# Safety Assessment of Fixed Steel Offshore Structures When Suffering Over-Design Environmental Loading in Vietnamese Sea Conditions

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**Abstract.** The demand for upgrading and life extension of existing fixed steel offshore structures in Viet Nam is a popular trend in recent years. Whereas the over-loading responses and local failure analyses of structures are the important problems. This paper presents research results on a new safety assessment method of the platform structures suffering over-design environmental loading while allowing members to work in nonlinear elastic - plastic phase into account fatigue crack effects. This study is developed based on reliability theory when structural geometry and material characteristics are considered as random variables and wave surface as stationary processes. These results can be used for the life extension assessment of existing offshore structures in Vietnamese sea conditions.

**Keywords:** Safety assessment · Fixed steel offshore structures
Over-Design environmental loading

## 1 Introduction

In current standards, platform assessment has already been implemented based on progressive collapse limit state (PLS) [1]. Moreover, there are some researches to consider to the interaction between global ultimate strength based on Reserve Strength Ratio (RSR) and fatigue crack effects using probability methods, such as [1–3]. However, RSR is considered corresponding to collapsed states, but the structures were no longer capable of operating at the lower conditions. The paper present a new method to perform reliability assessment of structures while they are suffering over design environmental loadings based on nondestructive condition for main member sections according to fully plastic and fatigue crack growth limitations.


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# 2 Safety Criterions for Fixed Steel Offshore Structures When Suffering Over-Design Environmental Loadings

## 2.1 Safety Condition According to Fatigue Crack Limitations

The crack size of a hotspot of fixed offshore structures increment due to a short- term sea-state represented by a significant wave height $H_{si}$ and a relative cycle number $N_i$ is determined as the following formula, using Paris law [4, 5]:

$$\Delta a_i = C.\left[\Delta\sigma_{eqi}.Y(a_{i-1}, t_d).\sqrt{\pi.a_{i-1}}\right]^m N_i \tag{1}$$

Herein, $\Delta\sigma_{eqi}$ is equivalent stress range of the sea-state. With assumption that crack only increases slowly, the stress range can be calculated as formula in [4] depended on effective stress ranges to give crack propagation, $\Delta\sigma_{ji}$. Whereas, $\Delta\sigma_{ji}$ can be calculated based on $\Delta K_{th}$ is threshold of intensity factor stress range; $a_{i-1}$ is crack depth of sea-state i − 1; $t_d$ is tubular thickness; C and m are material constants; $Y(a_{i-1}, t_d)$ is geometry function, calculated as formula in [3].

Assuming the crack size at any time is a Gaussian random variable, fatigue reliability can be expressed as:

$$P_m = P(a \le t_d) = 0.5 + \Phi(\beta_a);\ \beta_a = \frac{t_d - \mu_a}{\sqrt{Var(a)}} \tag{2}$$

Whereas, $\mu_a$ and $Var(a)$ are determined based on probability characteristics of $\Delta\sigma_{eqi}$ and $Y(a_{i-1}, t_d)$ using Taylor expansion.

Fatigue safety condition is $P_m \le [P]$. If the safety condition is satisfied at the end of a year, the crack size will be updated to the structures to re-analyze for the next year.

In NORSOK N-004, a crack on a tubular member section can be modeled as an equivalent dent depth $D_d$ by the formula as below, whereas, D is outside diameter; $A_c$ and A are crack and cross section area:

$$\frac{D_d}{D} = \frac{1}{2}\left(1 - \cos\pi\frac{A_c}{A}\right) \tag{3}$$

## 2.2 Safety Condition According to Fully Plastic States of Cross Sections

Strength safety condition is evaluated by fully plastic states of cross sections of main members. Fully plastic conditions for tubular member sections, according to perfectly plastic model, can be express as below [6].

For intact cross sections:

$$\Gamma_{b1} = \frac{\sqrt{M_y^2 + M_z^2}}{M_P} - \cos\left(\frac{\pi}{2}\frac{N}{N_P}\right) = 0 \tag{4}$$

For cracked sections:

$$\Gamma_{b2} = \left( (\frac{M_y}{M_{Py}})^2 + (\frac{M_z}{M_{Pz}})^2 \right)^{1/2} - \left(f_1^2 + f_2^2\right)^{1/2} = 0 \tag{5}$$

Whereas N, $M_y$, $M_z$ are axial force and moment components corresponding to y and z axis; $N_p$, $M_p$, $M_{py}$, $M_{pz}$ are plastic axial force, plastic moment of intact members, plastic moments around y and z axes of cracked members; $f_1$, $f_2$ are factors depended on axial force and dent depth of the cross sections [6].

