ABSTRACT

As one realized that most of the platforms were designed for 15 to 20 years service life (based on fatigue design criteria and the capacity of the reservoir). However, although some of the platforms have exceeded their service life, no serious accident has occurred in relation with the integrity of the platform. This has brought the curiosity whether the used of this platform may still be continued. Also, once the reservoir is dried up, can such platform be relocated for a new field? These are among the questions rising worldwide.

This paper will illustrate the effect of corrosion to platform strength capacities in term of joint capacity. An example of the structure ultimate capacity calculation also presented known as reserve strength ratio. The platform’s Reserve Strength Ratio (RSR) calculation based on analysis where the allowable stress ratio is modified to obtain the best estimate of its ultimate capacity should performed when the result of the study on linear elastic based, found that platforms do not satisfy the requirement. This is also known as “fitness to purpose” study, where the structure may not fulfill the code requirement but still fit to be used.

1. INTRODUCTION

During their lifetime the platform strength capacity is decrease due to environment effect or the damaged cause by the accident. Corrosion gives significant contribution to reduce the platform capacity. At the certain time of their lifetime the strength capacity may fall below the safety requirement.

The pushover analysis can perform to determine whether the ultimate platform resistance exceeds the capacity of the platform. This condition quantified in term of reserve strength ratio (RSR) which defined by:

\[
\text{Reserve Strength Ratio} = \frac{\text{Ultimate Platform Resistance}}{\text{Design Load}}
\]

In this study we performed the nonlinear pushover analysis for ZULU-JUNCTION a steel jacket platform located at Java Sea.
2. STRUCTURE RELIABILITY

Engineering decisions must be made in the presence of uncertainties arising from inherent randomness in many design parameters, imperfect modeling and lack of experience. Indeed, it is precisely on account of these uncertainties and the potential risks arising therefore that safety margins provided by the specification of allowable stresses, resistance factors, load factors, and the like, are required in design. While strength and load parameters are nondeterministic, they nevertheless exhibit statistical regularity. This suggests that probability theory should furnish the framework for setting specific limits of acceptable performance for design.

The use of statistical methodologies has stopped at the point where the nominal strength or load was specified. Additional load and resistance factors, or allowable stresses, were then selected subjectively to account for unforeseen unfavorable deviations from the nominal values. However, probability theory and structural reliability methods make it possible to select safety factors to be consistent with a desired level of performance (acceptably low probability of unsatisfactory performance). This affords the possibility of more uniform performance in structures and, in some areas where designs appear to be excessively conservative, a reduction in costs.

Analysis of structure reliability start with assumption where the loads and resistance are random variables and the statistical information necessary to describe their probability laws is known. A mathematical model is first derived which relates the resistance and load variables for the limit state of interest. Suppose that this relation is given by

\[ g(X_1, X_2, \ldots, X_n) = 0 \]  

where \( X_i \) = resistance or load variable (replacing R’s and Q’s), and that failure occurs when \( g < 0 \) for any ultimate or serviceability limit state of interest.

Failure, defined in a generic sense relative to any limit state, does not necessarily cannot collapse or other catastrophic events. The safety is assured by assigning a small probability \( P_f \) to the event that the limit state will be reached, i.e.,

\[ P_f = \int \int f_1(x_1, x_2, \ldots, x_n) dx_1 dx_2 \ldots dx_n \]  

In which \( f_i \) is the joint probability density function for \( X_1, X_2, \ldots \), and the integration is performed over the region where \( g < 0 \).

In the initial applications of this concept to structural safety problems, the limit state was considered to contain just two variables; a resistance \( R \) and a load effect \( Q \) dimensionally consistent with \( R \). The failure event in this case is \( R - Q < 0 \) and the probability of failure is computed as,

\[ P_f = P(R < Q) = \int_{-\infty}^{0} F_R(x) f_Q(x) dx \]  

in which \( F_R = \) cumulative probability distribution function (c.d.f.) in \( R \) and \( f_Q = \) probability density function for \( Q \).
This provides a basis for quantitatively measuring structural reliability, such a measure being given by \( P_F \). It is tacitly assumed that all uncertainties in design are contained in the joint probability law \( f_X \) and that \( f_X \) is known. However, in structural reliability analysis these probability laws are seldom known precisely due to a general scarcity of data. In fact, it may be difficult in many instances to determine the probability densities for the individual variables, let alone the joint density \( f_X \). In some cases, only the first and second order moments, i.e. mean and variance, may be known with any confidence. Moreover, the limit state equation may be highly nonlinear in the basic variables. Even in those instances where statistical information may be sufficient to define the marginal distributions of the individual variables, it usually is impractical to perform numerically the operations necessary to evaluate Eq. 2.

