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Structural Integrity of Fixed Offshore Platforms by Incorporating Wave-in-Deck

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Abstract: The structural integrity of offshore platforms is affected by degradation issues such as subsidence. Subsidence involves large settlement areas, and it is one of the phenomena that may be experienced by offshore platforms throughout their lives. Compaction of the reservoir is caused by pressure reduction, which results in vertical movement of soils from the reservoir to the mud line. The impact of subsidence on platforms will lead to a gradually reduced wave crest to deck air gap (insufficient air gap) and cause wave-in-deck. The wave-in-deck load can cause significant damage to deck structures, and it may cause the collapse of the entire platform. This study aims to investigate the impact of wave-in-deck load on structure response for fixed offshore structure. The conventional run of pushover analysis only considers the 100-year design crest height for the ultimate collapse. The wave height at collapse is calculated using a limit state equation for the probabilistic model that may give a different result. It is crucial to ensure that the reserve strength ratio (*RSR*) is not overly estimated, hence giving a false impression of the value. This study is performed to quantify the wave-in-deck load effects based on the revised *RSR*. As part of the analysis, the Ultimate Strength for Offshore Structures (USFOS) software and wave-in-deck calculation recommended by the International Organization for Standardization (ISO) as practised in the industry is adopted to complete the study. As expected, the new revised *RSR* with the inclusion of wave-in-deck load is lower and, hence, increases the probability of failure (*POF*) of the platform. The accuracy and effectiveness of this method will assist the industry, especially operators, for decision making and, more specifically, in outlining the action items as part of their business risk management.

Keywords: subsidence; wave-in-deck; probabilistic model; reserve strength ratio; probability of failure; structural reliability assessment

1. Introduction

Malaysia's oil and gas scene started in 1910 with the discovery of an onshore oil well in Miri, Sarawak [1,2]. With the right technology and knowledge, the exploration was extended to the offshore area in Peninsular Malaysia, with Tapis oil field being the first one discovered in 1969. Currently, there were more than 300 offshore platforms in Malaysia operated by PETRONAS Peninsular Malaysia Operation (PMO), Sarawak Operation (SKO) and Sabah Operation (SBO) [3,4]. According to Ayob et al. [5], in 2014, 65% of 191 offshore platforms have exceeded their design life, and the percentage will increase to 78% in 2019.

It is crucial to ensure that the structural integrity of offshore platforms in order to avoid structural failure [6–8]. Two factors may affect the structural integrity of an offshore platform. They are excessive load and insufficient strength [5]. The excessive loads come from the environmental loads, operational loads and accidental loads. Whereas, the insufficient strength may cause by an error in design, fabrication, installation and operation and degradation.

Various degradation issues faced by the ageing platform such as corrosion, weld crack, local denting, scour and subsidence [9–13]. These may affect the overall structural integrity of the platform as well as impede its operations as the redundancy and strength of the platform may decrease [14–16]. Besides, the lack of precise guidelines is also a significant issue while considering the life extension of aged facilities. The original design regulations and standards have often been used to document the safety of structures during the extended life. Most existing structures have a history of degradation, incidents and accidents, which are nowhere captured while assessment is carried out with original design guidelines [17,18]. There is a need to develop precise guidelines, which will take into account the historical condition of the structure in predicting its remaining life more accurately. This study considered as-is platform degradation, which is subsidence. The detailed methodology is presented in Section 3.

Subsidence may occur due to the depletion of the reservoir, hence causing it to be compacted [19,20]. The compaction will increase with the increase in the reservoir depletion. This eventually will lead to a surface subsidence over time. Figure 1 shows the offshore structures with subsidence (represent by Platform 1) and without subsidence (represent by Platform 2).

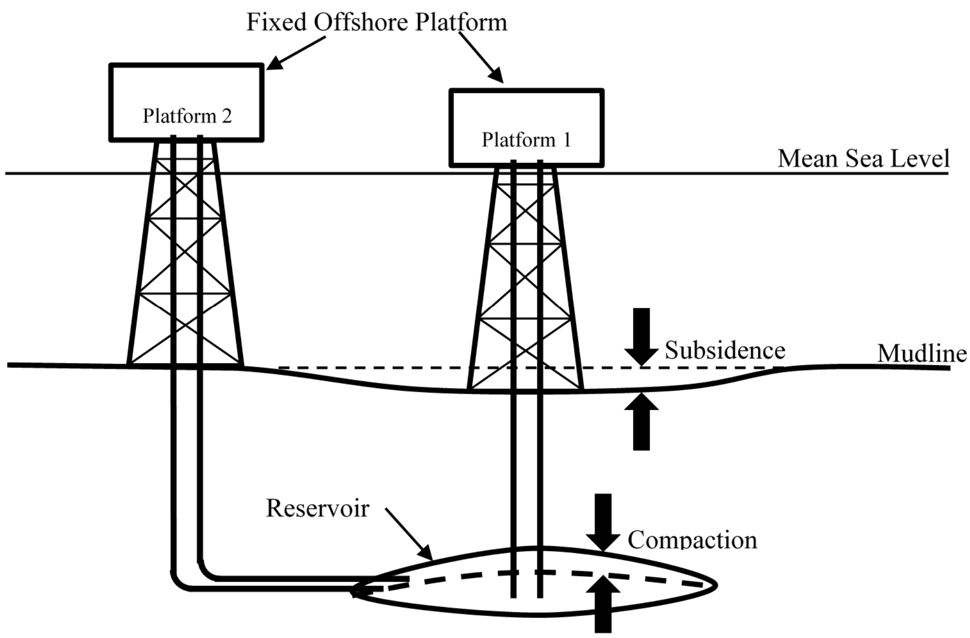
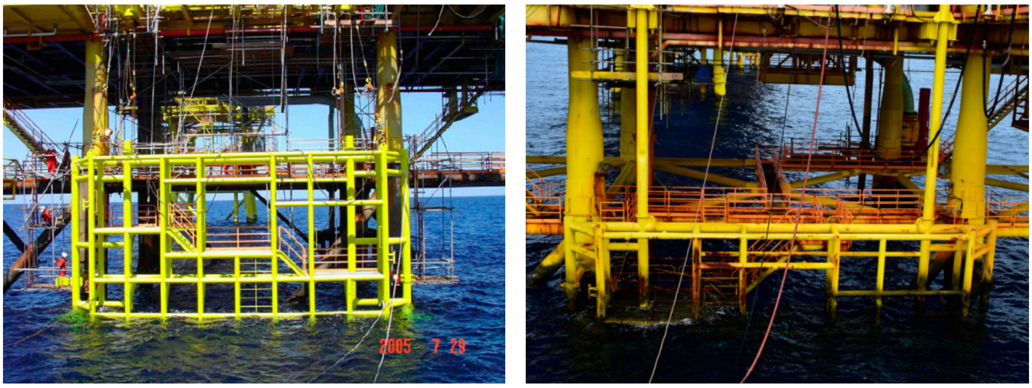


Figure 1. Offshore platform with and without subsidence.

