Probabilistic Shear Strength of Bolted Joints in Offshore

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Abstract. This study evaluates the formulations for prediction of ultimate shear capacity of steel joints in offshore structures in order to ascertain the safety of the structures in service using First Order Reliability Method (FORM) under the European code, EC3:1,8 (2003), design format. Two modes of failure were considered which depend on the embedding strength of the plates and bolts that influences the joints capacities and the failure due to bearing. It is generally observed that the increase in bolt diameter, bolt strength with corresponding steel grades, fasteners thickness improves the safety of the joints, while undue increases in load on the offshore structure may result in safety reduction, but not collapse. The provision for joints design in structures in EC3 (2003) is very robust and efficient. However, economy commensurate with structural safety can be achieved in the current formulations.

Keywords: Offshore · Structural safety · Shear strength · Steel bolts Plate thickness

1 Introduction

Offshore probabilistic design is preceded by a sequence of activities that result in the selection of a field development system that best fits the field characteristics and economics. Before feasible alternatives for producing oil and gas from an offshore field are identified and the most desirable production scheme is selected, exploratory work defining the reservoir characteristics have to be completed. First, a decision has to be made whether an offshore location has the potential for hydrocarbon reserves. This assessment is achieved through a study of geological formations. The basic strength of members in ships and offshore structures include support members (for example, stiffness and plate girder), plates and stiffened panels. During their lifetime, the structures that are constructed using these members are likely subjected to various types of loadings or deformations that are for the most part operational, but may in some cases, be extreme or even accidental (Moan 2005). The sources of such loadings and deformations include fabrication-related initial imperfections (for example, initial distortions, welding residual stresses, softening in the affected zone of welded aluminum structures); abnormal waves, winds and currents; dynamic pressure loads arising from sloshing, slamming or green water, low temperature in Arctic operation; cryogenic conditions resulting from liquefied natural gas cargo, ultra-high pressures in ultra-deep water, elevated temperature due to fire, blasts load due to explosion, impact

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loads associated with collision, grounding or dropped objects, and age related degradation such as corrosion, fatigue cracking and local denting damage (Paik 2017). These phenomena may occur in offshore structures, while they are in service. It is interesting to mention that all of them commonly give rise to non-linear structural consequences that involve both geometric and material non-linearity. In the past, criteria and procedure for the structural design of offshore platforms were primarily based on allowable stresses and simplified buckling checks for structural components. However, the ability to correctly assess the true margin of safety would also inevitably lead to improvements in related regulations and design requirements (Paik 2017).

Circular hollow sections combine excellent structural properties with an architecturally attractive shape. This has resulted in many applications in buildings, halls, bridges, barriers, masts, towers, offshore and special applications, such as glass houses, radio telescopes, sign gantries, parapets, cranes, jibs, sculptures, etc. (Wardenier et al. 2002). Offshore oil production is one of the most visible of these applications and represents a significant challenge to the design engineer.

Nowadays, the existing offshore steel structures are no longer safe with regards to overload from excessive wind, wave and current, in particular that the load pattern assumed in design has been significantly changed (Ersdal and Hörnlund 2008). The safety of structures for an expected design life generally designed according to established Standard or Codes and method has become deration, particularly due to fatigue and other continuous degradation mechanisms of structural materials, joint connections and foundation systems such as soil subsidence to unknown extent during decades in harsh weather (Ersdal and Hörnlund 2008). Tools to assess the structural safety of welded columns in an offshore structure especially intended to be used beyond its original design life, has become a major task. Two assessment approaches for this offshore structural strength were suggested (Ersdal and Hörnlund 2008), namely non-linear static or quasi-static pushover method and risk-based method.

It is for this reason that the structural reliability-based method is employed herein.

This presentation evaluates the probabilistic shear strength of bolted columns in offshore and as formulated in design codes.