The difficulties outlined above have motivated the development of first-order, second moment (FOSM) reliability analysis methods, so called because of the way they characterize uncertainty in the variables and the linearization performed during the reliability analysis. In principle, the random variables are characterized by their first and second moments. While any continuous mathematical form of the limit state equation is possible, it must be linearized at some point for purposes of performing the reliability analysis. Linearization of the failure criterion defined by Eq. 1 leads to:

\[
\begin{align*}
  z & \approx g(X_1^*, X_2^*, \ldots X_n^*) + \sum (X_i - X_i^*) \left( \frac{\partial g}{\partial X_i} \right) X_i^* \\
  & = g(X_1^*, X_2^*, \ldots X_n^*) + \sum (X_i - X_i^*) \left( \frac{\partial g}{\partial X_i} \right) X_i^* \\
  & = g(X_1^*, X_2^*, \ldots X_n^*) + \sum (X_i - X_i^*) \left( \frac{\partial g}{\partial X_i} \right) X_i^*
\end{align*}
\]

In earlier structural reliability studies, the point \((X_1^*, X_2^*, \ldots X_n^*)\) was set equal to the mean values \((\mu_{X_1}, \mu_{X_2}, \ldots \mu_{X_n})\). Assuming the X-variables to be statistically uncorrelated, the mean and standard deviation in \( Z \) are approximated by

\[
Z \approx g(\mu_{X_1}, \mu_{X_2}, \ldots \mu_{X_n})
\]

\[
\sigma_Z \approx \left[ \sum \left( \frac{\partial g}{\partial X_i} \right)^2 \sigma_{X_i}^2 \right]^{1/2}
\]

The extent to which Eqs. 5 and 6 are accurate depends on the effect of neglecting higher order terms in Eq. 4 and the magnitudes of the coefficients of variations of variation in \( X_i \). If \( g() \) is linear and the variables are uncorrelated, Eqs. 5 and 6 are exact.

The reliability index \( \beta \) (in some studies, \( \beta \) is termed the safety index) is defined by

\[\beta = \mu_Z/\sigma_Z\]

Illustrated by figure below:
In this development, no mention has been made of probability distributions; the reliability index $\beta$ depends only on measures of central tendency ($\mu$) and dispersion ($\sigma$) in the limit state function. However, it is important to realize that if the probability laws governing the variables in the limit state equation are known, there is a relation between $\beta$ and $P_f$. In the example just considered, if $R$ and $Q$ are normal and statistically independent, then $R-Q$ is normal with mean $\mu_R - \mu_Q$ and variance $\sigma_R^2 + \sigma_Q^2$. The safety index then:

$$\beta = \frac{(\mu_R - \mu_Q)}{\sqrt{(\sigma_R^2 + \sigma_Q^2)}}$$

(8)

It is important to realize that if the probability distributions for all design variables are known, then the safety index $\beta$ and $P_f$ are uniquely related, that is, for any prescribed set of means and uncertainties a unique $\beta$ corresponding to $P_f$ can be computed. Even when such distributions and probabilities cannot be obtained, however, $\beta$ is a useful comparative measure of reliability and can serve to evaluate the relative safety of various design alternatives, provided that the uncertainties are handled consistently. The reliability index $\beta$ is related to the percent point function of the standard normal distribution according to,

$$\beta = \Phi^{-1}(1-P_f)$$

$$P_f = \Phi(-\beta)$$

(9)

The probability of failure is then

$$P_f = \Phi\left[\frac{(\mu_Q - \mu_R)}{\sqrt{(\sigma_R^2 + \sigma_Q^2)}}\right]$$

(10)
The relationship between the probability of failure $P_f$ and safety index $\beta$ is constant, and independent of the type of distribution function. The relationship between the probability of failure and safety index for values in the range of interest are given in table below:

