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Reliability Analysis of Rolled Steel Beams in Offshore Platforms

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ABSTRACT

This study presents the structural reliability of rolled steel beams on offshore platforms exposed to corrosion during their design period and beyond. The rate of corrosion of the rolled steel beam was determined using a standard expression for extreme marine environment. Limit equations were used to evaluate the shear performance, deflection, and resistance moment of the rolled steel beams during their design period. The exposure to corrosion was evaluated using the First Order Reliability Method (FORM) in MATLAB with FERUM Version 4.0 (Finite Element Reliability Using Matlab). An increasing decline in the shear, resistance moment, and deflection was observed as the cross sectional area decreased due to corrosion, which is consistent with experimental data. However, it was also noted that the design formulation is robust enough to exceed the design life prediction in the code.

1. Introduction

Decades ago, extensive research projects have been conducted to investigate and evaluate the resistance of offshore structures to corrosion. This work consists of analytical studies and a series of tests on full size, rolled steel beam. As a result, design recommendations were prepared which are incorporated in the new AISC Design specifications [1].

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Rolled steel beams are typically I-beam which are extruded structural steel sections'. In some cases, the beam may be formed in a Z-shape or W-shape rather than I-shape. Rolled steel beams became popular in the late 1800's, when they were used in construction of railroad bridges. The sections were joined together using angles and rivets to obtain girders of desired size. By 1950's welded girders replaced riveted and bolted girders in developed world due to their better quality, aesthetics and economy [2].

Rolled steel beams can often be loaded beyond the web buckling load predicted by the classical plate buckling theory. This is with respect to the fact that the web is framed by flanges and transverse stiffeners, which allow for a redistribution of stress. Since a girder web is usually subjected to bending, shear, or a combination of the two, the stress redistribution will be summarized for these three cases [1]. By the use of structural health monitoring of offshore platforms, the prediction of possible damages and determination of structure's health can be achieved, which is valuable for such important structures. Several structural health monitoring techniques have been used in the past for offshore platforms [3].

It is with this notion that the stochastic evaluation of the resistance moment of rolled steel beams in offshore platforms is carried out in this work.

2. Literature review

2.1. Corrosion effect on offshore platforms

Corrosion of steel in offshore platforms is an electrochemical process which can be initiated by one of two modes; Chloride Induction corrosion or Carbonation Induction corrosion [4]. Basically, when steel comes in contact with sea water, the sea water acts as an electrolyte and the different potentials between different parts of the steel structure cause metal ions to move from the surface of the structure and diffuse into the electrolyte solution. Here, they react with oxide and hydroxide ions to form corrosion products. As a result of this process, pits can form at the metal surface of the platform and corrosion within these pits as well as within crevices, such as the structure's joints or imperfections in welding, occurs as an irregular *corrosion front* on the steel surface of the platform. Over time, the increased stresses caused by the pits, crevices, and other structural anomalies lead to fractures and breaks within the structure [5]. The major problem oil companies' face is corrosion as it leads to the creation of extremely unsafe working conditions. Corrosion can cause losses as it can break machines and platforms, causing businesses to lose time and money as they await repairs or new structures. At worst, corrosion can result in equipment failure and accidents on the rig [5].

The primary concern with corrosion of steel on offshore platform is not the loss of cross sectional area because offshore platforms are designed in such a way that the amount of steel used most of the time far exceeds that which is required for strength. As corrosion progresses and iron (Fe) reacts to form ferrous and ferric oxides (FeO_2 and FeO_3), a significant change in volume occurs. The change in volume associated with the corrosion process results in a buildup of pressure, which ultimately results in failure of the platform [6]. The structural strength of the steel in any type of offshore structure is provided by the residual cross-section that is not affected by rust.

