Determination of LRFD environmental load factor in the java sea for unbraced monopod structures using reliability analysis

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Abstract. Case studies were carried out for unbraced monopod structures located in the Java Sea by reviewing the value of the reliability index. This sea has a depth of 33 meters and a 100-yr wave height of 6.29 meters. The structures are designed optimally with a unity check ratio of 1.00 using API RP 2A WSD and API RP 2A LRFD standards under storm condition, assumed to be unmanned. For LRFD criteria, the value of the environmental load factor is varied, namely 1.35, 1.20, 1.10, and 1.00. Collapse analysis using static pushover analysis of the model produces a minimum RSR value meeting the minimum criteria for unmanned structures of 1.6. The reliability analysis process uses the First Order Reliability Method II (FORM II). The structural reliability index value has met the minimum criteria issued by DNV of 3.71 for the structure that is designed using the environmental load factor of 1.35. It can be concluded for unbraced monopod structures designed in the Java Sea, the environmental load factor value of 1.35 is following the minimum reliability index value proposed by DNV.

1. Introduction

The process of oil and gas field exploration requires an infrastructure that designed optimally using applicable design standards to be able to withstand the equipment and environmental loads later. Structures designed using Load Resistance Factor Design (LRFD) standards are much more reliable compared using Working Stress Design (WSD) criteria due to the use of load factor, based on the geographic location [1]. WSD criteria consider the uncertainty components related to resistance and load variable using the safety factor by reducing the allowable strength. While LRFD criteria include the uncertainty of resistance and load variable using the statistics of the resistance and load variable. The WSD criteria became uneconomical because it doesn't consider the uncertainty factor [8].

Commonly used standards are published by the American Petroleum Institute (API) which adapts water conditions in the Gulf of Mexico. The environment conditions in the Gulf of Mexico are relatively more extreme than in Indonesia, so the application of environmental load factors in Indonesia needs to be reviewed. Indonesia has relatively calmer environmental conditions and more shallow water fields. The use of monopod structures will be more economical for exploiting these conditions. The partial load factors in API were also calibrated for jacket platforms [10]. The load transfer between jacket platform and monopod platforms are also different, where monopod platforms distribute both vertical and horizontal loads to one member. It is necessary to study the use of monopod structures.

The calculation of offshore structure analysis is the subject of uncertainty. Uncertainty can be faced by considering the uncertainty parameters of the resistivity and load variables. The values are analyzed based on the amount of information about the statistical parameters that contain uncertainty [1].
Accordingly, uncertainty modeling is essential to perform reliability analysis towards offshore structures. Studies about the determination of environmental load factors in nearby regions produce different values. These values significantly depend on the geographic conditions of each region [9]. Kurian et. al. (2014) proposed the environmental load factor of 1.25 for jacket platform structure in Malaysian waters. This value is obtained from the system reliability of all jacket components. Meanwhile, Ronalds (2007) suggested the environmental load factor of 3.00 for concrete monopod structures in the Australia North West Sea. Reliability analysis was conducted by reviewing the Reserve Strength Ratio (RSR) of the structure's main leg with a reliability index target of 3.3. This target is converted from the probability of failure ($PoF$) of $5 \times 10^{-4}$ for unmanned structures (ISO 19902).

2. Methodology

2.1 Structure Modelling

This study took place in the Java Sea (Figure 1) with a depth of 33 m. With the same environmental and topside load conditions, the optimum design (UC = 1.00) is carried out by using API RP 2A WSD and API RP 2A LRFD, which may produce more than one different structure. For LRFD criteria, variations are applied for the environmental load factor values ($\gamma_E$), namely 1.35, 1.20, 1.10, and 1.00. A total of five structures are managed by changing the thickness of the leg, while keeping the diameter for all structures at the same value, using SACS.

The deck is located 12 m above Stillwater and modeled as a simple beam with a weight of 1000 kN and length of 10 m. The structure is assumed to be unmanned with zero marine growth. The whole structure is made of steel, assumed to have a yield stress of 248 MPa. The 100-yr return period storm load is used for design. The environmental loading only includes waves, other environmental loadings such as current and wind loading are neglected. The whole monopod configurations considered in this study are summarized in Table 1.

Figure 1. The wave data and the structure is located in the Java Sea.
Table 1. Monopod Configuration.

| Parameter           | Symbol | Units | Value |
|---------------------|--------|-------|-------|
| Water Depth         | h      | m     | 33    |
| Leg Diameter        | D      | cm    | 86    |
| Thickness           | t      | cm    | 1-1.5 |
| Topside Weight      | W      | kN    | 1000  |

The structure is assumed to fail by bending in the main leg. Besides bending, axial compression is also subjected to the main leg due to the weight of the beam.