The actual event of platform subsidence is illustrated in Figure 2a,b where the boat landing photo was taken in 2005 and 2016, respectively. It can be seen that the boat landing is no longer usable by 2016 as all the three stages of landing were submerged due to subsidence. Generally, it also means that the air gap of the platform is decreasing, and the condition may be worsened during the storm. Based on these figures, subsidence needs to be carefully assessed during the design stage of the platform in order to avoid catastrophic incident due to the condition where the water level becomes closer to the topside deck or, in other words, the loss of the air gap. Hence, there is the potential of a wave-in-deck scenario at the affected platform [21].



(**a**) (**b**)

Figure 2. Evidence of platform subsidence: (**a**) photo in 2005; (**b**) photo in 2016.

Structural reliability assessment using pushover analysis is widely used to calculate the reserve strength ratio and probability failure of offshore structures. Fairly recent studies had been performed by [5,22–26], which involved the structural reliability assessment. The pushover analysis had been performed by the authors to calculate the reserve strength ratio. The reserve strength ratio was then used to calculate the probability of failure. However, none of the authors included the wave-in-deck load during the determination of the reserve strength ratio.

Studies performed by [22,27–31] concluded that the wave-in-deck load needed to be considered in the pushover analysis. This was due to the fact that a huge wave hitting the offshore platform led to a high wave-in-deck load that could eventually resulted in significant platform damage and collapse.

The impact of waves on structures and the resulting impact load were significantly influenced by the water particle velocities, as proven from laboratory test conducted by Scharnke et al. [32]. The load estimates increased as the crest height and horizontal particle velocities increased. The loading from a wave impact event can be significant, and numerical prediction is quite challenging [33–35]. Wave impacts can be critical for local structural details as well as global structural integrity [32–37].

However, most of the collapse wave height was higher than the platform deck, and this led to the wave-in-deck scenario. Based on Golafshani et al. [38], the wave-in-deck load was ignored during the determination of RSR in the pushover analysis. In the conventional pushover analysis using USFOS software, the 100-year return period environmental load is incremented until the platform is collapsed and generate the ultimate resistance of the jacket. The forces are calculated up to the true sea surface based on 100-years return period; hence, the wave height is not scaled [21]. In the current study, wave-in-deck load will be considered if the wave crest height at collapse is higher than the bottom steel of the structure.

To avoid overestimation, studies performed by Mat Soom et al. [39] and Ayob et al. [40] carried out the RSR calculation up to the bottom of the steel structures by limiting the wave crest height. By doing so, they might overly underestimate the RSR in the event that the platform had the capacity to take a higher environmental load.

According to the literature review, there are three methods for calculating the wave-in-deck load, which are the silhouette method [41–44], component method [45,46] and computational fluid dynamics method, CFD [28,47–49]. Generally, the silhouette method is a simplified method based on projected area of the wave-in-deck; hence, no detailing is required. Unlike the silhouette method, the component method needs the details of the deck model in order to calculate the wave-in-deck load. The CFD method results compare well with the experiment results. The silhouette method as spelled in the International Organization for Standardization, 2007, [44] is adopted in this study as it is widely used for the design and assessment of the fixed offshore platform. The method is a simplified method to calculate the global wave-in-deck load acting on the topside structure. It is acknowledged that the wave-in-deck load is of dynamic in nature, for example, as per study performed by van Raaij [50], van Raaij and Gudmenstad [51] and Iwanowski et al. [52]. However, this study assumed that the wave-in-deck load acted together with the wave-on-jacket. Hence, it may give a conservative response, and this will in turn produce a conservative reserve strength ratio (*RSR*).

The conventional run of pushover analysis considers a 100-year crest height to calculate the reserve strength ratio. This study considers crest height at collapse by incorporating wave-in-deck load and investigates the impact on structure response for fixed offshore platform. It is expected that the reserve strength ratio of the platform will be reduced and, hence, increase the probability of failure of the platform when the wave-in-deck load is considered in the pushover analysis. This method will assist the industry for decision making for outlining of action items as part of their business risk management.

2. Wave-in-Deck Load and Reserve Strength Ratio

Wave-in-deck occurs when there is no deck clearance or air gap between the water level and the bottom steel of topside structure when it is hit by the waves [43,44]. To avoid wave-in-deck, all offshore platforms need to be adequately designed by providing an allowance for the air gap [31]. If the air gap is expected to reduce over time, the lowest deck may be designed at a higher elevation or the equipment seated on the deck need to be designed to cater for the wave-in-deck load.

Pushover analysis, also known as ultimate strength analysis, is commonly used in structural reliability assessment to determine the reserve strength ratio (*RSR*) of an offshore platform. The platform's ability to withstand a specific environmental load will be checked, especially for an ageing platform. The load as advised by the International Organization for Standardization, 2007, [44] is the 100-year environmental load.

The conventional run of pushover analysis only considers the 100-year design crest height for the ultimate collapse. However, when the *RSR* is higher, the wave height will also become higher [38]. However, no comprehensive study is carried out as to the effects of the wave-in-deck load, which are excluded during the *RSR* determination [38] as shown in Figure 3a. It is crucial to ensure that the *RSR* is not overly estimated, hence giving a false impression of the value. There is a possibility that the wave crest will reach the bottom steel of the deck structure, or even higher, and supposedly create a wave-in-deck scenario as shown in Figure 3b.

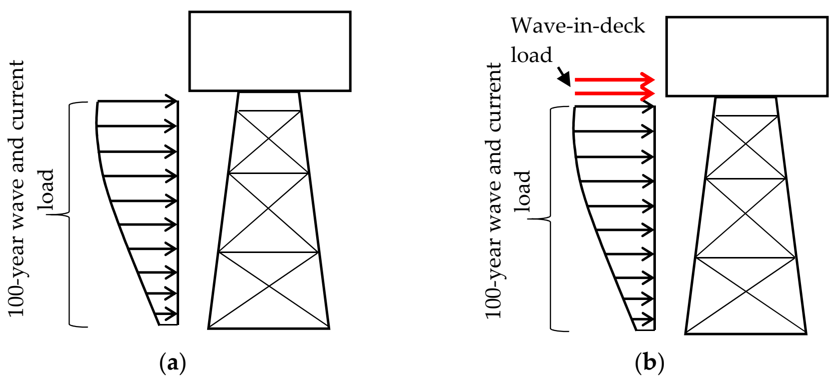


Figure 3. Comparison of the conventional method of limiting the 100-year wave and current load with the inclusion of wave-in-deck load: (**a**) conventional method; (**b**) with inclusion of wave-in-deck load.

The background of the problem for the existing platform may be summarized based on bow-tie diagram, which is widely used in risk analysis. It comprises fault trees and event trees, which are connected to the hazardous event [53,54]. The fault trees are the cause of the event. It is divided into two, which are the hazard and the threat, while the event trees are the consequences of the hazardous event. Figure 4 shows that the problem triggered by one of the hazard or degradation issue is the field subsidence. It will lead to insufficient air gap and possible wave-in-deck. The control barrier that needs to be undertaken is to determine the structure response strength and *RSR*. The hazardous event or the top event of the said hazard is the collapse of the platform.