2.2. Philosophy of structural reliability

Reliability theory in structural engineering deals with the rational treatment of uncertainties in structural engineering and with the methods for assessing the safety and serviceability of structures [7]. Reliability analysis is a probabilistic and numerical approach to determine a safety level of a structural element or a system. Reliability is defined as a probability of a system (or a structure, in structural engineering) to functionally perform under given conditions over a specified period of time (Ghasemi and Nowak, 2017). In other words, structural reliability is concerned with the calculation and prediction of a limit state violation for engineering structures at any stage during their lifetime [8]. The violation of a limit state is the attainment of an undesirable condition for the structure, that is, damage to a part of the structure (serviceability limit state) or total collapse of the structure (ultimate limit state) which could lead to loss of human lives.

2.2.1. Reliability analysis method

There are a handful of methods applied to determine if a structure satisfies the performance criteria. Some of the commonly used methods include; Limit state, Reliability index, First Order Second Moment methods, Hasofer-Lind Reliability index and Rackwitz and Fiessler procedure. These methods are discussed briefly in the following sub-sections.

2.2.1.1. Limit state

The concept of a limit state is used to help define failure in the context of structure reliability analyses. A limit state is the boundary between the desired and the undesired performance of a structure. For an offshore structure, an undesired performance is loss of ability to sustain design load. The undesired performance can include collapse of the offshore structure. Limit states can be divided into two categories; Ultimate limit states and Serviceability limit state (Ferrand, 2005).

Ultimate Limit States (ULS) are mostly related to the loss of load carrying capacity. When an Ultimate Limit State (ULS) is exceeded, a catastrophic failure of the structure occurs, such as collapse or loss of operability. ULSs can be the formation of a plastic hinge, crushing of concrete, buckling or loss of stability. These are the limit states considered in a reliability-based design code (Ghasemi and Nowak, 2017).

Serviceability Limit States (SLSs) are related to gradual degradation and user’s comfort. These limit states are usually not associated with an immediate structural collapse. SLSs can be an excessive bending on an offshore structure during its design life (Ferrand, 2005).

The acceptability criteria are often based on engineering judgment (arbitrary decision). For example, consider a beam that fails if the moment due to the loads exceeds the moment carrying capacity. Then the corresponding limit state function can be written as follows:

$$g(x) = g(x_1, x_2, x_3, \dots, x_n) = R - Q \quad (1)$$

Where; R represents the resistance (moment carrying capacity), Q represents the load effect (total moment applied) and x_n represents the random variables of load and resistance such as dead load, live load, length, depth, etc.

The limit state function represents the boundary beyond which the structure no longer functions (Ferrand, 2005). The probability of failure, P_f, is equal to the probability that the undesired performance will occur. Mathematically, this can be expressed in terms of the limit state function as Equation (2):

$$P_f = P(R - Q < 0) = P(g(x) < 0) \quad (2)$$

If both R and Q are continuous random variables, then each has a Probability Density Function (PDF). Furthermore, R - Q is also a random variable with its own PDF. The probability of failure is given by Equation (3):

$$P_f = \int_{-\infty}^{+\infty} F_R(x_i)\, f_Q(x_i)\, d\, x_i \quad (3)$$

Where: $F_R(x)$ is the Cumulative Density Function (CDF) of resistance R and $f_Q(x_i)$ is the PDF of the load Q.

2.2.1.2. Reliability index

An official definition of the reliability index is that it represents the shortest distance from the origin of standard space (reduced variable space) to the limit state line $g(Z_R, Z_Q) = 0$, in the reduced variables space, where Z_R is the reduced random variable for resistance and Z_Q is the reduced variable for load (Ferrand, 2005). The reduced form of a random variable, *X*, is given by Equation (4):

$$Z_x = \left| \frac{X - \mu_x}{\sigma_x} \right. \quad (4)$$

2.2.1.3. Simulation techniques

In some cases, the methods for the computation of reliability stated above can become very complicated. This happens especially when the limit state function is very complex or cannot be expressed in a closed form. In these situations, simulation methods are used. Examples of simulation methods used in computation of reliability includes; Monte Carlo Simulation (MCS) and Rosenblueth's 2K + 1 Point Estimate Method.