2.2 Wave Data Analysis

Hourly significant wave data set for 60 years from the Java Sea is used as the raw data for this study. The raw data will be processed to obtain the relationship between the wave height and wave period, and also the 100-yr return period wave height. The statistic parameters and distribution of the data set are firstly determined before conducting the main process. Normal, Lognormal, Log Pearson III, and Gumbel distributions are selected for distribution fitting tests. These distributions are tested using the Kolmogorov-Smirnov test.

The wave period value follows the wave height. To obtain the relationship between the wave period and wave height, all hourly significant wave data are plotted against its significant wave period. A regression analysis took place to determine the equation that closely matches with the relationship between the two variables. Later, the equation will be used to determine the period of the 100-yr return period wave height.

Extreme value analysis is performed to obtain the 100-year return period wave height. The maximum significant wave height from each year is collected and sorted from the lowest to the highest value. Then the 100-year return period wave height is calculated by using the extreme analysis equation from the selected distribution. Since the data set is in the form of significant wave data, it needs to be converted into maximum wave height data. Chakrabarti (1996) proposed the relationship between significant wave heights and maximum wave height in equation (1) below.

$$H_{\text{max}} = 1.86 H_{1/3}$$  \hspace{1cm} (1)

The 100-yr return period wave height and the calculated wave period obtained from extreme value analysis will be used in designing the model. While the maximum significant wave height of each year will be used for static in-place analysis to determine the wave base shear equation.

2.3 Static Pushover Analysis

Static pushover analysis is performed on each monopod to obtain the structure's maximum base shear value right before collapsing ($BS_{\text{collapse}}$). This value is used for the reliability index value calculation. The Reserve Strength Ratio (RSR) value of the structure must exceed the minimum value of 1.60 for unmanned structures with high consequence of failure.

2.4 Static In-place Analysis

This analysis is performed to obtain the wave base shear function. The wave base shear function ($BS_{\text{wave}}$) is defined as the relationship between the structure's maximum base shear and the maximum wave height that have been converted from maximum significant wave height data for 60 years. The maximum base shear value of the structures is obtained from the in-place analysis, then it is plotted against the maximum significant wave height.

Equation (2) shows the wave base shear expression:

$$L = BS_{\text{wave}} = aH^b$$  \hspace{1cm} (2)
Where $H$ is the wave height, while coefficients $a$ and $b$ are determined from curve fitting.

2.5 Reliability Analysis

The reliability analysis uses maximum wave height data statistical parameters. First Order Reliability Method (FORM) II is used for reliability analysis. The collapse failure mode is chosen for performance failure. Safety margin between resistance and load is indicated by limit state equation as follows:

$$g = R - L \leq 0$$

Where $g$ is the limit state function. Here, the collapse base shear acted as the resistance variable ($R$), while the wave base shear acted as the load variable ($L$). By inserting equation (2), equation (3) becomes equation (4).

$$g = BS_{\text{collapse}} - aH^b \leq 0$$

Reliability analysis is performed using the performance function in equation (3) where the wave height carries the uncertainty value. So, the overall system uncertainty is strongly dependent on the wave height. The obtained reliability indexes need to be compared with the target values. DNV proposed a recommended target reliability index ($\beta$) of 3.71, or equivalent with a probability of failure of $10^{-4}$ for a non-redundant structure.

3. Results and Discussion

3.1 Design Wave Height

The Kolmogorov-Smirnov test results are shown in Table 2.

| Distribution | Maximum $D_n$ |
|--------------|---------------|
| Normal       | 0.0695        |
| Lognormal    | 0.0299        |
| Rayleigh     | 0.0683        |
| Gumbel       | 0.0236        |

The test result above shows that Gumbel distribution is the closest distribution form to the 60 years of hourly wave data distribution. The critical $D_n^{\alpha}$ for 2358 data with significance level of $15\%$ is 0.0242. It is concluded that the Gumbel distribution can be accepted to represent the wave data. The probability density of data and each distribution can be seen in Figure 2.
Figure 2. The probability density function of wave data and test distributions.

Figure 3. Hourly significant wave data set plotted against its period.