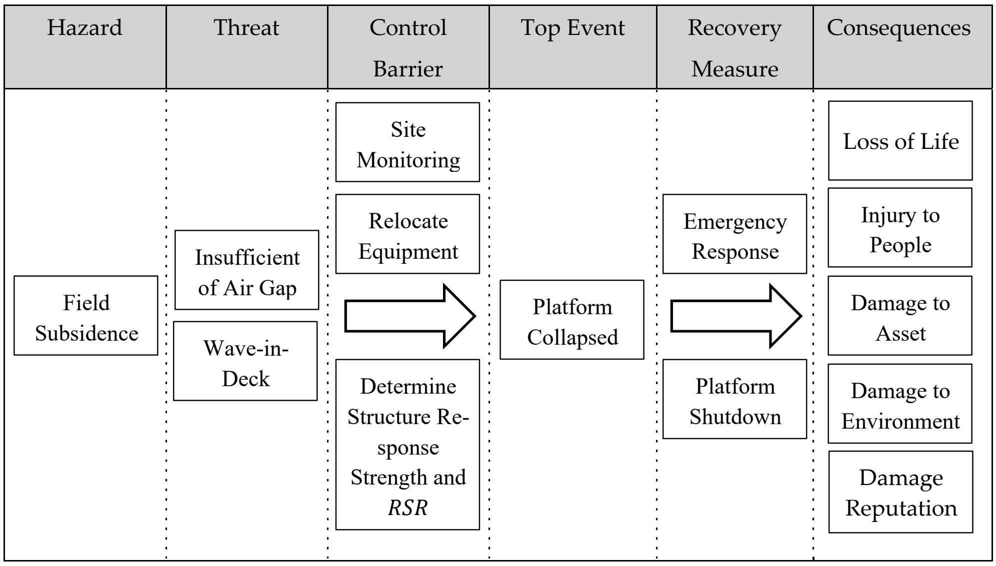


Figure 4. Bow-tie of existing jacket structure with field subsidence issue.

Other control barriers that may be considered are to perform site monitoring and to relocate the critical equipment in order to avoid the impact coming from the wave-in-deck. Right after the top event, the recovery measure needs to take place as soon as possible, such as emergency response and the platform shutdown. The consequences of the platform collapsing are loss of life, injury to people, damage to assets and environment and damage reputation. The bow-tie is one health, safety, security and environment (HSSE) tool support for as low as reasonably practicable (ALARP) [55].

3. Methodology

Reserve strength ratio (*RSR*) is defined as a ratio between a platform collapse load and a lateral environmental load, typically a 100-year return period (*RP*) load in terms of base shear or moment shear [5,25,56]. Therefore:

$$RSR = \frac{E_{collapse}}{E_{100}} \tag{1}$$

where RSR is the reserve strength ratio, $E_{collapse}$ is the base shear at the collapse of platform/ultimate capacity and E_{100} is the base shear of 100-year return period (*RP*).

The USFOS software that has been widely adopted for pushover analysis considers the wave forces up to the true sea surface. The wave load is scaled up proportionally but not the wave height [21]. In order to quantify the effects of wave-in-deck load, the maximum wave height at the collapse of the platform, h_{RSR} is determined by using the equation described by Ayob et al. [5] below:

$$h_{RSR} = RSR^{1/\alpha} \times h_{100} \tag{2}$$

where h_{RSR} is the maximum wave height at the collapse of the platform, h_{100} is the 100-year wave height, and α is the metocean constant, typically 1.7 to 2.0. In the absence of a more exact value, α can be taken as 1.7. The h_{RSR} is conservatively used to calculate the projected area of the wave-in-deck as explained in Equation (3).

For this research, wave-in-deck load is determined by using the equation introduced by the International Organization for Standardization, 2007 [44]. This equation falls under the silhouette approach. The wave-in-deck load, $E_{topsides}$:

$$E_{topsides} = \frac{1}{2}\rho_w C_d (\alpha_{wk} U_w + \alpha_{cb} U_c)^2 \times A_w \quad (3)$$

where $E_{topsides}$ is the wave-in-deck load, ρ_w is density of seawater, 1.025 MT/m^3, C_d is the empirical drag coefficient, α_{wk} is wave kinematic factor from met ocean data, U_w is the fluid velocity corresponding to crest height, α_{cb} is the current blockage factors, U_c is the current speed in line with the wave from met ocean data, and A_w is the projected area of the wave-in-deck. To avoid the overestimation of the *RSR*, the A_w is calculated from the bottom steel of cellar deck elevation up to the h_{RSR}. Even though the wave-in-deck load is taken at the collapse of the platform, it may give a good indication of the *RSR* rather than the case without considering the wave-in-deck load.

Reliability-based design and assessment (RBDA) introduced by Shell has been selected to calculate the probability of failure (*POF*) of the platform. The selected method adopts the convolution method, which consists the long-term load distribution, typically the 100-year, 1000-year and 10,000-year environmental loads. The equation presented in this section has been extracted from a study performed by Efthymiou and van de Graff [57] and Mat Soom et al. [26]. The probability of failure can be written as follows:

$$POF = \int_0^\infty (1 - P_E(x)).P_R(x)dx \quad (4)$$

where *POF* is the probability of failure, P_E is the probability density function, and P_R is the resistance probability density, Equation (4) can be rewritten as:

$$POF = A\ exp[-R_{mean}/E_0]exp\left(\sigma_R{}^2/2E_0{}^2\right) \quad (5)$$

where A and E_0 is the environment constant of $0.01 \times exp^{[2.3026/\alpha_L]}$ and $\alpha_L/2.3026$, respectively, with α_L being the constant of linearity (i.e., 0.31–0.37), R_{mean} is the mean of distribution of structural strength, and σ_R is the standard deviation of the distribution of structural strength.

Figure 5 illustrates the overall analysis procedure (accounting for the effect of wave-in-deck load in the pushover analysis) for this study. The steps for calculating the reserve strength ratio (*RSR*) and probability of failure (*POF*) with the inclusion of wave-in-deck are as follows:

(*A*) *Platform Identification and Modelling*. Selected platform was verified against latest as-built drawings, weight control report and inspection report to ensure that the analysis will represent the actual condition at site. Latest metocean data for 100-year return period were utilized consisting of maximum wave height, h_{100}, and associated period, t_{ass}, and performed long-term distribution. Dynamic analysis was carried out to generate inertia loads. In this step, SACS software was used.

(*B*) *USFOS Model Preparation* The analysis model from step (A) was then converted to a suitable format, in this case, user-friendly (UFO) format for the subsequence pushover analysis [58]. The converted model, known as the "model.fem" file, was verified to ensure that all items such as geometries, section properties and loading were properly converted. In this study, Struman software was used for the conversion. After that, the header file was prepared. The header, known as the "header.fem" file, consists of sets of commands for the software to execute pushover analysis.