<table>
<thead>
<tr>
<th>$\beta$</th>
<th>$P_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.33</td>
<td>0.01000</td>
</tr>
<tr>
<td>2.58</td>
<td>0.00500</td>
</tr>
<tr>
<td>2.88</td>
<td>0.00200</td>
</tr>
<tr>
<td>3.09</td>
<td>0.00100</td>
</tr>
<tr>
<td>3.29</td>
<td>0.00050</td>
</tr>
<tr>
<td>3.54</td>
<td>0.00020</td>
</tr>
<tr>
<td>3.72</td>
<td>0.00010</td>
</tr>
<tr>
<td>3.89</td>
<td>0.00005</td>
</tr>
<tr>
<td>4.13</td>
<td>0.00002</td>
</tr>
<tr>
<td>4.27</td>
<td>0.00001</td>
</tr>
</tbody>
</table>

3. ACCEPTABLE PROBABILITY OF FAILURE

Three fundamental approaches to determination of the reliabilities that may be used in platform design and re-qualification are summarized here:

a. Economic approach

The fundamental premise of the economics based approach is that it is desirable to minimize the total initial cost ($C_i$) and future cost ($C_f$). Thus: minimize $(C_t = C_i + C_f)$

In term of the expected value it can be written:

\[
\text{Minimize } E(C_t) = E(C_i) + E(C_f)
\]  (11)

It can be generally drawn in the following figure:

![Figure 2: Probability of Failure and Cost Relation](image)
b. Utility Approach
This approach is based on the probability of failure that include not only the environmental hazard, but also operating hazard such as fires, explosions, blowouts, and collision that can cause a total failure of the platform. Based on World Offshore Accident Databank, the annual probability of accident for drilling and production platform is approximately $5.0 \times 10^{-4}$.

c. Historic and current standard of practice
Historic probability of failure in Gulf of Mexico = $10^{-3}$, West Coast Africa = $8 \times 10^{-4} - 10^{-3}$, North Sea = $4 \times 10^{-4} - 10^{-3}$.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Structural Safety Level (SSL)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High Loss Consequence (SSL-1)</td>
</tr>
<tr>
<td>Annual $\beta$ target</td>
<td>3.7</td>
</tr>
<tr>
<td>Annual Probability of Failure ($P_f$)</td>
<td>$10^{-4}$</td>
</tr>
<tr>
<td>Lifetime failure Probability for 20 years</td>
<td>$0.04 - 0.20%$</td>
</tr>
</tbody>
</table>

Platforms in Indonesia (as well as elsewhere in the world, as adopted by ISO19900 series) are categorized into the following $3 \times 3$ matrix, i.e.:

<table>
<thead>
<tr>
<th>Life Safety Category</th>
<th>Consequence of Failure Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High Consequence of Failure (C-1)</td>
</tr>
<tr>
<td>Manned non-Evacuated (S-1)</td>
<td>SSL-1</td>
</tr>
<tr>
<td>Manned Evacuated (S-2)</td>
<td>SSL-1</td>
</tr>
<tr>
<td>Unmanned (S-3)</td>
<td>SSL-1</td>
</tr>
</tbody>
</table>

Based on the statistic of the environmental data (wave and seismic activity) and the targeted $\beta$-value, we may then be able to derive the associated load factors. The expected inspection interval may also be designed accordingly.

**Life Safety Categories**
The category that applies to a structure shall be determined by the owner of the structure prior to the design of a new structure and shall be agreed by the regulator where one exists.

**Manned – Non evacuated (S-1)**
The manned-non-evacuated category refers to a platform that is continuously (or near continuously) occupied by persons accommodated and living thereon, and personnel evacuation prior to design environmental event is either not intended or impractical.
A platform shall be categorized as S-1 Manned-non-evacuated unless the particular requirements for S-2 or S-3 apply throughout the expected life of the platform.

**Manned – Evacuated (S-2)**

The manned-evacuated category refers to a platform that is normally manned except during a forecast design environmental event. For categorization purposes, a manned platform shall be categorized as a manned-evacuated platform only if:

- Reliable forecast of a design environmental event is technically and operationally feasible, and the weather any such forecast and the occurrence of the design environmental event is not likely to inhibit an evacuation.
- Prior to design environmental event, evacuation is planned
- Sufficient time and resources exist to safely evacuate all personnel from the platform and all other platform likely to require evacuation for the same storm.