2.3. Probabilistic design

The deterministic design criteria in Equation (2.8) describes that the resistance should be greater than the load effect parameter to avoid a failure. The problem becomes probabilistic when statistical distributions of the variables are taken into account. There are three detail levels when talking about theoretical probabilistic methods. The first method considers the uncertainty with one parameter per uncertain variable. The generally accepted *Partial Coefficient Method* (PCM) that is used in Eurocodes falls into this category. The PCM is calibrated against other probabilistic methods and is really not a probabilistic method. The next method, called *First Order Reliability Method* (FORM), considers the uncertainty with two parameters, mean value and variance, for each uncertain parameter. A measure of the safety is given by the safety index, β. The risk of failure could be estimated by [9]:

$$P_f = \phi\,(-\beta) \tag{5}$$

The third and most fundamental theory utilizes the exact statistical properties for all variables. This method gives the most realistic measure of the probability of failure, P_f and often requires numerical methods to be solved. One method which utilizes the exact statistical properties of all variables is the *Monte Carlo method* (MC). This is really a simulation method which randomly picks numbers in pair from the **R** and **S** distributions. Each pair of numbers is compared in between. Now, let **R** be the resistance of the structure and **S** the load on the structure. If failure occurs, when **R** < **S**, the result is 1 otherwise 0. Mathematically, this will be expressed by an indicator function, I ( ), which returns 1 if the failure function is less than zero, G (x) ≤ 0, otherwise it returns 0. This leads to an approximation about the failure probability, P_f, and variance, given by [9]:

$$P_f \approx \acute{P}_f = \frac{1}{N} \sum I\left(G(x)\right) \leq 0 \tag{6}$$

$$\mathrm{Var}\left|\acute{P}\right| = \frac{1}{N} P_f(1 - P_f) \tag{7}$$

2.4. First order reliability method (FORM)

The First-Order Reliability Method (FORM) is designed to provide approximations of probability integrals occurring in structural reliability**.** Considering the Figure 1, the probability that the load effect, S, falls into an infinitesimal interval *ds* at s which is f_S (s).ds. Also, the probability that *R* falls in or under this interval is $\int_{-\infty}^{S} f_R\ (r)\ dr$. The probability that *S* falls in this interval *ds* when $R \leq S$ will result to $f_S\ (s).ds.\ \int_{-\infty}^{S} f_R\ (r)\ dr$.

The expression gives the probability that $R \leq S$ is:

$$P_f = P\,(R\text{-}S) \leq 0 = \int_{-\infty}^{\infty} f_S\ (s)\ .\ [\int_{-\infty}^{\infty} f_R\,(r).\ dr]\ ds = \int_{-\infty}^{\infty} f_S\ (s)\ f_R\,(r) ds \tag{8}$$

The equation (3.1) could be viewed upon as the volume of the two dimensional joint density functions from the failure surface, and can be seen in the Figure 1 below. Where, R and S are plotted as probability functions on the *r* and *s* axes. The limit state equation, G = R -S = 0, separates the safe from the unsafe region, dividing the volume into two parts. The volume of the part cut away and defined by $s > r$ corresponds to the probability of failure. The design point (r^*; s^*) is located on this straight line where the joint probability density is greatest. If failure is to occur it is likely to be there. The desired safe state is above the failure surface, defined as *R*-*S*=0, and undesired failure state is below (Bergstrom, 2006).

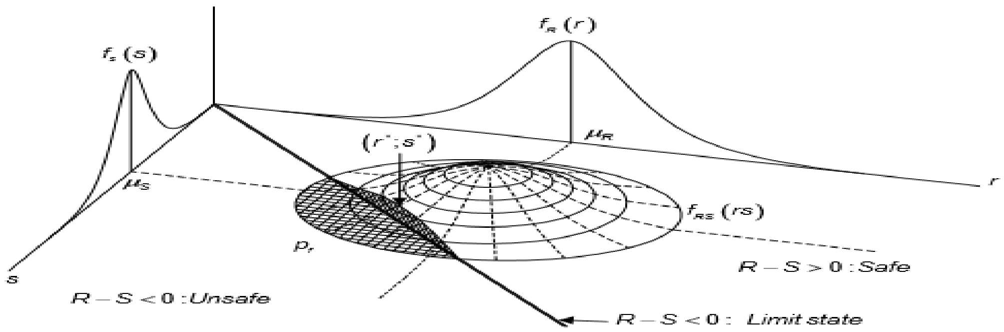

Fig. 1. The 3-D view of Two Random Joint Density Function f_{RS}(rs), (Bergstrom, 2006).