(*C*) *Non-Linear Pushover Analysis and RSR Determination* Two input files are required to perform the pushover analysis, which are model file and header file. The pushover analysis was performed by incrementing the 100-year environmental loads until the platform collapses. The *RSR* was determined based on the base shear at collapse load divided by the base shear of 100-year environmental loads, as per Equation (1). The failure mode of the platform was also determined to identify the governing failure.

(*D*) *Air Gap Analysis* From the *RSR* produced in Step 3, the wave height at collapse was calculated using the limit state equation for probabilistic model equation introduced by Ayob et al. [5] as per Equation (2). Next, the wave height at collapse, h_{RSR}, was compared against the bottom steel of cellar deck, CD_{EL}.

(*E*) *Non-Linear Pushover Analysis and RSR Determination (with Inclusion of Wave-in-Deck)* Wave-in-deck load was calculated for platform, which has the wave hitting the deck, as outlined by the ISO [44]. Dynamic analysis, considering wave-in-deck load, was also performed to generate new inertia loads. The wave-in-deck and inertia loads were then included in the pushover analysis and the new *RSR* was defined.

(*F*) *Probability of Failure (POF) Calculation* From the *RSR* results, the probability of failure of the platform was calculated using the reliability-based design and assessment (RBDA) methods [26,57]. The method adopts the convolution method and considers the long-term load distribution.

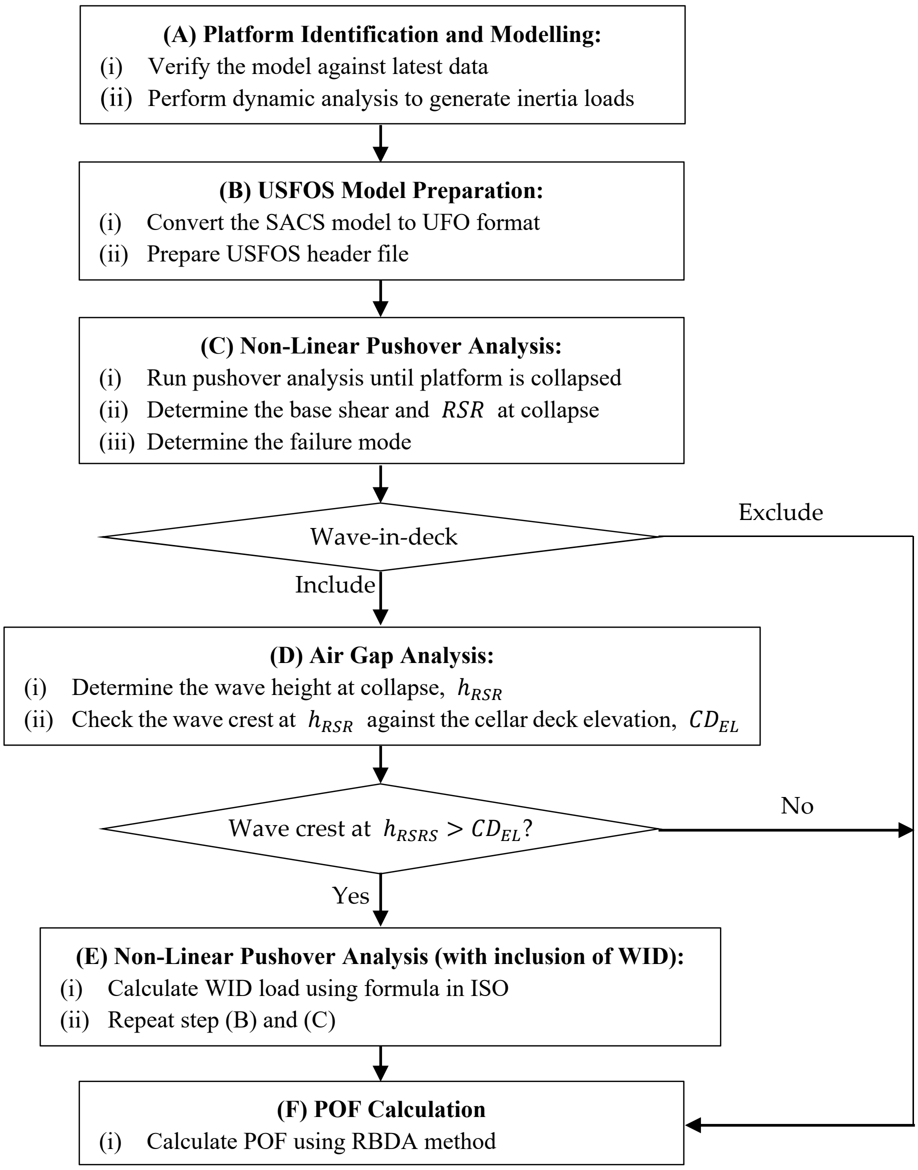


Figure 5. Flowchart of analysis procedure.

4. Test Structure Specification

Five platforms have been selected for this study, namely, PD4-40, PV3-88, PK4-88, PP8-88 and PD4-130. The platforms are located at either Sarawak Operation (SKO) or Sabah Operation (SBO) as shown in Table 1. The types of platforms are two drilling, a vent, a compression and a production platform.

Table 1. Test structures specification.

No.	Platform Name	Field	Operation	No. of Leg	Installation Year	Water Depth As-Installed (m)	Water Depth in 2015 (m)
1	PD4-40	Sabah Operation (SKO)	Drilling	4	1980	40.3	40.4
2	PV3-88 PK4-88 PP8-88	Sarawak Operation (SKO)	Vent Compression Production	3 4 8	1982 1999 1982	88.4	93.7 93.8 93.7
3	PD4-130	Sarawak Operation (SKO)	Drilling	4	2002	129.9	132.8

The platforms have been selected based on three criteria. The first criterion is the similarity in terms of water depth during installation. The selected platforms are 3-legged, 4-legged and 8-legged. The second criterion is the difference in terms of water depth, which are from 40.3 m to 129.9 m with respect to mean sea level (MSL) at the time of installation. Those platforms that have the same number of legs are 4-legged platforms. Platform PD4-130 is located at the deepest water depth of 129.9 m, whereas platform PD4-40 is located at the shallowest water depth of 40.3 m at the time of installation. The remaining three platforms are located at the water depth of 88.4 m at the time of installation.

The third criterion is based on the subsidence. Three out of five of the selected platforms are subsiding more than 5 m over time. The platforms are PV3-88, PK4-88 and PP8-88, which are located at the same field. The other two (2) platforms are PD4-130 subsided by 2.864 m and PD4-40 subsided by 0.103 m in 2015. Hence, the air gap of the platforms is also decreasing. These criteria are selected in order to see the impact of the pushover analysis with the inclusion of the wave-in-deck load. It is crucial to include the subsidence in the analysis as the wave-in-deck is highly likely to occur at the platform. Detailed specification of the platforms is shown in Table 2.

Table 2. Detailed specifications of the platforms.

