435	-9.01%	-1.97%	0.37
					80	452.00	1.00	0.00%	5,411	552	8.56%		
					85	452.00	1.05	-4.55%	6,176	630	-4.48%		
					90	452.00	1.15	-4.17%	6,769	690	-4.18%		
					95	452.00	1.30	0.00%	7,644	779	-0.14%		
					100	452.00	1.45	-3.33%	8,496	866	-2.57%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	111	1.80	-10.00%	7,791	794	-9.54%	-5.28%	0.81
					80	111	2.25	-6.25%	9,722	991	-6.50%		
					85	111	2.65	-5.36%	11,467	1,169	-5.48%		
					90	111	3.10	-6.06%	13,422	1,368	-5.14%		
					95	111	3.65	-3.95%	15,684	1,599	-4.54%		
					100	111	3.85	-1.28%	16,669	1,699	-0.51%		

TABLE 13.
MARINE GROWTH REMOVAL 50% OF WATER DEPTH

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action						
						Marine Growth Removal 50% of Water Depth						
			Hw m	Tp Sec.		RSR	% increase	Base Shear Collapse kN	Base Shear Collapse MT	% increase	Average % increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	2.25	18.42%	6,552	668	-0.90%	4.15%	0.93
					80	2.30	27.78%	7,012	715	8.08%		
					85	2.30	17.95%	7,175	731	-0.45%		
					90	2.35	23.68%	7,505	765	6.39%		
					95	2.30	27.78%	6,969	710	8.06%		
					100	2.25	25.00%	6,718	685	3.72%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	1.00	25.00%	4,549	464	-3.06%	4.60%	0.27
					80	1.15	15.00%	5,718	583	14.71%		
					85	1.40	27.27%	6,960	709	7.65%		
					90	1.45	20.83%	7,212	735	2.10%		
					95	1.60	23.08%	7,956	811	3.92%		
					100	1.80	20.00%	8,919	909	2.29%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	2.30	15.00%	8,408	857	-2.39%	-5.07%	0.78
					80	2.80	16.67%	10,246	1,044	-1.46%		
					85	3.30	17.86%	12,074	1,231	-0.47%		
					90	3.85	16.67%	14,086	1,436	-0.44%		
					95	3.95	3.95%	14,457	1,474	-12.00%		
					100	3.95	1.28%	14,462	1,474	-13.68%		

TABLE 14.
MARINE GROWTH REMOVAL 75% OF WATER DEPTH

Type of Structure	Topside Weight MT	Water Depth m	100-year wave		Percentage from the original Pile Depth	Mitigation Action						
						Marine Growth Removal 75% of Water Depth						
			Hw m	Tp Sec.		RSR	% increase	Base Shear Collapse kN	Base Shear Collapse MT	% increase	Average % increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	2.25	18.42%	6,548	668	-0.97%	5.32%	0.93
					80	2.30	27.78%	7,576	772	16.78%		
					85	2.30	17.95%	7,051	719	-2.17%		
					90	2.30	21.05%	7,116	725	0.87%		
					95	2.30	27.78%	6,843	698	6.10%		
					100	2.30	27.78%	7,207	735	11.28%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	1.00	25.00%	4,499	459	-4.14%	3.50%	0.27
					80	1.15	15.00%	5,685	580	14.06%		
					85	1.40	27.27%	6,876	701	6.36%		
					90	1.45	20.83%	7,173	731	1.54%		
					95	1.60	23.08%	7,915	807	3.39%		
					100	1.85	23.33%	8,703	887	-0.19%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	2.30	15.00%	8,368	853	-2.84%	-2.99%	0.78
					80	2.85	18.75%	10,315	1,052	-0.79%		
					85	3.30	17.86%	12,008	1,224	-1.02%		
					90	3.90	18.18%	14,119	1,439	-0.21%		
					95	3.95	3.95%	14,377	1,466	-12.49%		
					100	3.60	-7.69%	16,656	1,698	-0.59%		

TABLE 15.
JACKET STRENGTHENING

Type of Structure	Topside Weight MT	Water Depth m	100-year wave Hw m	100-year wave Tp Sec.	Percentage from the original Pile Depth	Mitigation Action – Jacket Strengthening: RSR	% increase	Base Shear Collapse kN	Base Shear Collapse MT	% increase	Average % increase of Collapse BS	Load Intensity TW/WD^2
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	2.00	5.26%	6,913	705	4.55%	11.18%	0.93
					80	2.00	11.11%	6,887	702	6.15%		
					85	2.00	2.56%	6,932	707	-3.82%		
					90	2.00	5.26%	6,951	709	-1.46%		
					95	2.00	11.11%	6,976	711	8.17%		
					100	2.95	63.89%	9,942	1,014	53.51%		
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	1.15	43.75%	5,158	526	9.91%	18.97%	0.27
					80	1.20	20.00%	7,174	731	43.93%		
					85	1.25	13.64%	7,423	757	14.82%		
					90	1.30	8.33%	7,709	786	9.14%		
					95	1.50	15.38%	9,210	939	20.31%		
					100	1.65	10.00%	10,089	1,028	15.71%		
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	2.05	2.50%	9,078	925	5.40%	7.17%	0.78
					80	2.35	-2.08%	10,440	1,064	0.40%		
					85	2.90	3.57%	15,414	1,571	27.06%		
					90	3.30	0.00%	14,678	1,496	3.75%		
					95	3.85	1.32%	16,620	1,694	1.16%		
					100	3.85	-1.28%	17,638	1,798	5.27%		