In this paper, safety condition of the structural strength is evaluated by probabilistic model with using Monte Carlo method, when steel elastic modulus, yield limit, diameters and thicknesses of main members are considered as random variables. To reduced number of trials, a method is proposal to replace the fully plastic functions $\Gamma_{b1}$ and $\Gamma_{b2}$ by polynomial functions of the random variables according to regression method which uses the ordinary least squares model [7].

Strength safety probability can be predicted through the minimum probability of the most dangerous cross section, $P_b$, which is expressed as the following formula:

$$P_b = P(\Gamma < 0) = \frac{n_1}{n} \tag{6}$$

Whereas $n_1$ and n are number of trials for $\Gamma < 0$ and total of number of trials.

## 3 Prediction of Over-Design Environmental Condition Probability and Risk Assessment

According to Borgman (1963) [8], probability of a over-design wave height value $H^T_{max}$ with return period T in every year can be expressed as below:

$$P(H \geq H^T_{\max}) = \frac{1}{T} \tag{7}$$

Based on (6) and (7), risk probability can be written as the formula below:

$$P_r = (1 - P(\Gamma < 0))P(H \geq H^T_{\max}) = \left(1 - \frac{n_1}{n}\right)\frac{1}{T} \tag{8}$$

The risk probability will be increased in time due to effects of fatigue cracks. So extended time of the structures is depended on the fatigue problem with an acceptable risk probability.

# 4 A Vietnamese Application

## 4.1 Input Data

- **Structural data**

To consider a typical jacket platform structure with four main legs, piles are driven inside legs (Fig. 1). Some main parameters are assumed in the Table 1.

**Table 1.** Platform data assumption

| Items | Parameter |
|---|---|
| Platform function | Living quarter |
| Topside dimension | 40 × 20 × 9 (m) |
| Topside weight | 800 (T) |
| Legs cross section | 813 × 20.6 (mm) |
| Pille cross section | 711 × 19 (mm) |
| Max. Brace cross section | 609 × 12.7 (mm) |

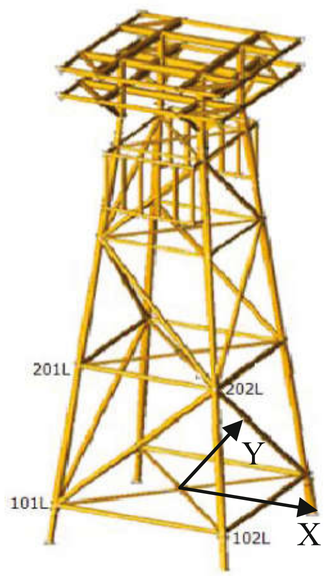


**Fig. 1.** Structural model

Steel grade API 5L, Expected value $\mu_{Fy}$ = 345 MPa, $\mu_E$ = 2.1 × $10^5$ MPa, Standard deviation $\mu_{Fy} = 6\%\ \mu_{Fy}$, $\sigma_E = 5\%\ \mu_E$. Outside diameter deviation of leg members = 0.5% of the expected value. Thickness deviation of leg members = 1% of the expected value. Fatigue S-N curve: API X. Fracture Parameters: $K_{th}$ = 3.15 MPa$\sqrt{m}$; $K_c$ = 15 MPa.$\sqrt{m}$; C = 1.46 × $10^{-11}$; m = 5.03.

- **Environmental data of Su Tu Nau Field in Vietnamese sea area**

Water depth at MSL: 45.6 m; Maximum tide amplitude +2.0 m; Minimum tide amplitude −2.5 m. Maximum wave height is 14.78 m, associated period is 12.79 s, corresponding to East direction with return period of 100 years. The relative current velocities are 1.4 m/s at surface and 0.73 m/s at seabed. Fatigue wave data is shown in Table 2, with P-M spectrum.

## 4.2 First Stage Fatigue Analysis Results

Firstly, to predict locations of cracks, the first stage fatigue analysis will be performed according to Palmgren-Miner rule using spectral method in SACS Software. The minimum fatigue life is approximately 27 years, at crown-point on chord of joint 202L (Fig. 1). So the joint will be chosen to analyze crack propagation after the fatigue life to lengthen operating time of the structure.

**Table 2.** Fatigue wave data for a year

| Hs (m) | 45° | 90° | 135° | 180° | 225° | 270° | 315° |
|---|---|---|---|---|---|---|---|
| 0.5 | 0.0723 | 0.0137 | 0.0016 | 0.0022 | 0.0322 | 0.0205 | 0.0347 |
| 1.0 | 0.0785 | 0.0209 | 0.0018 | 0.0019 | 0.0614 | 0.0006 | 0.0066 |
| 1.5 | 0.0516 | 0.0199 | 0.0010 | 0.0020 | 0.0856 | 0.0001 | 0.0008 |
| 2.0 | 0.0388 | 0.0144 | 0.0003 | 0.0014 | 0.0803 | 0.0000 | 0.0001 |
| 2.5 | 0.0192 | 0.0054 | 0.0000 | 0.0007 | 0.0661 | 0.0000 | 0.0000 |
| 3.0 | 0.0050 | 0.0003 | 0.0000 | 0.0003 | 0.0462 | 0.0000 | 0.0000 |
| 3.5 | 0.0009 | 0.0000 | 0.0000 | 0.0000 | 0.0265 | 0.0000 | 0.0000 |
| 4.0 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0145 | 0.0000 | 0.0000 |
| 4.5 | 0.0000 | 0.0000 | 0.0000 | 0.0000 | 0.0069 | 0.0000 | 0.0000 |