Platform / Features	Description				
	PD4-40	PV3-88	PK4-88	PP8-88	PD4-130
Design Safety Category	Unmanned	Unmanned	Manned	Manned	Unmanned
Brace Type	K-brace	K-brace	Combination of X-brace and K-brace	K-brace	X-brace
Number of Legs	4 (46.5″Ø)	3 (46.5″Ø)	4 (60″Ø)	8 (60″Ø)	4 (80″Ø)
Number of Pile	4 (42″Ø)—Through Leg	3 (42″Ø)—Through Leg	4 (54″Ø)—Through Leg	8 (54″Ø)—Through Leg	8 (84″Ø)—Skirt Pile
Number of Risers	4 (1 × 8″Ø and 3 × 6″Ø)	2 (18″Ø)	None	2 (1 × 30″Ø and 1 × 18″Ø)	3 (2 × 24″Ø and 1 × 20″Ø)
Number of Caisson	1 (24″Ø)	None	2 (30″Ø)	1 (24″Ø)	1 (30″Ø)
Boat Landing	1	1	None	2	2
Conductor	6 (2 × 36″Ø and 4 × 26″Ø)	None	None	None	12 (26″Ø)
Bridge Link	None	None	2	3	None

Table 2. *Cont.*

Features \ Platform	Description				
	PD4-40	**PV3-88**	**PK4-88**	**PP8-88**	**PD4-130**
Deck Configuration	2-Level Deck: Wireline Deck and Cellar Deck	1-Level Deck: Cellar Deck	2-Level Deck with 2-Modules: Module Support Frame Deck and Cellar Deck	2-Level Deck: Upper Deck and Cellar Deck	3-Level Deck: Helideck, Main Deck and Cellar Deck
Material	Carbon Steel—Mild Strength (248 MPa)	Carbon Steel—Mild Strength (248 MPa)	Carbon Steel—High Strength (345–355 MPa)	Carbon Steel—Mild Strength (248 MPa)	Carbon Steel—High Strength (340–355 MPa)

5. Results

5.1. Platform Subsidence

Latest platform subsidence was calculated based on an air gap survey conducted in 2015 (courtesy of Shell). The global positioning system (GPS) survey was used to obtain the latest elevation of the platform, with respect to the mean sea level (MSL). The survey campaign was split into two parts, which were platform levelling and platform lateral movement. Two (2) sets of global navigation satellite system (GNSS) receivers and levelling equipment were used, and the survey data received in the field were processed using the Trimble Business Centre (TBC) software. The latest elevation acquired on site was compared to as-built level during platform installation. Figure 6 shows the comparison of cellar deck level in 2015, the as-built level at the time of installation and the total subsidence of each platform.

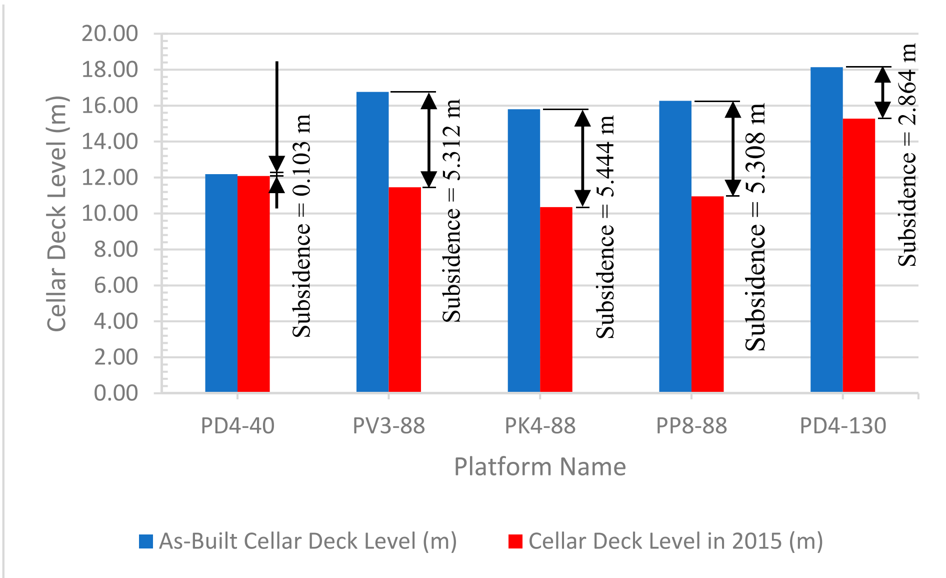


Figure 6. Platform subsidence.

From Figure 6, platform PD4-40 had the least subsidence, which may be due to minimum reservoir depletion with the total subsidence of 0.103 m followed by platform PD4-130, which has a total subsidence of 2.864 m. It was also observed that platforms PV3-88, PK4-88 and PP8-88 have subsidence close to one another, with the maximum subsidence of 5.444 m. This was due to the location of the platforms, which were in the same field; hence, there was a possibility that the platforms were sharing the same reservoir. When the reservoir was depleted over time, it would have caused compaction and led to subsidence of the area.

5.2. Wave Height

Air gap analysis was performed to determine whether there was a wave-in-deck issue on the platforms based on the RSR generated from pushover analysis. The limit state equation for a probabilistic model was used to calculate the maximum wave height at collapse, h_{RSR}. From the h_{RSR}, the wave crest height at collapse was calculated and compared with the bottom steel of the structure. Figure 7 shows the wave crest height at collapse and the bottom steel height of the platform.

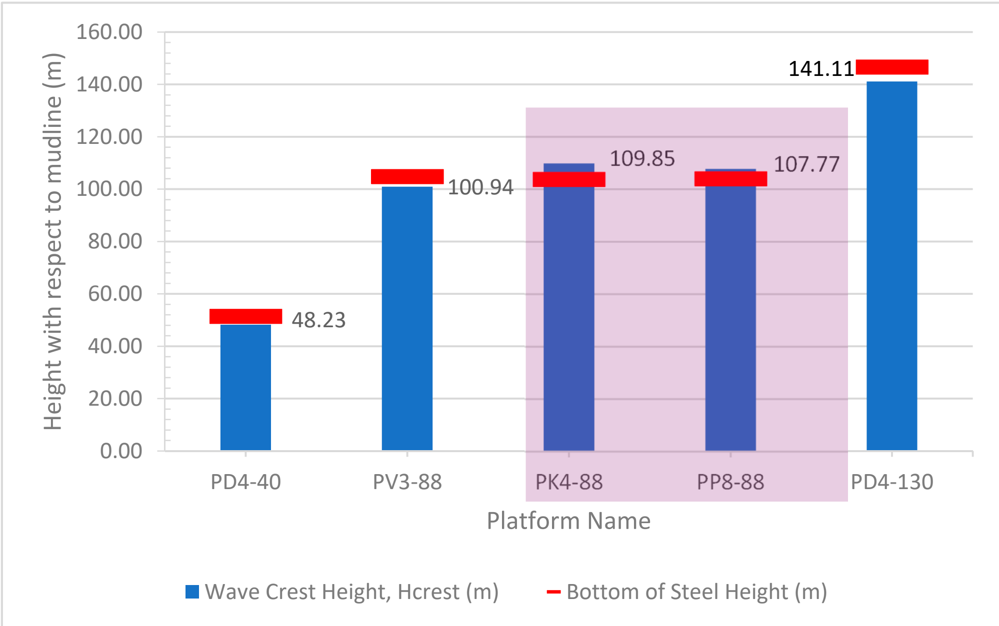


Figure 7. Comparison of bottom steel height and wave crest height at collapse.