TABLE 16.
JACKET STRENGTHENING WITH EVENTS

Type of Structure	Topside Weight MT	Water Depth m	100-year wave Hw m	100-year wave Tp Sec.	Percentage from the original Pile Depth	Mitigation Action – Jacket Strengthening: Event
Tripod-1 (SWP-J)	2,091	47.30	6.20	7.60	75	Local buckling on jacket leg (1102-2102), pile plastic at Pile (1190) and hinged at soldier pile member (SPX3-SPX4)
					80	Local buckling on jacket leg (1102-2102), pile plastic at Pile (1190) and hinged at soldier pile member (SPX3-SPX4)
					85	Local buckling on jacket leg (1102-2102), pile plastic at Pile (1190) and hinged at soldier pile member (SPX3-SPX4)
					90	Local buckling on jacket leg (1102-2102), pile plastic at Pile (1190) and hinged at soldier pile member (SPX3-SPX4)
					95	Local buckling on jacket leg (1102-2102), pile plastic at Pile (1190) and hinged at soldier pile member (SPX3-SPX4)
					100	Local Buckling on soldier pile member (SPX3-SPX4) and hinged at jacket leg (1119-SP02)
Tripod-2 (WPN-3)	1,186	66.60	6.60	7.70	75	Joint failure at jacket joint (SP01), pile nearing punch thru and pile plastic at Pile (1190)
					80	Joint failure at jacket joint (SP05), pile nearing punch thru and pile plastic at Pile (1190)
					85	Local buckling on soldier pile members (SP01-2189) & (SP01-2189) and pile plastic at Pile (1190)
					90	Pile nearing punch thru and pile plastic at Pile (1190)
					95	Joint failure at jacket joint (SP05), pile nearing punch thru and pile plastic at Pile (1190)
					100	Joint failure at jacket joint (SP05), pile nearing punch thru and pile plastic at Pile (1190)
4-legged (WPS-2)	4,031	71.70	6.60	8.70	75	Pile nearing punch thru at piles (111P and 112P) and pile plastic at Piles (111P and 112P)
					80	Pile nearing punch thru at piles (111P and 112P) and pile plastic at Piles (111P and 112P)
					85	Pile nearing punch thru at piles (111P and 112P) and pile plastic at Piles (111P and 112P)
					90	Pile nearing punch thru at piles (111P and 112P) and pile plastic at Pile (112P)
					95	Pile nearing punch thru at piles (111P and 112P) and pile plastic at Piles (112P and 114P)
					100	Plastic member at soldier pile (SPX1-SPX2) and pile plastic at Piles (112P)

The outputs of the pushover analysis of tripods and a 4-legged jacket platform are plotted in the relationship between the collapse base shears, RSR, and increased RSR values versus pile depths for all the scenarios in **Figures 11.** to **19.** most events happened until the platform structures collapsed because the pile was plastic and nearing the punch trough.