Figure 3 shows the relationship between the wave period and wave height that can be expressed in the equation (5) below.

\[ T(H) = 6.241 H^{0.4126} \]  

(5)

From the extreme value analysis using Gumbel distribution, we obtain the design wave height of 6.29 m, with a wave period of 11.47 s. These values will be used as the environmental load for structure design.
3.2 Optimized Structure Design

Five structures are obtained after optimally designed using different environmental load factor values that are shown in Table 3. One structure is designed based on WSD criteria, and the rest are designed using LRFD criteria with a different environmental load factor.

| Criteria | $\gamma_E$ | $D$ (cm) | $t$ (cm) |
|----------|------------|----------|----------|
| WSD      | 1.00       | 86       | 1.375    |
| LRFD     | 1.35       | 86       | 1.350    |
|          | 1.20       | 86       | 1.280    |
|          | 1.10       | 86       | 1.235    |
|          | 1.00       | 86       | 1.190    |

The UC value of each structure is checked using allowable combined stress in equation (6) and equation (7) for cylindrical members from API RP 2A WSD and API RP 2A LRFD respectively. Axial compression and bending stress are subjected in the leg member. A cylindrical member under combined compression and bending loads should be proportioned to satisfy the following requirements at all cross-sections along their length.

$$\frac{f_a}{F_a} + \frac{C_m \left( f_{bx}^2 + f_{by}^2 \right)^{1/2}}{\left( 1 - \frac{f_a}{F_a} \right) F_b} \leq 1.0$$  \hspace{1cm} (6)

where:
- $C_m$ = Reduction factor, for restrained end members, 0.85
- $F_a$ = Allowable axial compressive stress, ksi (MPa)
- $f_a$ = Acting axial stress, ksi (MPa)
- $F_b$ = Allowable bending stress, ksi (MPa)
- $f_{bx}, f_{by}$ = Bending stress corresponding to the member x and y-axis respectively, ksi (MPa)
- $F_e$ = Critical elastic buckling stress, ksi (MPa)

$$\frac{f_c}{\phi_c F_{cn}} + \frac{1}{\phi_b F_{bn}} \left( \frac{C_{my} f_{by}}{\left( 1 - \frac{f_c}{\phi_c F_{cy}} \right)} \right)^2 + \frac{C_{mz} f_{bz}}{\left( 1 - \frac{f_c}{\phi_c F_{cz}} \right)^2} \leq 1.0$$  \hspace{1cm} (7)

where:
- $C_{my}, C_{my}$ = Reduction factor corresponding to the member y and z-axis respectively, for restrained end members, 0.85
- $F_{bn}$ = Nominal bending stress, ksi (MPa)
- $f_{by}, f_{bz}$ = Bending stress corresponding to the member y and z-axis respectively, ksi (MPa)
- $F_{cn}$ = Nominal axial compressive strength, ksi (MPa)
- $f_c$ = Axial compressive strength due to factored loads, ksi (MPa)
- $F_{cy}, F_{cz}$ = Euler buckling stress corresponding to the member y and z-axis respectively, ksi (MPa)
- $\phi_b$ = Resistance factor for bending stress, 0.95
- $\phi_c$ = Resistance factor for axial compressive strength, 0.85
The applied stress subjected to the main leg member due to external loads and its allowable stress for each structure are shown in Table 4.

Table 4. The applied stress value and allowable stress value for each structure.

| Criteria | \(\gamma_E\) | \(D\) (cm) | \(t\) (cm) | Axial Stress (MPa) | Bending Stress Y (MPa) | Bending Stress Z (MPa) | Axial Stress (MPa) | Euler Stress (MPa) | Bending Stress Y (MPa) | Bending Stress Z (MPa) |
|----------|--------------|-----------|-----------|--------------------|----------------------|----------------------|--------------------|----------------|----------------------|----------------------|
| WSD      | 1.00         | 86        | 1.375     | 25.45              | 0                    | 92.05                | 60.7               | 60.7           | 233.08               | 233.08               |
| LRFD     | 1.35         | 86        | 1.350     | 28.44              | 0                    | 126.46               | 74.23              | 87.33          | 282.22               | 282.22               |
|          | 1.20         | 86        | 1.280     | 29.77              | 0                    | 118.26               | 74.35              | 87.48          | 278.6                | 278.6                |
|          | 1.10         | 86        | 1.235     | 30.7               | 0                    | 112.18               | 74.43              | 87.57          | 276.07               | 276.07               |
|          | 1.00         | 86        | 1.190     | 31.7               | 0                    | 105.67               | 74.51              | 87.66          | 273.36               | 273.36               |

The ratio of the applied stress against allowable stress and the UC value for main leg member of each structure are summarized in Table 5.