*Note: 0° direction coincides with the positive X-axis.*

## 4.3 Fatigue Crack Propagation Analysis and Assessment on Structural Extension Time

Initial crack depth of joint 202L is assumed to be 1 mm. Due to threshold limit of crack propagation, equivalent stress ranges are not stationary processes so probability characteristics must be determined with a large enough number of the stress records. Since the stresses results, the expected values and variances of crack depth a are calculated at the end of each year. Table 3 shows the crack size results in 7$^{th}$ and 8$^{th}$ years, the maximum extended life time is 8 years with fatigue safety probability $P_m = 0.9993$.

**Table 3.** Crack size prediction results

| Year | $a_i$ (m) | Hs (m) | $\mu_{\Delta\sigma}$ (MPa) | Var($\Delta\sigma$) (MPa$^2$) | Average of N in a year | $\mu_a$ (m) | $\sqrt{Var(a)}$ (m) |
|---|---|---|---|---|---|---|---|
| 7 | 0,00520 | 4,5 | 79,385 | 5,033 | 146,82 | **0,00759** | **0,000223** |
| | | 3,5 | 64,868 | 1,530 | 341,38 | | |
| | | 2,5 | 53,135 | 1,357 | 39,60 | | |
| 8 | 0,00837 | 4,5 | 74,628 | 8,857 | 222,09 | **0,01371** | **0,000467** |
| | | 3,5 | 61,258 | 0,348 | 535,35 | | |
| | | 2,5 | 53,135 | 1,357 | 39,60 | | |

## 4.4 Safety Assessment in Extended Life Time

- ***First case***

At the end of 7$^{th}$ year, the crack depth of chord member at joint 202L can be estimated by the expected value +3,3*standard deviation = 8,3 mm.

To simplify the calculations, random variables in input data are transformed into normal forms as below:

$$\overline{H}_{\max}^{T} = \frac{H_{\max}^{T} - \mu_{H_{\max}^{T}}}{\sqrt{Var(H_{\max}^{T})}}; \overline{E} = \frac{E - \mu_E}{\sqrt{Var(E)}}; \overline{F}_y = \frac{F_y - \mu_{F_y}}{\sqrt{Var(F_y)}}; \overline{D}_{i_1} = \frac{D_{i_1} - \mu_{Di_1}}{\sqrt{Var(D_{i_1})}}; \overline{t_{i_1}} = \frac{t_{i_1} - \mu_{t_{i_1}}}{\sqrt{Var(t_{i_1})}}$$

Based on 78 sets of selected values in these above variables of 78 trials, USFOS software is used to determine Γ values of the most dangerous cross section. Then the equivalent fully plastic, with squares sum of errors 0,004, can be written as:

$$\begin{aligned}\Gamma = & -0,580 + 0.086\overline{H}_{\max} - 0,040\overline{F}_y - 0,0700\overline{t}_2 - 0,025\overline{t}_3 + 0,033\overline{H}_{\max}^2 + 0,0100\overline{F}_y^2 \\ & + 0,070\overline{t}_2^2 + 0,035\overline{t}_3^2 - 0,081\overline{H}_{\max}\overline{F}_y - 0,021\overline{H}_{\max}\overline{E} - 0,021\overline{H}_{\max}\overline{D_1} - 0,021\overline{H}_{\max}\overline{D_2} \\ & - 0,021\overline{H}_{\max}\overline{D_1} - 0,021\overline{H}_{\max}\overline{D_4} - 0,034\overline{H}_{\max}\overline{t_1} - 0,003\overline{H}_{\max}\overline{t_2} + 0,011\overline{H}_{\max}\overline{t_3} - 0,021\overline{H}_{\max}\overline{t_4} \\ & - 0,130\overline{F}_y\overline{t_2} + 0,020\overline{F}_y\overline{t_3} + 0,060\overline{F}_y\overline{t_4} + 0,01\overline{E}\overline{t_2} + 0,01\overline{D_2}\overline{t_2} - 0,01\overline{t_1}\overline{t_3} - 0,05\overline{t_2}\overline{t_3}\end{aligned} \quad (9)$$

Strength safety probability of the cross section is assessed related to each over-design wave height value. Summarized results are shown in Table 4 as below. Herein, safety probability is 0,992 in relation with $H_{max}$ = 19,4 m, the number of trials is 226430 and precision of Monte Carlo simulation is 99,9%. At the end of $7^{th}$ year the structure can resist maximum wave height of 19,1 m with risk probability $P_r$ = 0%.