Based on Figure 7, two of the platforms, namely, PK4-88 and PP8-88, as shaded, had wave-in-deck issues due to the wave crest heights at collapse being higher than the bottom steel of the decks. The wave crest heights at platforms PK4-88 and PP8-88 were found to be higher by 6.064 m and 3.720 m, respectively, from the bottom steel of the structures. So, the wave-in-deck loads should be included in the pushover analysis for these two platforms.

The other three platforms, namely, PD4-40, PV3-88 and PD4-130, which had wave crest heights lower than the bottom of steel and no wave-in-deck load, were included in the pushover analysis. The wave crest heights for platforms PD4-40, PV3-88 and PD4-130 were found to be lower by 3.211 m, 3.910 m and 5.530 m, respectively, from the bottom steel of the structures.

Sections 5.3–5.5 focuses on the platforms that had wave-in-deck issues; hence, only the results of platforms PK4-88 and PP8-88 as shaded in Figure 7 were compared. The results were inclusive of reserve strength ratio and probability of failure of the platform.

5.3. Wave-in-Deck Load

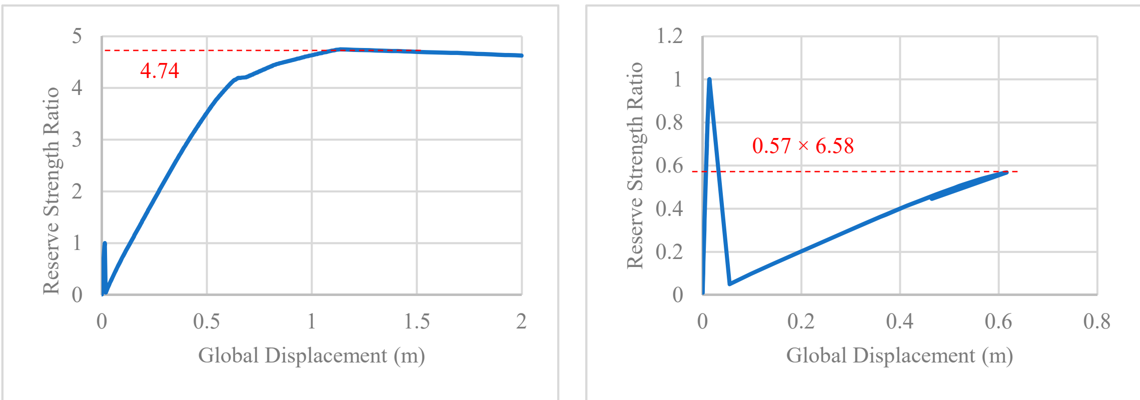
Wave-in-deck load for platforms PK4-88 and PP8-88 were calculated using Equation (3). As explained in Section 5.2, the wave crest height at collapse, h_{RSR}, of both platforms was higher than the bottom of steel (BOS) of the structures. Hence, the wave-in-deck load was added in the subsequent pushover analysis. It assumed that the platforms were moderately equipped. The waves had been calculated using Stoke's 5th wave theory. The current speeds were extracted from the metocean data. The detailed calculations are presented in Table 3.

Table 3. Wave-in-deck load calculation.

Item	PK4-88	PP8-88
Deck width perpendicular to the wave (m)	32.000	55.860
Distance between BOS and h_{RSR} (m)	6.064	3.720
Density of seawater, ρ_w (MT/m^3)	1.025	1.025
Drag coefficient, C_d	2.000	2.000
Wave kinematic factor, α_{wk}	1.000	1.000
Fluid velocity corresponding to crest height, U_w (m/s)	10.225	9.238
Current blockage factors, α_{cb}	1.000	1.000
Current speed in line with the wave, U_c (m/s)	0.900	0.900
Projected area of the wave-in-deck, A_w (m^2)	194.048	207.799
Wave-in-deck load (MT)	2522.918	2231.533

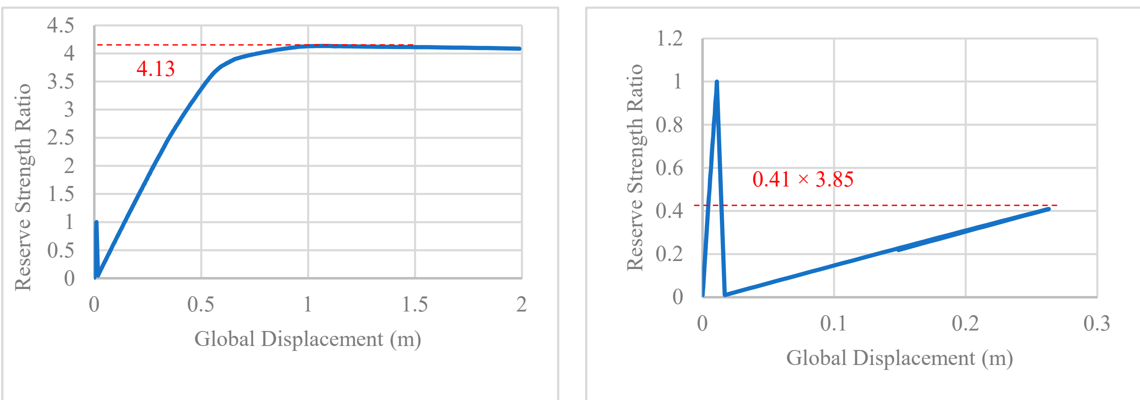
5.4. Reserve Strength Ratio

Based on air gap analysis results, which were tabulated in the previous section, the pushover analysis for the platforms, which had the wave crest height higher than the bottom of steel, was rerun to include the wave-in-deck load. They were platforms PK4-88 and PP8-88. The reserve strength ratio versus global displacement are presented in Figures 8 and 9 for PK4-88 and PP8-88, respectively.



(**a**) (**b**)

Figure 8. Global load versus global displacement for PK4-88: (**a**) PK4-88: without wave-in-deck load; (**b**) PK4-88: with wave-in-deck load.



(**a**) (**b**)

Figure 9. Global load versus global displacement for PP8-88: (**a**) PP8-88: without wave-in-deck load; (**b**) PP8-88: with wave-in-deck load.