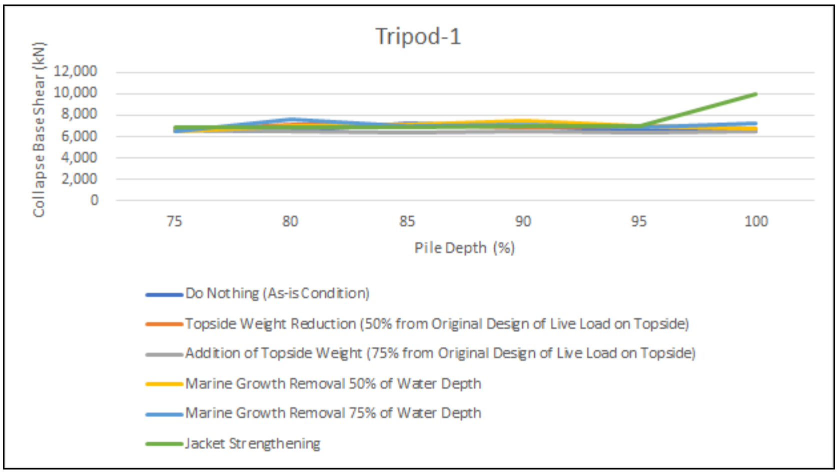

Figure. 11. Collapse Base shear versus pile depth (%) for Tripod-1

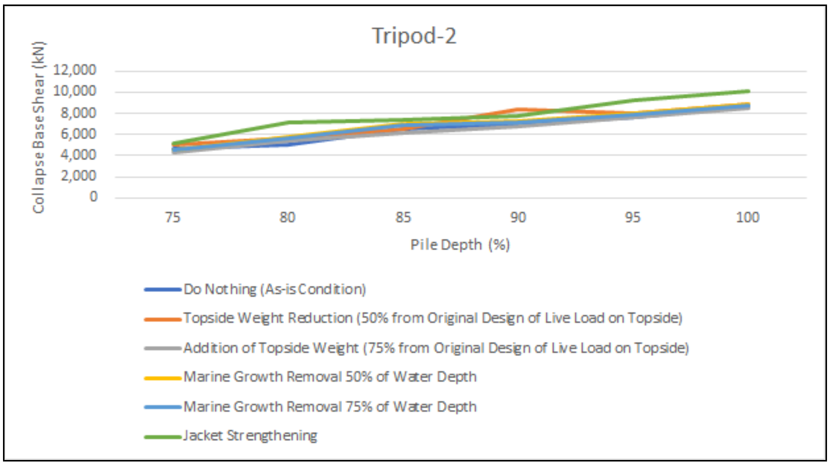

Figure. 12. Collapse Base shear versus pile depth (%) for Tripod-2

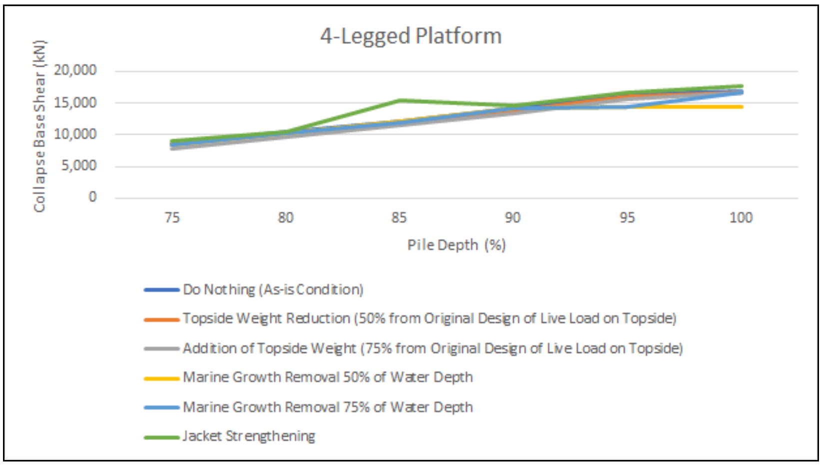

Figure. 13. Collapse Base shear versus pile depth (%) for 4-legged platform

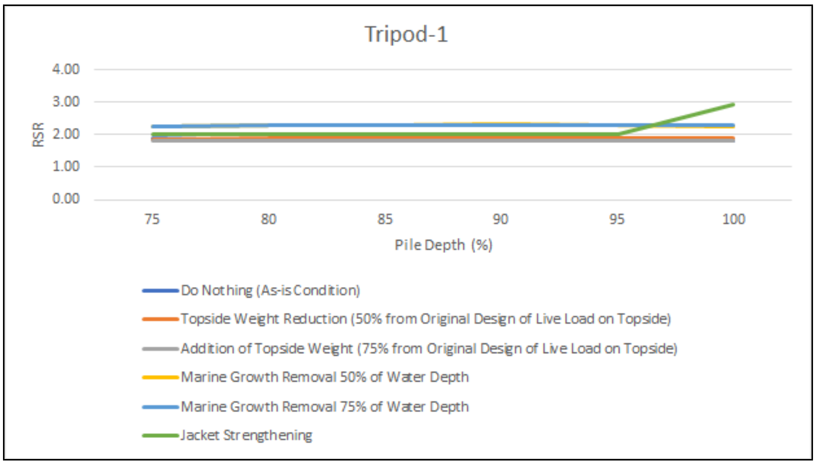

Figure. 14. RSR versus pile depth (%) for Tripod-1

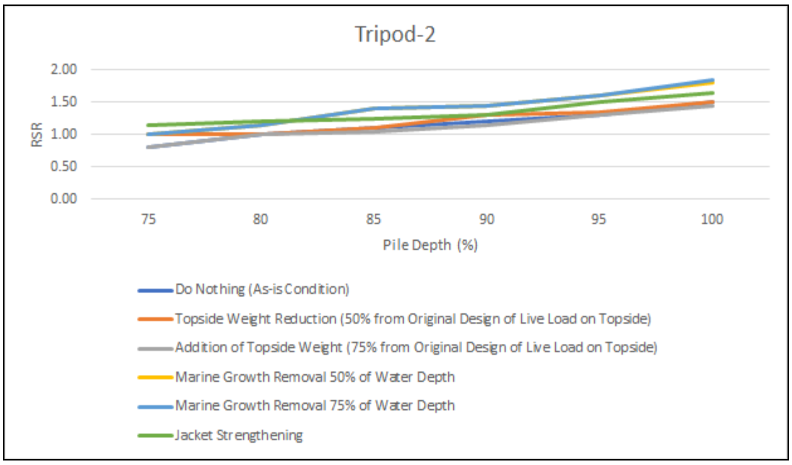

Figure. 15. RSR versus pile depth (%) for Tripod-2

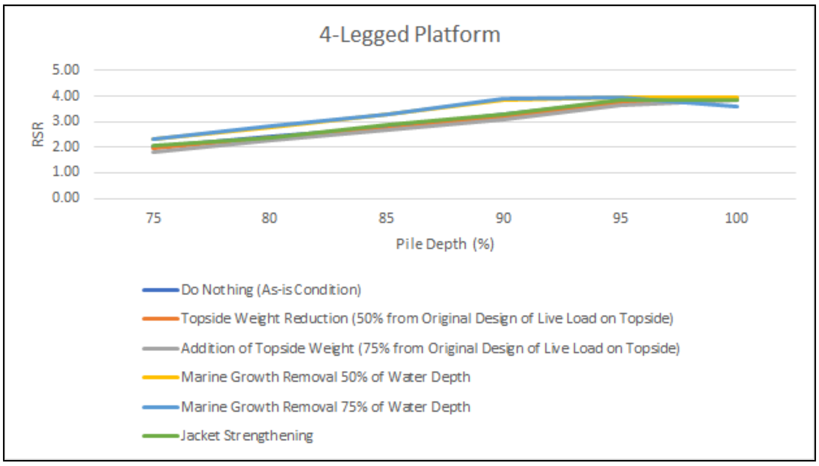

Figure. 16. RSR versus pile depth (%) for 4-legged platform

Figures 11. to **16.** above illustrate that soil stiffness, as indicated by pile depth, is the main factor driving the gradual increase in RSR values and collapse base shears for all the scenarios. Additionally, the jacket strengthening scenario results in the highest RSR values and collapse base shear across all variations of pile depths. Consequently, this scenario is more advantageous for platforms, even those with limited pile capacity.