Table 5. Ratio of the applied stress against allowable stress and the UC value for main leg of each structure.

| Criteria | \(\gamma_E\) | \(D\) (cm) | \(t\) (cm) | Ratio | UC |
|----------|--------------|-----------|-----------|-------|----|
|          |              |           |           | Axial Stress | Bending Stress Y | Bending Stress Z |    |
| WSD      | 1.00         | 86        | 1.375     | 0.419  | 0.395 | 1.00 |
| LRFD     | 1.35         | 86        | 1.350     | 0.383  | 0.448 | 1.00 |
|          | 1.20         | 86        | 1.280     | 0.400  | 0.424 | 1.00 |
|          | 1.10         | 86        | 1.235     | 0.412  | 0.406 | 1.00 |
|          | 1.00         | 86        | 1.190     | 0.425  | 0.387 | 1.00 |

The table above shows that the bigger the environmental load factor, the bigger the bending stress ratio.

3.3 Pushover Analysis

The \(D/t\) ratio values of all structures are greater than 60 so local buckling due to axial compression should be considered in pushover analysis. Both elastic and inelastic local buckling should also be considered for all structures due to the value of thickness more than 6 mm. As shown in table 6, the greater the thickness value, the greater the base shear value needed to make the structure collapse. The value of the corresponding base shear can be called a collapse base shear \(BS_{\text{collapse}}\).

Table 6. Pushover analysis results of each structure consisting of RSR and \(BS_{\text{collapse}}\) value.

| Criteria | \(\gamma_E\) | \(D\) (cm) | \(t\) (cm) | RSR | \(BS_{\text{collapse}}\) (kN) |
|----------|--------------|-----------|-----------|-----|------------------|
| WSD      | 1.00         | 86        | 1.375     | 2.96| 47.44            |
| LRFD     | 1.35         | 86        | 1.350     | 2.89| 46.38            |
|          | 1.20         | 86        | 1.280     | 2.72| 43.77            |
|          | 1.10         | 86        | 1.235     | 2.62| 42.11            |
|          | 1.00         | 86        | 1.190     | 2.52| 40.44            |
The RSR values of all structures are above the recommended value of 1.60 for unmanned structure with high consequence of failure. All structure's failure is caused by local buckling in the main leg as figure out in structure deformation in figure 4.

Figure 4. Structure deformation in pushover analysis for each design criterion.
3.4 Wave Base Shear

Figure 5 shows the relationship between wave base shear and wave height values. The result shows that same maximum base shear values are produced from all environmental load factor values. This indicates that the maximum base shear value caused by the wave is only affected by the diameter of the structure.

![Figure 5. Maximum base shear plotted against wave height.](image)

Equation \( y \) shows the relationship between the two variables.

From the figure above, we obtain the wave base shear equation in equation (8).

\[
BS_{\text{wave}} = 0.4341 H^{2.37}
\]

This equation is used as the performance function component for reliability analysis.

3.5 Reliability Index

The performance function for reliability analysis can be seen in equation (9).

\[
g = BS_{\text{collapse}} - 0.4341 H^{2.37} \leq 0
\]

When conducting reliability analysis, collapse base shear \( BS_{\text{collapse}} \) performed as the deterministic variable, while wave height \( H \) is the uncertainty variable. Uncertainty parameters used in this analysis is the maximum wave height statistical parameter. The maximum wave height data distribution follows the Gumbel distribution. Statistical parameters such as mean and standard deviation are used in the FORM II calculation. The calculation results of the reliability index using FORM II are shown in Table 7.
Table 7. Reliability Index (β) and the probability of failure for each design criteria using FORM II.

| Criteria | γ | $D$ (cm) | $t$ (cm) | β  | PoF           |
|----------|---|----------|----------|----|---------------|
| WSD      | 1.00 | 86       | 1.375    | 3.77 | $8.087 \times 10^{-5}$ |
| LRFD     | 1.35 | 86       | 1.350    | 3.71 | $1.033 \times 10^{-4}$ |
|          | 1.20 | 86       | 1.280    | 3.55 | $1.905 \times 10^{-4}$ |
|          | 1.10 | 86       | 1.235    | 3.45 | $2.830 \times 10^{-4}$ |
|          | 1.00 | 86       | 1.190    | 3.34 | $4.231 \times 10^{-4}$ |

The results above show that the reliability index of the unbraced monopod structure designed with an environmental load factor of 1.35 meets the recommended reliability index value proposed by DNV for a non-redundant structure of 3.71. The size of the structure may give contributions to the structure’s safety.

4. Conclusion

The reliability index of the structure designed with API RP 2A LRFD environmental load factor value of 1.35 meets the requirement proposed of 3.71 for non-redundant structure. This indicates that the environmental load factor is sufficient for the unbraced monopod structure. However, this research uses only one topside load value and dimensions that are not too diverse. Further studies using varied topside weight values, dimensions, and more resistivity parameters need to be conducted.

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