**Unmanned (S-3)**

The unmanned category refers to a platform that is only manned for occasionally inspection, maintenance, and modification visits. For categorization purposes a platform shall be categorized as unmanned only if:

- Visits to the platform are undertaken for specific planned inspection, maintenance or modification operations on the platform itself.
- Visits are not expected to last more than 24 hours during seasons when severe weather may be expected to occur
- The criteria for S-2 manned – evacuated platforms are also met.

A platform in this category may also be described as not normally manned.

**Consequence of failure categories**

The degree to which negative consequences could result from platform collapse is a judgment which should be based on the importance of the structure to the safety of any personnel associated with the platform on a normal basis or in relation to the failure, the potential damage to the environment, the owner’s overall operation and the level of economic losses that could be sustained as a result of the collapse.

When considering the cost of mitigating of pollution and environment damage, particular attention should be given to the hydrocarbons contained in the topside process inventory, possible leakage of damaged wells or pipelines, and the proximity of the platform to the shoreline or to environmentally sensitive areas such as coral reefs, estuaries, and wildlife refuges. The potential amount of liquid hydrocarbons or sour gas released from these sources should be considerably less than the available inventory from each source. The factors affecting the release from each source are discussed below.

When considering economic losses, in addition to loss of the platform and associated equipment, and damage to connecting pipelines, the loss of reserves should be considered if the site is subsequently abandoned. Removal costs include the salvage of the collapse structure, reentering and plugging damaged wells, and cleanup of the sea floor at the site. If the site is not to be abandoned, restoration costs should be considered, such as replacing the structure and equipment and reentering the wells. Other costs include repair, rerouting or reconnecting pipelines to the new structure.

Consequences of failure should include consideration of:
a. Life safety of personnel on or near to the platform as may be brought in to react to any consequence of failure.

b. Damage to the environment

c. Anticipated losses to the owner, the other operators and the industry in general.

High consequences (C-1)
The high consequence of failure category refers to major platforms and or those platform that have the potential for well flow of either oil or sour gas in the event of platform failure. In addition, it includes platforms where the shut in of the oil or sour gas production is not planned, or not practical, prior to the occurrence of the design event (such as areas with high seismic activity). Platforms that support major oil transport lines and or storage facilities for intermittent oil shipment are also considered to be in the high consequence category.

A platform shall be categorized as C-1 High consequence unless the particular requirements for C-2 or C-3 apply throughout the expected life of the platform.

Medium consequence (C-2)
The medium consequence of failure category refers to platform where production will be shut in during the design event. The following criteria shall apply:

   a. All wells that could flow on their own in the event of platform failure shall contain fully functional, subsurface safety valves, which are manufactured and tested in accordance with applicable specifications
   
   b. Oil storage is limited to process inventory and “surge” tanks for pipeline transfer.
   
   c. Pipelines shall be limited in their ability to release hydrocarbons, either by virtue of inventory and pressure regime, or by check valves or seabed safety valves.

Low Consequence (C-3)
The low consequence of failure category refers to minimal platforms where production will be shut-in during the design event. These platforms may support production departing from the platform and low volume infield pipelines. The following criteria shall apply:

   a. All wells that could flow on their own in the event of platform failure shall contain fully functional, subsurface safety valves, which are manufactured and tested in accordance with applicable specifications
   
   b. Oil storage is limited to process inventory

Pipelines shall be limited in their ability to release hydrocarbons, either by virtue of inventory and pressure regime or by check valves or seabed safety valves.
4. STUDY CASE

In this study we performed the analysis on Zulu-Junction platform located at the Java Sea, 107 feet water depth. Structure model presented in figure below:

![Computer Model of the Structure](image)

**Figure 3: Computer Model of the Structure**

4.1. Corrosion Effect on the Platform Strength Capacity

The analysis conducted based on assumption that corrosion rate is 0.300 mm/year. Due to the corrosion, structure self weight also decrease as shown in figure below:

![Corrosion Effect on the Structure Self weight](image)

**Figure 5: Corrosion Effect on the Structure Self weight**
Corrosion on jacket members shall reduce the diameter and wall thickness of the tubular member. When joint can analysis performed using SACS the allowable stress decrease as the diameter and wall thickness decrease. Results of the analysis presented in following figures:

![Figure 6: Corrosion Effect on the Nominal Stress at the Members](image)

![Figure 7: Nominal Stress due to Dead and Live Load with the Allowable Stress related with Platform Lifetime](image)
Figure 8: Nominal Stress due to Dead, Live and Wave Load with the Allowable Stress related with Platform Lifetime

Statistic parameter of the above result represented in the tables below:

### Table 2: Nominal Stress at the Brace due to Dead and Live Load

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Nominal Stress</th>
<th>Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-Year</td>
<td>10-Year</td>
</tr>
<tr>
<td>Mean</td>
<td>-0.111</td>
<td>-0.113</td>
</tr>
<tr>
<td>Deviation Standard</td>
<td>0.308</td>
<td>0.312</td>
</tr>
<tr>
<td>Maximum Value</td>
<td>0.550</td>
<td>0.560</td>
</tr>
</tbody>
</table>

### Table 3: Nominal Stress at the Brace due to Dead, Live Load and Wave Load

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Nominal Stress</th>
<th>Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-Year</td>
<td>10-Year</td>
</tr>
<tr>
<td>Mean</td>
<td>-0.114</td>
<td>-0.115</td>
</tr>
<tr>
<td>Deviation Standard</td>
<td>1.567</td>
<td>1.704</td>
</tr>
<tr>
<td>Maximum Value</td>
<td>5.490</td>
<td>5.970</td>
</tr>
</tbody>
</table>
The results above indicate that the average joint can allowable stress will decrease around 8% in 10 years and around 15% for 20 years of the platform lifetime caused only by corrosion. Table 3 shows that maximum nominal stress will increase around 8.7% for 10 years and 20.2% for 20 years platform lifetime.

If we want to re-use this platform after reached their service life we need to reduce the load design or strengthening the platform to obtain the same acceptable risk. In the probability concept the design load related with the design life of the structure. When we must apply smaller load in a design we should have shorter design life.

4.2. Pushover Analysis

In this study, we perform static pushover analysis to investigate the reserve strength of the Zulu Junction Platform employing USFOS, a computer program for progressive collapse analysis of steel offshore structures. In the static pushover analysis, a load pattern, usually conform to code-specified static equivalent earthquake load, is selected. The static load, with the selected pattern, is applied incrementally. For each step the structure stiffness is assembled and the global displacement increment calculated. The element force increment is calculated by using the tangential stiffness matrix and the element displacement increment. At each level, elements are checked to see whether buckling or plastic capacity has been reached. If such event is predicted, the step is reduced to being the response to just reach that event.

The pushover analysis is performed in two stages. In the first stage, gravity load is applied to the structure in small increments until full factored gravity is applied. In the second stage, a lateral load pattern representing the earthquake load is applied incrementally until collapse load is reached. The structural responses in the first stage of the pushover analysis had been verified by comparing with result of a linear analysis employing SACS program.

![Figure 9: Pushover Loading versus Top Deck Displacement at X direction until Collapse Occur](image_url)
Linear analysis with SACS for 800-year earthquake spectrum gives the total base shear = 1550 kips. At this condition some of the piles already yield. This base shear doesn’t represent the actual structure capacity; SACS already reduce this base shear as a safety factor. The actual structure linear capacity is 1550/0.85 = 1829.41 kips. Non-linear analyses with USFOS show that the structure ultimate strength = 3215.99 kips. From these result the reserve strength ratio is around 1.75. This result show that the platform has large reserve strength in the form of over-strength ratio and ductility such that the platform will suffer only minor damage in the earthquake even with 800 years return period.

5. CONCLUSION

Existing structures strength assessment is important as a based of engineering decision to determine platform re-use and requalification. Data related with the platform operation needed in order to obtain the results that describe the structure condition. Decreasing of the structure capacity is a fact that hard to avoid. Justification on the working load that fit with the platform condition is need to be consider in platform re-use.

6. REFERENCE