If R and S are normally distributed and statistically uncorrelated, then:

$R \in N$ ($M_{R,} \sigma_R$)

$S \in N$ (M_S, σ_S)

The safety margin, M is defined as:

M = R – S (9)

Then it is also valid that:

M $\in N$ (M_M, σ_M)

Where $M_M = M_R - M_S$ and $\sigma_{M} = \sqrt{\sigma}_{R} + \sigma_S$ (10)

The distribution of M is schematically given in Figure 2 below. Failure occurs when M <0

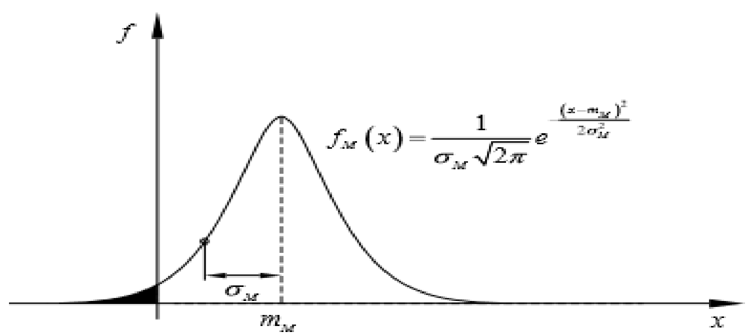

Fig. 2. Distribution for the variable M. Failure occurs when M < 0. (Bergstrom, 2006).

P_f = P (M < 0) = $\int_{-\infty}^{0} f_M\ (x)\ dx = \frac{1}{\sigma_{M\sqrt{2\pi}}} \int_{-\infty}^{0} e^{\frac{x-m_M}{2\sigma^2 M}}$ dx (11)

Now let,

$\frac{x-m_M}{\sigma_M}$ = y $\rightarrow \frac{1}{\sigma_M}\ dx = dy$ (12)

P_f = P (M < 0) =$\frac{1}{\sqrt{2\pi}} \int e^{\frac{y^2}{2}}$ dy = $\phi\left(-\frac{m_M}{\sigma_M}\right) = \ \phi(-\beta)$ (13)

ϕ (.) is the standardized normal distribution function. This function, given in Figure 3 below, has mean value 0 and standard deviation 1. If *R* and *S* are normally distributed, the safety index, β, is calculated as:

$$\beta = \frac{m_M}{\sigma_M} = \frac{m_R - m_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \tag{14}$$

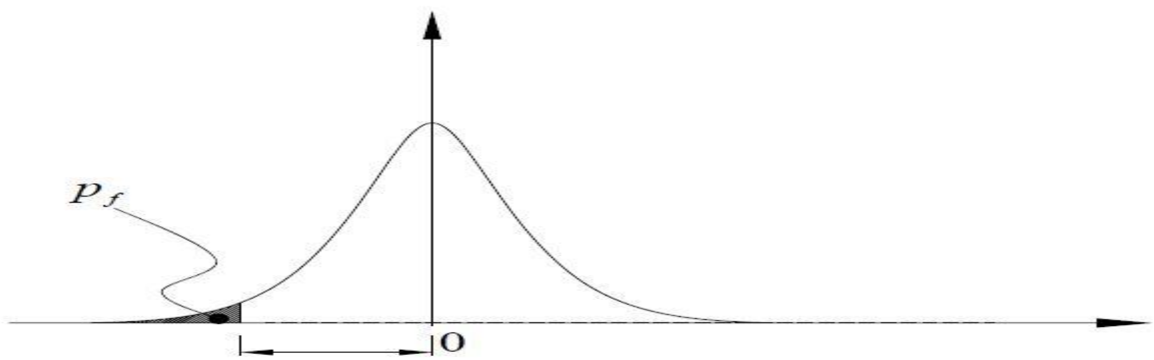

Fig. 3. The Standardized Normal Distribution Function Safety Index β, is connected to a certain Probability of Failure, P_f.β (Bergstrom, 2006).