The above results have confirmed that the interaction of pile and soil is significant in determining the ultimate capacity of the platform (Ranjbar & Malayjerdi, 2015).

The RSR values for Tripod-1 and the 4-legged platform meet the ultimate level state (ULS) requirements, as shown in **Figures 14** and **16**. However, Tripod-2 with the original pile design is slightly below the target RSR, as shown in **Figure 15**.

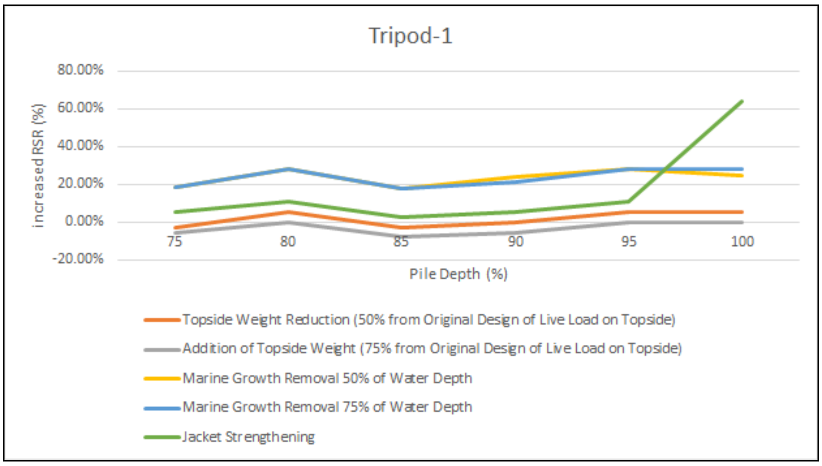

Figure. 17. Increased RSR versus pile depth (%) for Tripod-1.

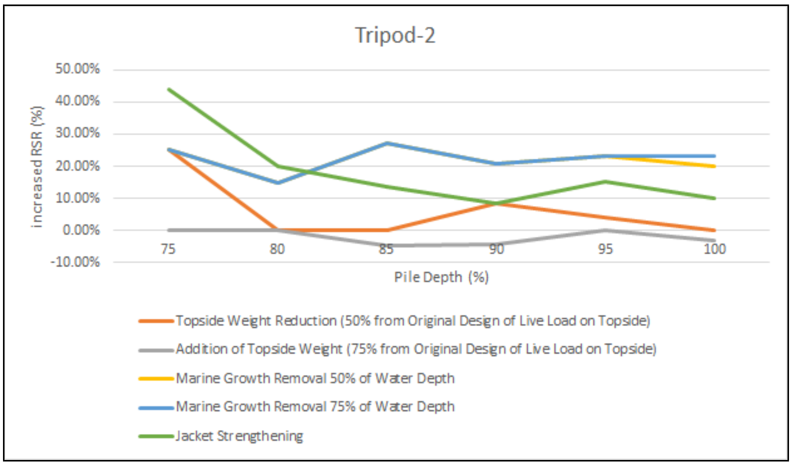

Figure. 18. Increased RSR versus pile depth (%) for Tripod-2

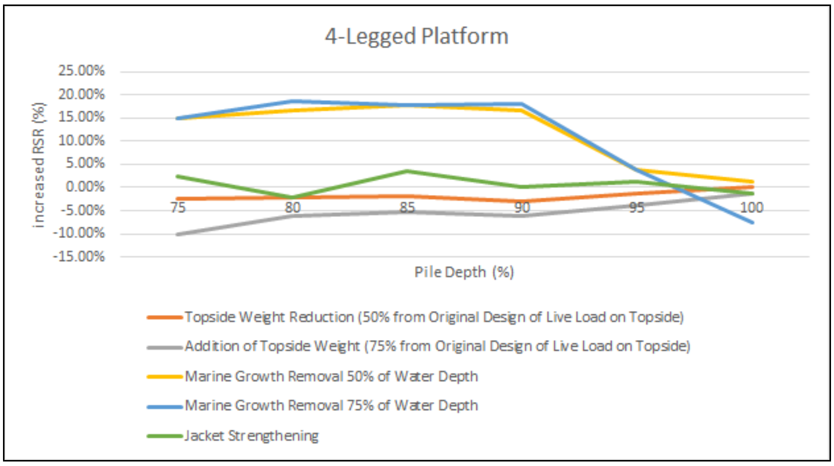

Figure. 19. Increased RSR versus pile depth (%) for 4-legged platform

Figure 17 to **Figure 19** show that, in the case of topside weight reduction, RSR values increase insignificantly compared to the as-is condition for Tripod-1 and 4-legged platform models with variations in pile depths. However, for Tripod-2, RSR values significantly increase compared to the as-is condition for some pile depth.