The objectives of this study include: (i) providing a probabilistic assessment of the offshore steel shear capacity as formulated in codes; (ii) providing an acceptable level of safety by defining minimum requirements for offshore columns structures and structural components and (iii) obtain optimum design criteria that will be widely accepted as per ISSC (2012), API (2000), ISO 19902 (2004) and EC3 (2003) recommendations.

There are various limitations that affect the assessment of steel joints in offshore structures. Only the shear strength capacity of bolted columns is considered herein, using a risk-based method as formulated in the First Order Reliability Method (FORM).

Some of these limitations are stated as follows: (i) fatigue assessment for structural members and columns are not considered; (ii) earthquake and dynamic loads are excluded; (iii) environmental loads including wave inundated load on deck are treated as static loads and (iv) secondary structures are modeled to generate weight and environmental loads.

2 Historical Development of Offshore Structures

Offshore steel structures are often called jackets, which originates from the early days of the offshore industry when a trussed structure jacket was placed over the piles to provide lateral stiffness to enable it withstand wave current and wind forces. Offshore jacket structures are installed in deep water depths. These structures are often exposed to corrosive action of salt water as well as extremely low temperatures (Armand *et al.* 2009). They are usually built on concrete or steel legs, or both, anchored directly on to the seabed, supporting a deck with space for drilling rigs, production facilities and crew quarters. Such structures by their virtue of immobility are designed for very long time use. Various types of structures are used, steel jackets, concrete caissons, floating steel and even floating concrete. Steel jackets are vertical sections made of tubular steel members, and are usually piled into the seabed. Concrete caisson structures, pioneered by the Condeep concept, often have in-built oil storage in tanks below the sea surface and these tanks were often used as a floating capability, allowing them to be built close to shore and then floated to their final position where they are sunk to the sea bed. Fixed platforms are economically feasible for installation in water depths up to about 520 m (Banerjee et al. 2007). Offshore exploration and production of minerals is advancing into deeper waters at a fast pace. Many deep water structures have already been installed worldwide. New oil and gas fields are being discovered in ultra-deep water. Many of these fields are small and their economic development is a challenge today to the offshore engineers. This has initiated the development of new structures and concepts. Many of these structures are unique in many respects and their efficient and economic design and installation are a challenge to the offshore community. There are two important features that need to be taken into consideration for a proper assessment to be done and these are durability and ultimate strength of steel.

The corrosion of steel can be considered as an electrochemical process that occurs in stages and requires the simultaneous presence of water and oxygen. In the absence of either, corrosion does not occur (Baddoo 2008). Marine environments are recognized to be very corrosive for mild and alloy steels. For reasons of economy, such steels are still the preferred materials for many engineering structures, such as, offshore structures. As the age of a structure increases, the loss of the structure's thickness caused by corrosion induces a decrease in the ultimate strength of the structure and potentially a loss of integrity through pit, whose loss is particularly critical when protective coating systems, such as galvanizing or painting coatings, are ineffective. Conventionally, a deterministic approach is used to assess the remaining life of the offshore structures, but it is now recognized that the probabilistic method is a better approach because it is more robust, reliable and justifies safety with associated economy. Several probabilistic models have been proposed with the majority of them focused on predicting the amount of pitting corrosion for a structure with different ages. Jelovica et al. (2014) evaluated the amount of reduction in the buckling strength of sandwich plates as a result of general corrosion. The main findings were that a 0.5-mm decrease in plate thickness will cause 25.5% decrease in their buckling strength, which is quite severe and worrisome. However, corrosion will definitely be an important hazard for the structure in cases where the corrosion protection is not sufficient for the extended life.

Hence, the corrosion effects on ageing structures in a life extension would need a specific investigation (Hørnlund *et al.* 2005).