3. Methodology

3.1. Standard expression for corrosion rate

In most cases, aside from contamination problems, the primary concern is the life (usually life in years) of the equipment involved. A good corrosion rate should involve;

(i) familiar units
(ii) Easy calculation with minimum opportunity for error
(iii) Ready conversion to life in years
(iv) Penetration
(v) Whole numbers without cumbersome decimal.

The formula for calculating corrosion rate is:

$$\frac{mm}{years} = 87.6\ \frac{W}{DAT}$$

Where; *W* is the weight loss in milligrams

D is the density in g/mm^3

A is the area in mm^2

T is the time of exposure in hours

3.2. Performance function for reliability analysis

Performance-based design (PBD) is a design method implemented for civil engineering structures undergoing severe loading events such as seismic loads, and it is based on achieving certain performance objectives when the structure is loaded. The ettle of this design approach is to obtain equilibrium between cost and structural performance while also considering the importance of the structure [10].

Table 1 gives the essential design parameters for the stochastic analysis used in the probabilistic determination of the safety criteria of the steel beams:

Table 1
Parameters of Stochastic Model

S/N	Basic Variable	Parameter
1	Modulus of elasticity, E	210,000 N/mm²
2	Shear modulus, G = E/[2(1 + v)]	often taken as 81,000 N/mm²
3	Poisson's ratio, v	0.3
4	Coefficient of thermal expansion, α	12 x 10^{-6}/°C (in the ambient temperature range).

3.2.1. Bending criterion

An assessment is made on a typical simply supported beam and assuming ACI 318, (2002) design code for loading. The ACI code prescribes the highest loading on offshore platforms; considering maximum moments occurring at the mid span is given as:

$$M_a=\frac{wl^2}{8}; (s) = \frac{w_u l^2}{8} = (1.4G_k + 1.7Q_k)\frac{l^2}{8} \quad (15)$$

Applied moment (ACI 318)

$$S = (1.4\frac{G_k}{Q_k} + 1.7)\frac{Q_k l^2}{8} \quad (16)$$

Let $\frac{G_k}{Q_k} = \alpha = Load\ ratio$

$$S = 0.125Q_k l^2(1.4\alpha + 1.7) \quad [11] \quad (17)$$

Hence, the moment of resistance, M_r, for the section may be evaluated as:

M_r= $0.66\sigma_{bc}Z$

Where, z = section modulus

σ_{bc} = Flexural stress

The limit state function for flexure mode is given as:

$$G(x) = R - S \tag{18}$$

Where, R = M_r and S = M_a

Consequently, the expression for the performance function for compression failure G(X), in the limit state of bending would be:

$$G(X) = M_r - M_a \qquad i = 1, 2 \dots n. \tag{19}$$

This, on substitution of values, gives:

$$G(X) = 0.66\sigma_v Z - 0.125 Q_k l^2 (1.4\alpha + 1.7) \qquad [11] \tag{20}$$

Equation (15) provides the expectation that the resistance moment is greater than the applied moment. Expectation of a variable X, that is, *E(X)* is the weighted average or mean of all the values that the random variable may take. Consequently, it indicates the value or level of safety attained in the structural member.