In the case of additional loads on the topside, most of the RSR values decrease compared to the as-is condition for all models with variations in pile depths.

For marine growth removal and jacket strengthening, most RSR values increase compared to the as-is condition for all models with variations in pile depths.

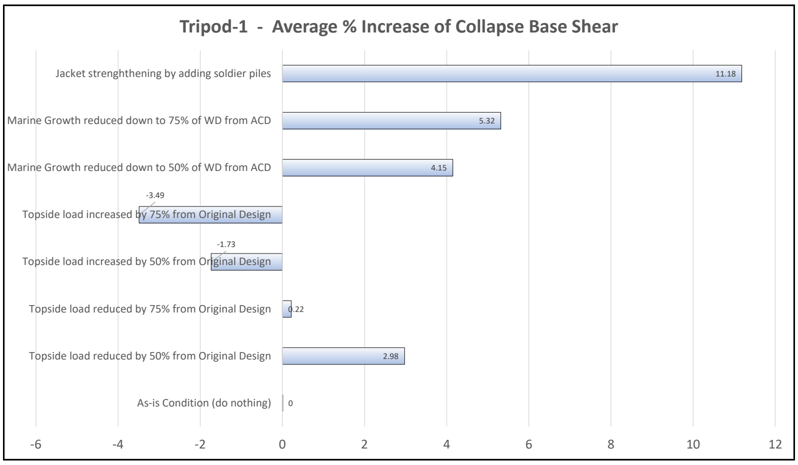

Figure. 20. Tripod-1 - Average % Increase of Collapse Base Shear

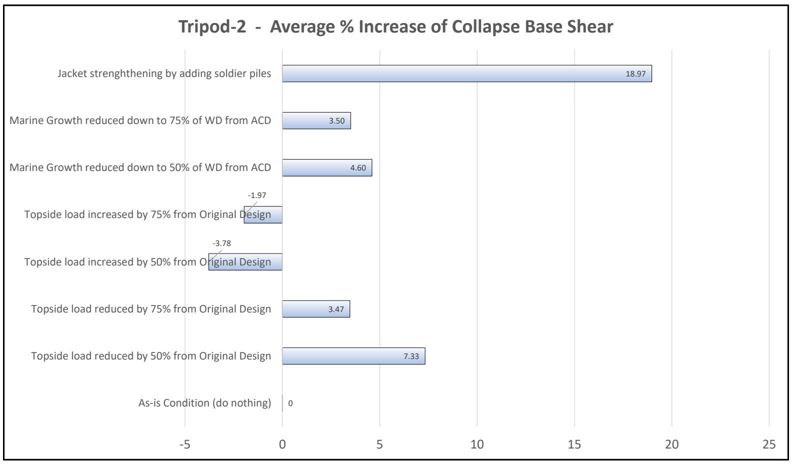

Figure. 21. Tripod-2 - Average % Increase of Collapse Base Shear

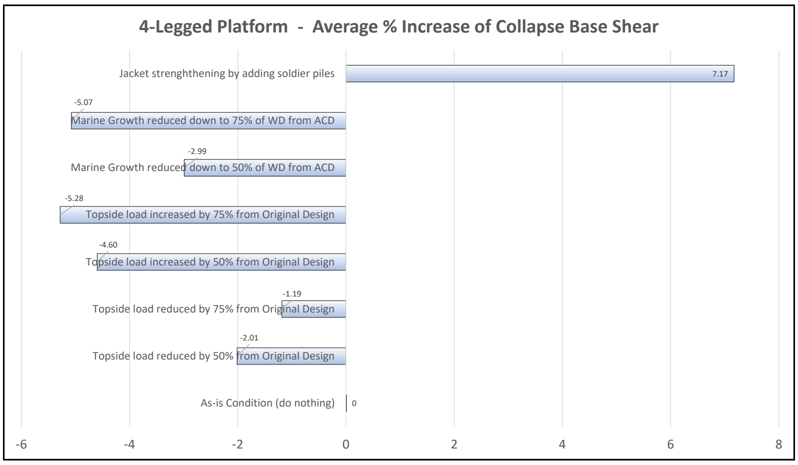

Figure. 22. 4-Legged Platform - Average % Increase of Collapse Base Shear

Figures 20 to 22 show that, in the case of jacket strengthening, most collapse base shears increase compared to the as-is condition across all models with variations in pile depths.

The tripod-2 model has been chosen for reliability assessment since it is a wave-dominated platform. A simple reliability assessment of pile compression failure for selected cases is performed to obtain the Probability of Failure (PoF). The results are compared with the target of reliability per the platform categorisation. For this research, the observed platforms are categorised as L-1 criteria with a Probability of Failure (PoF) of 1x10^{-3} corresponding to a 1000-year return period. The probability density function (Pdf) for Tripod-2 is shown in **Figure 23**. **Tables 17.** and **18.** show the cumulative annual probability of failure for one of the six pile depth cases for each as-is condition and jacket strengthening for Tripod -2 (WPN-3 platform). From the tables, the platform's corresponding return period is higher for the jacket strengthening case and meets the L-1 API consequence class criteria.