For platform PK4-88, as shown in Figure 8a, the *RSR* without considering wave-in-deck load is 4.74. In order to calculate the *RSR* for the case with the inclusion of wave-in-deck load, the *RSR* from Figure 8b needs to be multiplied by the ratio of the base shear at collapse of the platform (e.g., which includes the wave-in-deck load) and base shear of the 100-year return period. In this case, the ratio of the said base shear is 6.58, and this will bring the *RSR* with the inclusion of wave-in-deck load as 3.75. For platform PP8-88, as shown in Figure 9a, the *RSR* without considering wave-in-deck load is 4.13. The ratio of the base shear at collapse of the platform (e.g., which includes the wave-in-deck load) and base shear of the 100-year return period is 3.85. This will bring the *RSR* with the inclusion of wave-in-deck load to 1.58 after multiplying the ratio of the base shear with 0.41 as shown in Figure 9b. The reserve strength ratio with and without the wave-in-deck loads are tabulated in Figure 10.

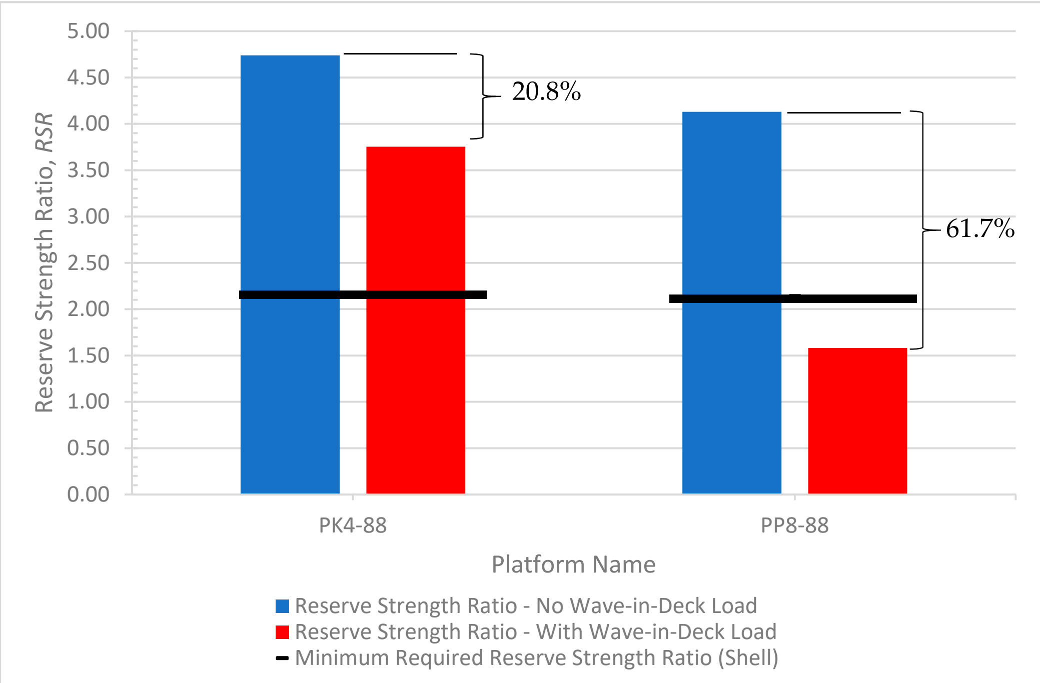


Figure 10. Comparison of reserve strength ratio with and without inclusion of wave-in-deck load.

From Figure 10, it is observed that the reserve strength ratio with the inclusion of wave-in-deck load is lower if compared to the one without wave-in-deck load. For platform PK4-88, the *RSR* with and without wave-in-deck load are 3.75 and 4.74, respectively; whereas, for platform PP8-88, the *RSR* with and without wave-in-deck load are 1.58 and 4.13, respectively. In terms of percentage, the differences of the reserve strength ratio with and without wave-in-deck loads are 20.8% and 61.7% for platforms PK4-88 and PP8-88, respectively. It also found that the *RSR* with the inclusion of wave-in-deck load for platform PP8-88 did not meet the minimum required *RSR* of 2.14 as per the calculation adopted by Shell.

It was also found that platform PK4-88 had a higher *RSR* if compared to platform PP8-88. That is why the wave crest height at collapse for platform PK4-88 is higher than platform PP8-88. It is also observed that the wave-in-deck load significantly reduced the *RSR* of platform PP8-88. Even though the wave crest height at collapse for platform PP8-88 was lower if compared to platform PK4-88, the wave-in-deck load for platform PP8-88 was found to be higher. This was because platform PP8-88 had a higher projected wave-in-deck area as the platform was more prominent than platform PK4-88. Based on the bow-tie presented in Section 1, platform PP8-88 may collapse in the event of wave-in-deck and, hence, may lead to loss of life, injury to people, damage to asset and environment and damage reputation.

5.5. Probability of Failure

The probability of failures is calculated using the RBDA method from the *RSR* and base shear of the 100-year and 1000-year period. Both probability of failures with and without wave-in-deck load of platforms PK4-88 and PP8-88 were considered. The calculation of the probability of failure (*POF*) is presented in Table 4 below while Figure 11 shows the comparison of probability of failure with and without wave-in-deck load:

Table 4. Calculation of probability of failure.

Platform	Wave-in-Deck	Return Period	*RSR*	Base Shear (MN)	α	*A*	E_0	*POF*
PK4-88	Without	100 1000	4.74 3.86	4.46 5.86	0.31	15.34	0.14	2.64×10^{-12}
	With	100 1000	3.75 3.86	4.46 5.86				5.03×10^{-10}
PP8-88	Without	100 1000	4.13 2.90	7.69 10.50	0.36	5.52	0.16	5.39×10^{-10}
	With	100 1000	1.58 2.9	7.69 10.5				3.97×10^{-4}