**Table 4.** Strength safety assessment and risk probability results – $7^{th}$ year

| No. | $H_{max}$ (m) | T | P(H ≥ $H_{max}$) | P(Γ < 0) | $P_r$ |
|---|---|---|---|---|---|
| 1 | 19,1 | 1200 | 0,00083 | 0,9999 | 0,00000 |
| 2 | 19,4 | 1400 | 0,00071 | 0,9920 | 0,00001 |
| 3 | 20 | 1900 | 0,00053 | 0,8210 | 0,00010 |

- ***Second case***

At the 8th year, the crack depth can be estimated by 15,25 mm. Similar to $1^{st}$ case, the equivalent fully plastic of the most dangerous cross section, with squares sum of errors 0,007, can be written as:

$$\begin{aligned}\Gamma = & -0,580 + 0.086\overline{H}_{\max} - 0,040\overline{F}_y - 0,0700\overline{t}_2 - 0,025\overline{t}_3 + 0,033\overline{H}_{\max}^2 + 0,0100\overline{F}_y^2 \\ & -0,025\overline{t}_3^2 - 0,078\overline{H}_{\max}\overline{F}_y - 0,005\overline{H}_{\max}\overline{E} - 0,005\overline{H}_{\max}\overline{D_1} - 0,005\overline{H}_{\max}\overline{D_2} - 0,005\overline{H}_{\max}\overline{D_3} \\ & -0,005\overline{H}_{\max}\overline{D_4} - 0,014\overline{H}_{\max}\overline{t_1} + 0,005\overline{H}_{\max}\overline{t_2} - 0,033\overline{H}_{\max}\overline{t_3} - 0,005\overline{H}_{\max}\overline{t_4} + 0,02\overline{F}_y\overline{t}_2 + 0,02\overline{F}_y\overline{t}_3 \\ & -0,01\overline{F}_y\overline{t}_4 + 0,01\overline{E}\overline{t_2} + 0,02\overline{E}\overline{t_3} - 0,01\overline{D}_1\overline{t}_2 + 0,02\overline{D}_1\overline{t}_3 + 0,01\overline{D}_2\overline{t}_2 + 0,02\overline{D}_2\overline{t}_3 + 0,01\overline{D}_3\overline{t}_2 \\ & + 0,02\overline{D}_3\overline{t}_3 + 0,02\overline{D}_4\overline{t}_2 + 0,02\overline{D}_4\overline{t}_3 + 0,01\overline{t_1}\overline{t_2} + 0,02\overline{t_1}\overline{t_3} + 0,01\overline{t_2}\overline{t_3} - 0,03\overline{t_2}\overline{t_4} + 0,02\overline{t_3}\overline{t_4}\end{aligned} \quad (10)$$

In this case, the summarized results are shown in Table 5 as below. Herein, safety probability is 0,992 in relation with $H_{max}$ = 19,1 m, the number of trials is 131033 and precision of Monte Carlo simulation is 99,9%. At the end of $8^{th}$ year the structure can resist maximum wave height of 18.5 m with $P_r$ = 0%.

**Table 5.** Strength safety assessment and risk probability results – $8^{th}$ year

| No. | $H_{max}$ (m) | T | $P(H \geq H_{max})$ | $P(\Gamma < 0)$ | $P_r$ |
|---|---|---|---|---|---|
| 1 | 18.5 | 800 | 0.00125 | 0.9999 | 0,00000 |
| 2 | 19.1 | 1200 | 0.00083 | 0.9920 | 0,00001 |
| 3 | 19.4 | 1400 | 0.00071 | 0.8200 | 0,00019 |

The results of the $1^{st}$ case and $2^{nd}$ case have shown the effects of fatigue cracks on structural strength and extended time over design life. At the end of $7^{th}$ year the structure can resist maximum wave height of 19.1 m with $P_r$ = 0% but after 1 year the structure only can resist maximum wave height of 18.5 m. The suitable extended time is 8 year with safety probability is 0.9993 and is decided by fatigue conditions.

## 5 Conclusion

This article presents a method to assess existing fixed steel offshore structures working in over-design conditions in extended life time and over-design environmental loading. The method is based on fatigue analysis in crack propagation phrase and strength analysis in fully plastic phrase of main member sections according to probabilistic models.

Large complexity due to nonlinear responses and a large number in random variables of structural and environmental data can make certain errors. To reduce the time and errors of the analysis, it is necessary to study on the computational automation.

The method can be applied for real conditions in Vietnam with using reliable survey data and suitable safety factors.

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