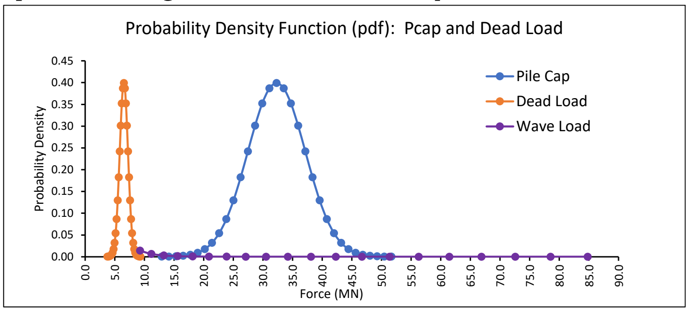

Figure. 23. Probability Density Function (PDF) for Tripod-2

TABLE 17.
CUMULATIVE ANNUAL PROBABILITY OF FAILURE AND CORRESPONDING RETURN PERIOD FOR TRIPOD-2 (AS-IS CONDITION – 100% PILE DEPTH)

Item	Hmax	Pre	M_μ	M_σ	β	Pof	λHmax	PHme	Delta λ	Overall Cumulative Prob.
1	6.0	9.9109	15.9335	4.9201	3.2384	0.0006	0.0259	0.0256	0.0139	8.38E-06
2	6.5	11.9429	13.9015	4.9314	2.8190	0.0024	0.0117	0.0117	0.0064	1.53E-05
3	7.0	14.1939	11.6505	4.9463	2.3554	0.0093	0.0053	0.0053	0.0029	2.68E-05
4	7.5	16.6692	9.1752	4.9655	1.8478	0.0323	0.0024	0.0024	0.0013	4.23E-05
5	8.0	19.3741	6.4703	4.9900	1.2966	0.0974	0.0011	0.0011	0.0006	5.77E-05
6	8.5	22.3135	3.5309	5.0206	0.7033	0.2409	0.0005	0.0005	0.0003	6.46E-05
7	9.0	25.4922	0.3522	5.0583	0.0696	0.4722	0.0002	0.0002	0.0001	5.72E-05
8	9.5	28.9146	-3.0703	5.1041	-0.6015	0.7263	0.0001	0.0001	0.0001	3.98E-05
9	10.0	32.5853	-6.7409	5.1591	-1.3066	0.9043	0.0000	0.0000	0.0000	2.24E-05
10	10.5	36.5084	-10.6640	5.2244	-2.0412	0.9794	0.0000	0.0000	0.0000	1.10E-05
11	11.0	40.6880	-14.8437	5.3010	-2.8002	0.9974	0.0000	0.0000	0.0000	5.05E-06
12	11.5	45.1282	-19.2838	5.3901	-3.5776	0.9998	0.0000	0.0000	0.0000	2.29E-06
13	12.0	49.8327	-23.9883	5.4927	-4.3673	1.0000	0.0000	0.0000	0.0000	1.03E-06
14	12.5	54.8053	-28.9609	5.6099	-5.1625	1.0000	0.0000	0.0000	0.0000	4.68E-07
15	13.0	60.0496	-34.2052	5.7425	-5.9565	1.0000	0.0000	0.0000	0.0000	2.11E-07
16	13.5	65.5692	-39.7248	5.8915	-6.7427	1.0000	0.0000	0.0000	0.0000	9.55E-08
17	14.0	71.3675	-45.5231	6.0577	-7.5150	1.0000	0.0000	0.0000	0.0000	4.32E-08
18	14.5	77.4478	-51.6035	6.2416	-8.2677	1.0000	0.0000	0.0000	0.0000	1.95E-08
19	15.0	83.8136	-57.9692	6.4439	-8.9960	1.0000	0.0000	0.0000	0.0000	8.82E-09
20	15.5	90.4679	-64.6236	6.6651	-9.6959	1.0000	0.0000	0.0000	0.0000	7.28E-09
										3.55E-04

Cumulative Annual Probability of Failure = 3.55E-04
Corresponding Return Period = 2,819 years

TABLE 18.
CUMULATIVE ANNUAL PROBABILITY OF FAILURE AND CORRESPONDING RETURN PERIOD FOR TRIPOD-2 (JACKET STRENGTHENING – 100% PILE DEPTH)