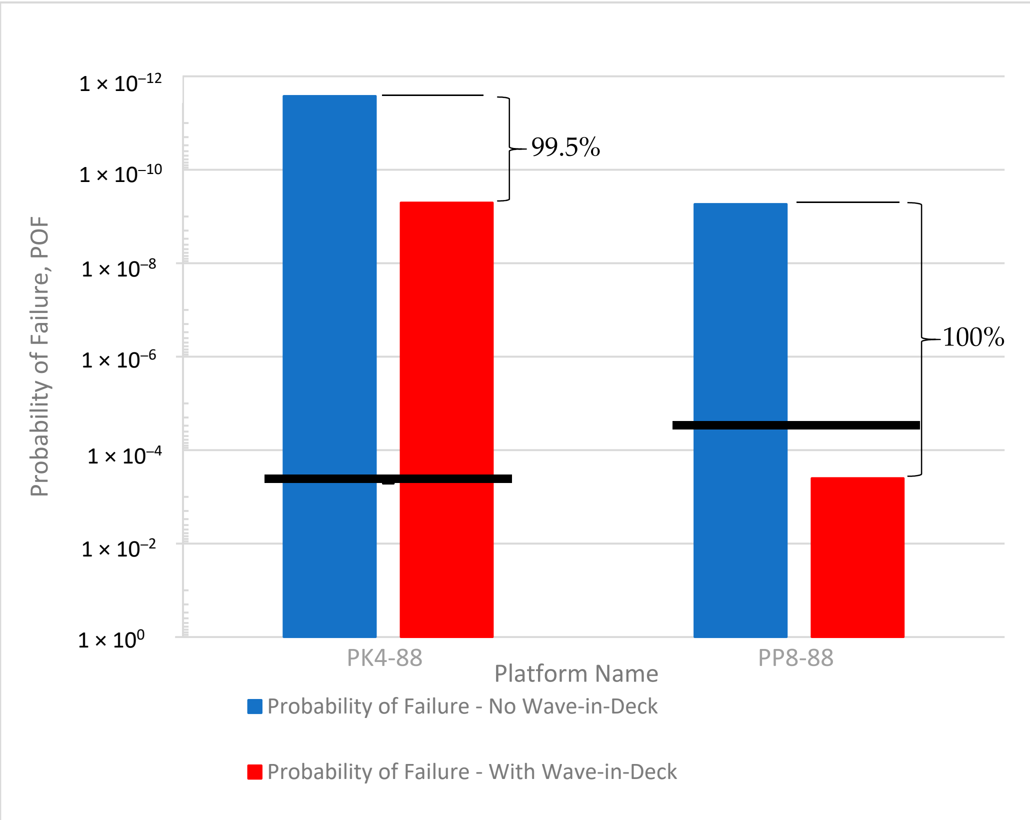


Figure 11. Comparison of probability of failure with and without inclusion of wave-in-deck load.

From Figure 11, it is observed that the probability of failure with the inclusion of wave-in-deck load was found to be higher than the exclusion of wave-in-deck load. For platform PK4-88, the probabilities of failure with and without wave-in-deck load were 5.03×10^{-10}

and 2.64×10^{-12}, respectively. For platform PP8-88, the probabilities of failure with and without wave-in-deck load were 3.97×10^{-4} and 5.39×10^{-10}, respectively. In terms of percentage, the differences in the probabilities of failure with and without wave-in-deck load are 99.5% and 100.0% for platforms PK4-88 and PP8-88, respectively. It means that the chances of the platform failing or collapsing were bigger when the wave-in-deck load was considered in the pushover analysis.

If compared to the requirement of the International Organization for Standardization, 2007 [44], the probability of failure with and without wave-in-deck load for platform PK4-88 was higher than the minimum required probability of failure of 5.00×10^{-4}, which was set by the L2 installation requirement. For platform PP8-88, the probability of failure without wave-in-deck load was higher than the minimum required probability of failure of 3.00×10^{-5}, which was set by the L1 installation requirement. However, with the inclusion of wave-in-deck, the platform PP8-88 did not meet the minimum requirement.

It was also observed that the probability of failure with the inclusion of wave-in-deck load for platform PP8-88 was significantly reduced if compared to platform PK4-88. This was because the wave-in-deck load for platform PP8-88 was higher if compared to platform PK4-88. It also means that the chances of the platform failing were higher if the platform is hit by the wave-in-deck load.

For platform PP8-88, it is recommended that further assessment is to be made to minimize the impact of wave-in-deck. The current study adopted the wave-in-deck load calculation based on the silhouette method as recommended by the ISO [44]. It is suggested that the component method [45,46] or computational fluid dynamic method [28,47–49] are to be utilized to calculate the wave-in-deck load in order to reduce the conservatism of the silhouette method. The risk of platform damage may be reduced via modification such as localized protection or relocation of exposed sensitive equipment or via procedures such as production shut-in and temporary evacuation during storm season [59,60]. Another method that was successfully performed was by jacking-up the platform in order to increase the air gap of platform [61]. This is crucial so that the effect of wave-in-deck can be minimized to avoid major damage or failure to offshore platforms as reported by Botelho et al. [62], Puskar et al. [63] and Forristall [64].

6. Conclusions

Pushover analysis has been widely used to calculate the *RSR* of the platforms. It has also been used in structural reliability assessment to calculate the probability of failure of offshore platforms. Even though the wave-in-deck load may lead to a disastrous effect, it has been excluded in the *RSR* determination. Some studies had limited the *RSR* up to the bottom steel of platforms only [38,39]. Generally, there are two methods of calculating the wave-in-deck load, which are the silhouette method and component method. In this study, the silhouette method introduced by the International Organization for Standardization, 2007, [44] was adopted to calculate the wave-in-deck load.

No comprehensive study has been made on the effects of the wave-in-deck load as the conventional pushover analysis does not take the wave-in-deck load into account when calculating the *RSR*. This is because the forces are calculated up to the wave crest height of the 100-year environmental load only. It is noted that high *RSR* has a high wave crest at collapse; hence, the wave crest may be higher than the wave crest height of the 100-year environmental load and bottom steel of structures that supposedly create a wave-in-deck. Thus, the wave-in-deck load cannot be ignored because it will lead to an overestimation of the *RSR* value.

Five fixed offshore platforms with water depth ranging from 40.3 m to 129.9 m have been selected for this study. The platforms are platforms PD4-40, PV3-88, PK4-88, PP8-88 and PD4-130. Three of the platforms, which are platforms PV3-88, PK4-88 and PP8-88, are subsiding more than 5 m. Three out of five platforms have past their initial design life of 30 years, which are platforms PP8-88, PV3-88 and PD4-40.

A procedure to consider the wave-in-deck load in determining the *RSR* from the pushover analysis has been introduced in this research. Air gap analysis is performed based on the *RSR* value from the conventional pushover analysis. It is crucial to determine whether the wave crest at collapse of the platform is higher than the bottom steel of the structure or lower than that. If the wave crest at collapse of the platform is higher than the bottom steel of the structure, another run of pushover analysis needs to be performed. Based on the results, two of the platforms, which are platforms PK4-88 and PP8-88, have wave crest heights higher than the bottom steel of the structures. This time, the wave-in-deck loads need to be considered in the analysis of those two platforms. The silhouette method, as explained by the International Organization for Standardization, 2007, [44] is adopted in this research to calculate the wave-in-deck load.

From this research, it was found that the *RSR* with the inclusion of the wave-in-deck load is lower than without the wave-in-deck load with a maximum difference of 61.7%. Based on the reliability-based design and assessment (RBDA), the probability of failure (*POF*) is higher with the inclusion of the wave-in-deck load with a maximum difference of 100.0%. Higher *POF* means that the chances of the platform to fail or collapse are more significant with the addition of the wave-in-deck load. It can be concluded that it is crucial to include the wave-in-deck load in the *RSR* determination in order to avoid the overestimation of the value.

The results given in this research may be further investigated based on the following recommendations:

1. The metocean constant, α used in this research may be further studied depending on the location of the offshore platform. Currently, α is conservatively taken as equal to 1.7, as suggested by Ayob et al. [5], for Malaysia waters of fixed offshore platform.
2. Current study focuses on horizontal wave-in-deck only. It is recommended that investigation on the impact of vertical wave-in-deck load should also be carried out.
3. Current study does not consider the dynamic effect of the structure when exposed to the wave-in-deck load as the analysis is performed under static non-linear pushover.

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