The techniques in Finite Element Method (FEM) have matured for ultimate strength evaluation of plated structural components. Many researchers have applied Finite Element Method (FEM) to predict ultimate strength of unstiffened plates and stiffened plates, such as Zhao and Zhang (2007) and Paik (2008). In these applications, both geometric and material nonlinearities were considered. It may be said that it is fairly straightforward to use Finite Element Method (FEM) for ultimate strength prediction of plates and stiffened plates. The last decade sees many developments and innovations of tubular connections in the offshore industry. Such applications include the more widespread adoption of thick-walled sections in offshore and onshore structures, internally or externally reinforced tubular connections, *etc*. Recent research effort also focuses more on the failure assessment of tubular connections with initial defects, since fatigue induced cracks remain as a potential threat for offshore steel platforms in the event of extreme environmental loading. These practical concerns in the industry do not find corresponding theoretical background in the literature or design codes (API 2000; ISO 19902 2004). Zhao 2005 has noted that the chord stress effect for Circular Hollow Section (CHS) and Rectangular Hollow Section (RHS) joints still remain as an issue to be solved. Therefore, there is need for a more detailed understanding on the ultimate strength of tubular connections with due emphasis on larger wall thickness, presence of initial defects, provision of reinforcement, and the effect of chord stresses, for a safe and economical design.

In offshore structures, the service life is usually assumed to be equal to the initiation time, when corrosion is caused by chloride ingress. In terms of the consequences of localized corrosion, the period of propagation is traditionally not taken into account (Hobbacher 2009). Crude Monte Carlo simulation offers a direct method for estimating the failure probability. In essence, the technique involves sampling a set of values of the basic variables at random from the probability density function, and evaluating the failure function for the values to see if failure occurs. The drawback with crude Monte Carlo simulation is the computational effort involved. In order to produce a reasonably accurate estimate of the failure probability large data from at least $100/P_f$ trials may be required. This is quite cumbersome and thus, knowledge of the failure region (for example from first-order methods) can be exploited to significantly improve the efficiency of Monte Carlo simulation by tailoring the sampling scheme to the particular situation. However, if used intelligently, Monte Carlo methods are a readily understood and easily applied tool. They can be used to generate accurate answers to problems, and to such other ones that cannot be accurately modeled using first- or second-order methods. Because corrosion is a function of many variables, and of uncertain nature, a probabilistic model is more appropriate to describe the expected corrosion (Melchers 1999).

Many developments and innovations of tubular connections in the offshore industry were witnessed in recent times. Such applications include the more widespread adoption of thick-walled sections in offshore and onshore structures, internally or externally reinforced tubular connections, *etc*. Recent research effort also focuses more on the failure assessment of tubular connections with initial defects, since fatigue induced cracks remain as a potential threat for offshore steel platforms in the event of extreme environmental loading. These practical concerns in the industry do not find corresponding theoretical background in the literature or design codes (API 2000; ISO 19902 2004). Zhao (2005) has noted that the chord stress effect for Circular Hollow Section (CHS) and Rectangular Hollow Section (RHS) joints still remains as an issue to be solved for the upcoming version of the International Institute of Welding (IIW) design guidelines.

The European standard (prEN 1993-1-8: 2003), has suggested application rules to determine the static resistances of uni-planar and multi-planar joints in lattice structures composed of circular, square or rectangular hollow sections, and of uni-planar joints in lattice structures composed of combinations of hollow sections with open sections. In particular, the EC3 (2003) provision for rotation capacity in these sections: (i) do not establish any specific procedures for the evaluation of the rotation capacity; and (ii) optimum capacity of bolted or welded joints. However, they state the need to ensure adequate rotation capacity either by testing in accordance with EN 1990 (2004).