For the beam to be structurally safe against shear failure due to web buckling,

$$P_v > F_v \tag{21}$$

Where, P_v = Shear capacity of member

F_v = Maximum applied vertical shear force

G(x) = $V_{res} - V_{app}$

Where, V_{res} = Ultimate shear resistance

V_{app} = Applied shear forces

$$G(X) = \phi_v 0.6 F_y A_v C_v - \frac{(1.4G_k + 1.7Q_k)L}{2} \qquad [11] \tag{22}$$

Or

$$G(X) = \phi_v 0.6 F_y A_v C_v - \frac{(1.35G_k + 1.5Q_k)L}{2} \qquad \text{(EC-3:2003)} \tag{23}$$

Or

$$G(X) = \phi_v 0.6 F_y A_v C_v - \frac{(1.2G_k + 1.6Q_k)L}{2} \qquad \text{(AISC-360: 2005)} \tag{24}$$

Where, ϕ_v= Shear resistance factor

F_y= Design strength of Steel

A_v= Shear area

C_v= Web shear coefficient

G_k= Characteristic Dead (permanent) load

Q_k= Characteristic Imposed (live) load

L= Beam span

For the beam to be structurally safe against deflection

G(x) = $\delta_{max} - \delta_a$

Where, δ_{max} = Max deflection allowed

δ_a = Deflection due to applied load

$$G(x) = \frac{l}{300} - \frac{5WL^4}{384EI} \qquad \text{(EC-3:2003)} \qquad (25)$$

For the beam to be structurally safe due to flange buckling

G(x) = $\frac{b}{T} \leq 40$

Where, b = Half width of section

T = Thickness of flange

For the beam to be structurally safe due to web buckling

G(x) = $\frac{d}{t} \leq 40$

Where, d = Depth between fillets

t = Thickness of web

The typical beam is checked for all the above conditions and must be found adequate.

4. Results and discussion

4.1. Results

First order reliability analysis result (FERUM IN MATLAB)

This section discusses the findings of the reliability analyses conducted on the rolled steel beam in MATLAB to examine and simulate the behavior of the steel beam on exposure to corrosion during a design period of 100 years. Through an interval of 10 years, the reduction in cross-sectional area was examined in order to check the effect of the rolled steel beam in service resisting the effect of moment, deflection and shear. FORM coded in MATLAB [7] is employed in the computation, making use of the tabulated data in Table 2 and the relevant limit state functions.

Table 2
Parameters of the Stochastic Model for Rolled Steel Beams on Offshore Platforms.

S/No	Design Variables	Unit	Distribution Type	COV.	E($\boldsymbol{xi}$)	S($\boldsymbol{xi}$)
1	Steel strength (f_{yk})	N/mm^2	Lognormal	0.15	350	52.5
2	Span (L)	mm	Normal	0.05	10000	500
3	Breadth (b)	mm	Normal	0.03	418.5	12.555
4	Depth (d)	mm	Normal	0.03	911.8	27.354
5	Area of Section (A)	mm^2	Normal	0.03	43231.32	1296.9396
6	Imposed Load (Q_k)	kN/m^2	Lognormal	0.3	$5.0\text{x}10^{-3}$	$1.5\text{x}10^{-3}$
7	Wind Load (W_k)	kN/m^2	Lognormal	0.3	$5.0\text{x}10^{-3}$	$1.5\text{x}10^{-3}$

The reliability levels were calculated using the deterministic and statistical parameters of Table 2 based on FORM in MATLAB program, Table 3 gives the summary of the result of the initial year obtained in the program for moment, shear and deflection failures of the beam.

Table 3
Results obtained by FERUM in MATLAB (R2015b) for the beam at initial stage.

Failure mode	Time to complete Analyses	Reliability Index $\boldsymbol{\beta}$	Probability of Failure $\boldsymbol{P_f}$
Moment	0.888363	2.2625E+01	0.00314E-04
Deflection	1.588546	3.5013E+01	0
Shear	0.878415	2.9046E+01	0

In order to find out how the reliability indices and probability of failures for the beam will be, the analysis was also evaluated for various reduced cross-sectional areas as stated above using the corrosion rate per year. The safety values tend to be decreasing for moment, deflection, and shear limit state equations. The probability of failure due to shear and deflection is 0, while that of moment tends to 0.00314E-04.

Table 4
Reliability indices and probability of failures for the corroded beam section.