Item	Hmax	Pre	M_μ	M_σ	β	Pof	λHmax	PHme	Delta λ	Overall Cumulative Prob.
1	6.0	9.2884	16.4756	4.9183	3.3499	0.0004	0.0259	0.0256	0.0139	5.64E-06
2	6.5	11.1928	14.5712	4.9282	2.9567	0.0016	0.0117	0.0117	0.0064	9.90E-06
3	7.0	13.3023	12.4617	4.9413	2.5220	0.0058	0.0053	0.0053	0.0029	1.69E-05
4	7.5	15.6222	10.1418	4.9582	2.0455	0.0204	0.0024	0.0024	0.0013	2.67E-05
5	8.0	18.1572	7.6068	4.9798	1.5275	0.0633	0.0011	0.0011	0.0006	3.75E-05
6	8.5	20.9120	4.8520	5.0067	0.9691	0.1662	0.0005	0.0005	0.0003	4.46E-05
7	9.0	23.8910	1.8730	5.0399	0.3716	0.3551	0.0002	0.0002	0.0001	4.30E-05
8	9.5	27.0985	-1.3345	5.0803	-0.2627	0.6036	0.0001	0.0001	0.0001	3.31E-05
9	10.0	30.5386	-4.7746	5.1289	-0.9309	0.8241	0.0000	0.0000	0.0000	2.04E-05
10	10.5	34.2153	-8.4513	5.1866	-1.6295	0.9484	0.0000	0.0000	0.0000	1.06E-05
11	11.0	38.1324	-12.3684	5.2544	-2.3539	0.9907	0.0000	0.0000	0.0000	5.01E-06
12	11.5	42.2937	-16.5297	5.3334	-3.0992	0.9990	0.0000	0.0000	0.0000	2.29E-06
13	12.0	46.7027	-20.9387	5.4246	-3.8599	0.9999	0.0000	0.0000	0.0000	1.03E-06
14	12.5	51.3629	-25.5989	5.5289	-4.6300	1.0000	0.0000	0.0000	0.0000	4.68E-07
15	13.0	56.2778	-30.5138	5.6473	-5.4033	1.0000	0.0000	0.0000	0.0000	2.11E-07
16	13.5	61.4507	-35.6867	5.7805	-6.1736	1.0000	0.0000	0.0000	0.0000	9.55E-08
17	14.0	66.8849	-41.1208	5.9294	-6.9351	1.0000	0.0000	0.0000	0.0000	4.32E-08
18	14.5	72.5833	-46.8193	6.0946	-7.6820	1.0000	0.0000	0.0000	0.0000	1.95E-08
19	15.0	78.5493	-52.7853	6.2768	-8.4095	1.0000	0.0000	0.0000	0.0000	8.82E-09
20	15.5	84.7856	-59.0216	6.4765	-9.1132	1.0000	0.0000	0.0000	0.0000	7.28E-09
										2.58E-04

Cumulative Annual Probability of Failure = 2.58E-04
Corresponding Return Period = 3,882 years

Figures 24 to 29 show the PoF and Return Period for the variation of pile depth for the As-is condition and Jacket Strengthening, along with the percentage reduction of PoF and increment of Return Period.

For the jacket strengthening scenario on the Tripod-2 platform, the cumulative annual Probability of Failure (PoF) for the as-is condition is 3.55×10^{-4}, corresponding to a return period of 2,819 years for a pile depth of 126 m (100% pile depth). For jacket strengthening, the PoF is 2.58×10^{-4}, corresponding to a return period of 3,882 years for the same pile depth. The PoF of the Jacket strengthening scenario is reduced by about 27%, and the return period is increased by about 38%.