Alternatively, using appropriate calculation models based on the result of tests, for end-plate beam-to-column steel joints, these conditions specified in the code basically suggests that either the columns web panels in shear controls the behavior of the joint or, alternatively, the end plate or the column flanges in bending, which is reproduced as Eq. (1) (Da Silva 2008):

$$\frac{d}{tw} < 69\varepsilon \tag{1}$$

$$\text{Where } \varepsilon = \sqrt{\frac{235}{f_y}}.$$

$$\mathrm{t} < 0.36\mathrm{d}$$

2.1 Shear Connections in Single Plates

The static resistances of the joints are expressed in terms of maximum design axial and/or moment resistances for the brace members. The application rules are valid both for hot finished hollow sections to EN 10210 (1993) and for cold formed hollow sections to EN 10219 (1993), if the dimensions of the structural hollow sections fulfill the necessary requirements. The nominal wall thickness of hollow sections should be limited to a minimum of 2.5 mm and should not be greater than 25 mm unless special measures have been taken to ensure that the through thickness properties of the material adequate. These types of joints are covered by standard prEN 1993-1-8 (2003). The application rules given in prEN 1993-1-8 (2003) may be used only where all the given conditions are satisfied.

2.1.1 Bolted Connections

Even if bolted connections to hollow sections are used to assemble prefabricated elements or space structures, the most used method to assemble CHS members is welding, especially for trusses. According to prEN 1993-1-8 (2003), the design joint resistances of connections between hollow sections and of connections of hollow sections to open sections, should be based on standard failure modes as applicable and specified in the code.

Connecting two hollow section members or a hollow section and an open profile or a plate directly to each other by bolting can be difficult unless the joint is located close to the open end of a hollow section member. Otherwise, it is necessary to take measures, such as cutting a hand access hole in the structural hollow section member to enable the bolt to be tightened from the inside or using “through” or “blind” bolts. The reason for this special situation is evident, as the hollow section allows free access only to the outside; any access to the inside is restricted. Bolted connections remain nonetheless desirable in many cases in spite of the unique condition of non-accessibility to the inside of a hollow section. However, in these cases, the hollow sections can be joined indirectly using flange or capping plates, which makes it possible to effect such bolted connections in a simple and economical manner. The main methods of assembly by bolting are described below. Bolted connections are mostly detachable. They are selected for the on site assembly in order to avoid site welding, which may cause welding errors due to environmental difficulties. Site welding is also more costly than site bolting. This is presented in accordance with “Design guide for structural hollow sections in mechanical applications” of Wardenier et al. (2002) and “Guide on the use of bolts: single sided blind bolting systems of Yeomans et al. (2002). The main types of bolted connections for hollow section structures are: Bolted knee joints, Flange connections, Splice joints, Joints with fork ends, Screwed tensioner, Through bolting, Bolted connections with flattened ends, Hinged support, Column bases, Fish plate connections, Bolted subassemblies, and Fixing bolts through hand access holes (CIDECT 1995). These connections are realized using intermediate connecting steel devices, which are welded on the hollow section members, the bolted connections themselves being designed as normal connections according to prEN 1993-1-8 (2003). For this reason, design of hollow section connections does not imply specific requirements.

2.2 Design Formulations

The design formulations of EC3 (2003) as earlier recommended in prEN 1993-1-8 (2003) are as follows:

Shear resistance for all fasteners

$$\mathrm{F}_{\mathrm{v,Rd}} = \frac{n\alpha v f_{ub} A}{\gamma M_2} \tag{2a}$$

Bearing resistance

$$\mathrm{F}_{\mathrm{b,Rd}} = \frac{nk1\alpha b f_{ub} A}{\gamma M_2} \tag{2b}$$

Where γM_2 is partial safety factor for joints given as 1.25 for all fasteners and γM_2 is partial safety factor for all hybrid connections given as 1.25, where the shear plane passes through the threaded portion of the bolt (A is the tensile area of the bolt A_s):

α_v = 0.6 for Classes 4.6, 5.6 and 8.8 bolts
α_v = 0.5 for Classes 4.8, 5.8, 6.8 and 10.9 bolts

Where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt) $\alpha_v = 0.6$

$\alpha_v = 1.0$, $K_1 = 2.5$

f_{ub} is the ultimate tensile strength for bolts.

The yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolt classes 4.6, 5.6, 6.8, 8.8 and 10.9 are given in Table 1. These values should be adopted as characteristics values in design calculations. Also Table 2 indicates the categories of bolts used in connections in offshore structures.