Failure mode	Moment		Deflection		Shear	
Year	Reliability Index β	Probability of failure P_f	β	P_f	β	P_f
10	2.2559E+01	0.00271E-04	3.5013E+01	0	2.8948E+01	0
50	2.2282E+01	0.00210E-04	3.5014E+01	0	2.8538E+01	0
100	2.1901E+01	0.00157E-04	3.5016E+01	0	2.7979E+01	0

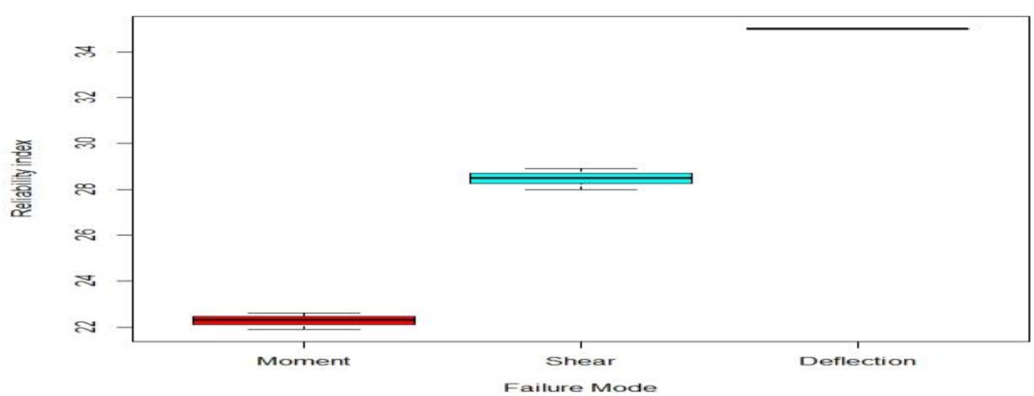

Fig. 4. A boxplot showing the relationship between reliability indices to the failure modes at various years during its service life.

Based on the result output in MATLAB, it was observed that P_f increases with reduction in cross-sectional area due to corrosion and thickness for the moment failure mode, whereas for deflection and shear failure it recorded minimal reduction in values of reliability index, β, with probability of failure, $P_f = 0$. It should be noted that the higher values of safety index obtained for moment, deflection and shear failures implies that the beam is too safe against these failure modes. The consequence of this is a conservative design with a very high cost that is uneconomical. Normally, a lower value of the reliability index, β, implies unsafe structure as observed for the moment of resistance which gave values of 2.2625E+01and probability of failure P_f of 0.00314E-04. Based on these findings, the cross-sectional area has a significant effect on the values of reliability indices and the probability of failure.

Usually, the variability of the statistical parameters significantly affects the magnitude of failure associated with the beam, therefore, the uncertainty in these parameters should be considered for the rolled steel beams design.

5. Conclusion and recommendation

5.1. Conclusion

This study focuses mainly on the reliability analysis of rolled steel beams on offshore platforms. The analysis also shows that the intrinsic structural safety of the shear capacity, resistance moment, and deflection of the beam decreases as the cross sectional area reduces due to corrosion. It was observed that increase in the cross sectional area, grade, and exposure condition affects the structural safety or reliability of the rolled steel beams. The result of probabilistic appraisal of the rolled steel beam using generated limit state functions and First Order Reliability Method under uncertain loading based on the provision of Eurocode-3 has been presented. The reliability indices for moment, shear, and deflection of the beam, indicates that the reliability obtained is slightly higher than recommended by Joint committee on Structural Safety (JCSS) risk assessment in engineering.

5.2. Recommendation

A lower section can be adopted for the design of rolled steel beam for an offshore structure, the grade of steel used and the cross-sectional area should be readily selected through a reliability based design in order to design a safe structure. However, Finite Element Analysis as coded in ABAQUS, MATLAB, and numerous computer based analytical packages should be used to enhance studies on offshore structural design including concrete elements. Also, a reliability-based design methodology should be established for offshore structures before construction is carried out.

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