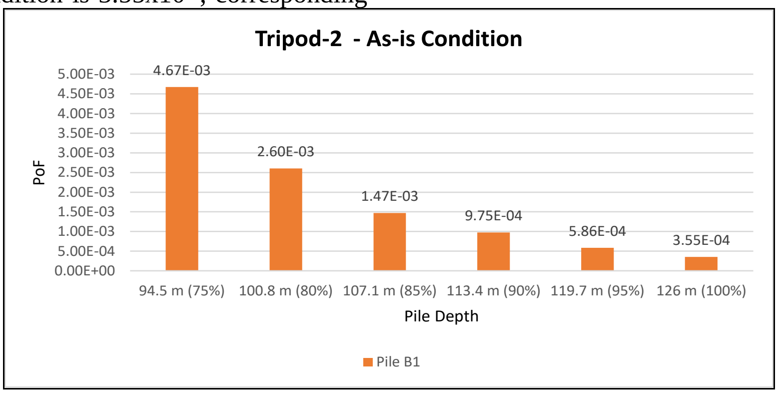

Figure. 24. Tripod-2 - PoF of As-is Condition

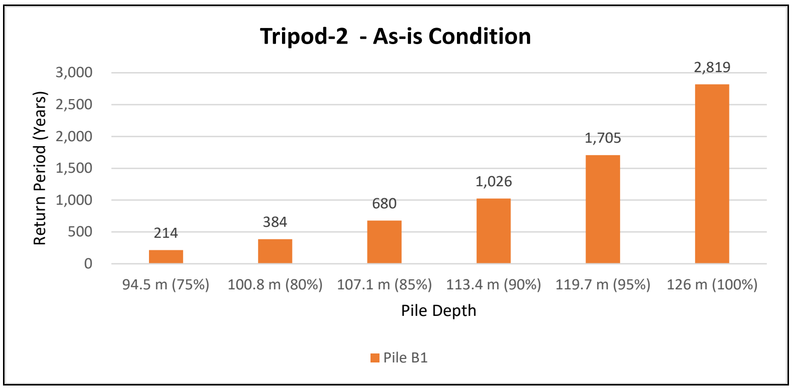

Figure. 25. Tripod-2 - Return Period of As-is Condition

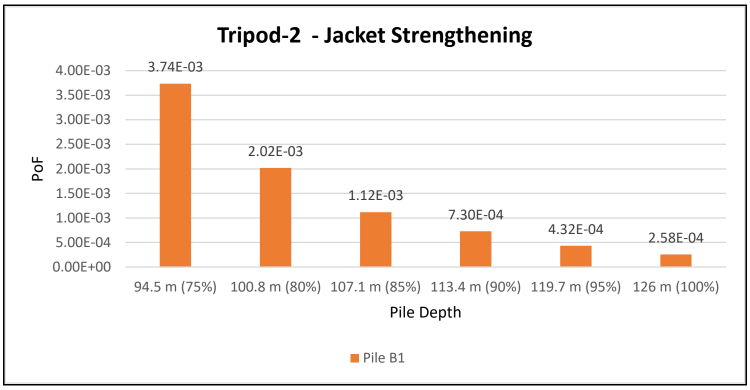

Figure. 26. Tripod-2 - PoF of Jacket Strengthening

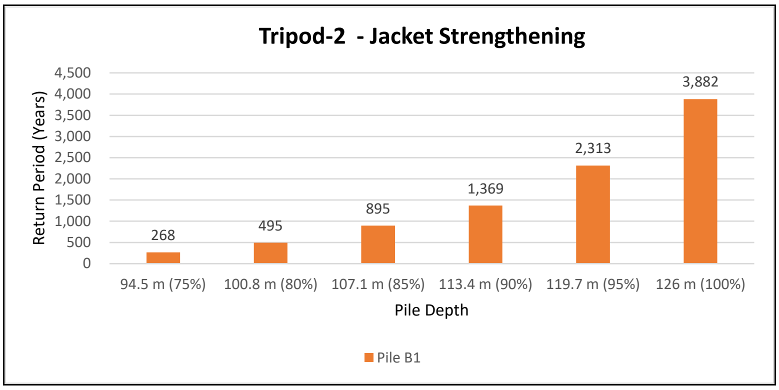

Figure. 27. Tripod-2 - Return Period of Jacket Strengthening

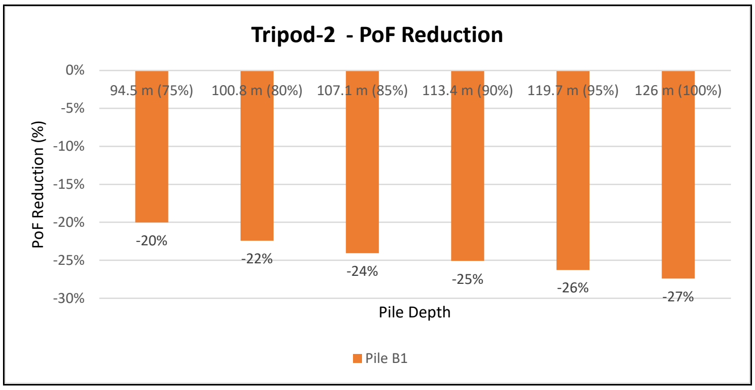

Figure. 28. Tripod-2 - PoF Reduction (%)

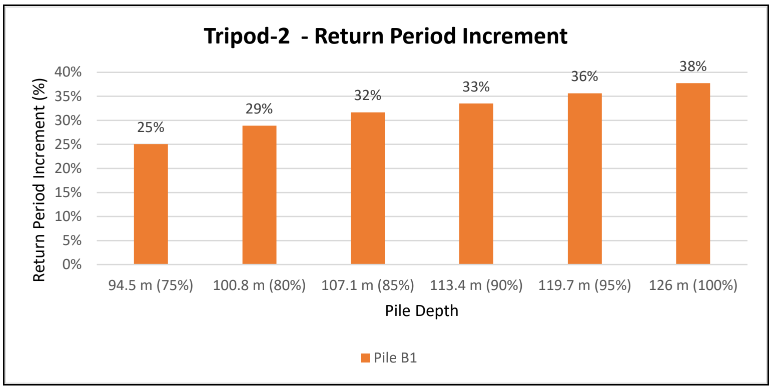

Figure. 29. Tripod-2 - Return Period Increment (%)

Hence, it is also confirmed that reinforcing or strengthening the jacket platform may reduce its probability of failure (PoF) [40].

V. Conclusion and Recommendation

Some conclusions and recommendations are taken as follows:

1. Both structural integrity and reliability assessments of ageing jacket platforms are essential to determine whether they are fit-for-purpose or risk-reduction measures that should be considered for continuing their operation, especially for offshore platforms that have approached or exceeded their design life. The observed platforms show that the scenarios of reducing topside loads, marine growth removal and jacket strengthening can be the options for risk reduction measures.
2. The pushover analysis results confirmed that the pile-soil interaction expressed by T-Z, Q-Z, and P-Y are the most critical parameters affecting the structural analysis of the platform's collapse. Those parameters are essential in determining the platform's ultimate capacity, represented by RSR and PoF values. The collapse behaviour of an ageing fixed offshore platform has shown that most events happened until the platform structures collapsed because the pile was nearing the punch trough.
3. Reducing the topside loads and removing the marine growth will slightly increase the collapse base shear from 0.22% to 7.33% and increase the RSR while reducing the PoF. Strengthening the jacket platform by installing soldier piles will significantly increase the collapsing base shear from 7.17% to 18.97%, as well as increase the RSR while reducing the PoF by about 27% and increasing the return periods by about 38% or from 2,819 years to 3,882 years, especially for a wave-dominated platform. Hence, jacket strengthening is the most suitable mitigation strategy for extending the platform's lifespan.

Based on the assessments conducted in this research, the recommendations may be considered in relation to platform life extension, as follows:

1. From the structural integrity and reliability perspective, the jacket strengthening scenario should be the most suitable mitigation strategy for extending the service life of an ageing offshore jacket platform.
2. A comprehensive cost feasibility analysis is advised to evaluate whether implementing the jacket strengthening scenario is feasible or if other risk reduction measures should be considered to continue operating the ageing offshore jacket platform.

Acknowledgements

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