Table 1. Nominal values of the yield strength, f_{yb}, and ultimate tensile strength, f_{ub}, for bolts (Eurocode3 1-8, 2003)

Bolt classes	4.6	5.6	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	300	480	640	900
f_{ub} (N/mm^2)	400	500	600	800	1000

Table 2. Categories of shear connections in bolts

Category	Criteria	Remarks
Bearing type: A	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required for bolt classes from 4.6 to 10.9 may be used
Slip resistant at serviceability: B	$F_{v,Edser} \leq F_{v,Rdser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability
Slip-resistant at ultimate: C	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate

These formulations for the design and structural safety of bolts in steel structures will be evaluated in a probabilistic setting using appropriate probabilistic methods.

3 Methodology

3.1 Probabilistic Reliability Assessment Using Form

Considering optimization of the structural element here only and as usual as regards other cases, it can be said that, a structural element will fail if its resistance R is less than the applied load S acting on it (Melchers 1999). The probability of failure (P_f) of the structural element can be stated in any of the following ways:

$$P_f = P(R - S) \tag{3a}$$

R = Resistance,
S = Applied load on structure.

$$P_f = P(R - S) \leq 0) \tag{3b}$$

$$= P(R|S) \leq 1) \tag{4}$$

$$= P(\ln R - \ln S) \leq 1) \tag{5}$$

It can also be expressed in general as:

$$P_f = P\ (G(R, S))\ \leq 0 \tag{6}$$

Where G(X) is the limit state function and the probability of failure (P_f) is identical with the limit state violation. For any random variable, X, the cumulative distribution function $F_x(X)$ is given by:

$$F_x(X) = \int_{-\infty}^{x} F_x(y)dy \tag{7}$$

Provided that X $\geq$ Y.

It follows that for the common, but special case when R and S are independent, the expression for the probability of failure can be given as:

$$P_f = P(R - S) \leq 0) = \int_{-\infty}^{+\infty} F_R(rds) \tag{8}$$

It could also be written as:

$$P_f = P(R - S) \leq 0) = \int_{-\infty}^{+\infty} F_R(X)drds \tag{9}$$

This integral is known as convolution integral and $F_R(X)$ is the probability that R $\leq$ x, or is the probability of the actual resistance, R, of the member less than some values of x.

The term $F_S(x)$ represent, the probability that the load effect, S, acting on the member has a value between x and $x + \Delta x \rightarrow 0$.

Considering the possible value of x, the total failure probability is obtained by:

$$P_f = \int_{-\infty}^{+\infty} F_S(x)(F_R(X))dx \tag{10}$$

This is the sum of all cases of resistance in which the applied load exceeds the resistance. The convolution integral may be integrated for a few distributions of R and S. This is easy when R and S are normal variables with mean μ_R, μ_s and variance $\sigma R_2, \sigma S_2$, respectively.

The safety margin will be:

$$Z = R - S \tag{11}$$

Therefore, the mean and variance will be given by:

$$\mu_z = E(z) = \mu_R - \mu_S \tag{12a}$$

$$\sigma_Z^2 = \text{var}(\mu) = \sigma_R^2 + \sigma_S^2 \tag{12b}$$

$$\sigma_z = \sqrt{\left(\sigma_R^2 + \sigma_S^2\right)}$$

Using well known rule for addition/subtraction of normal random variables Eq. (7) becomes:

$$P_f = (R - S) \leq 0) = P(z \leq 0) = \text{ø}\left[\frac{\mu z}{\sigma z}\right] \tag{13a}$$

Where ø is the standard normal distribution function, the reliability index, *β,* is defined as the ratio μ_z/σ_z.

$$P_f = \text{ø}\left(\mu_z/\sigma_z\right) = \text{ø}(-\beta) \tag{13b}$$

Hence, P_f increases with decreases in resistance and reliability indices are one of the commonly used probabilistic measures of safety. First Order Reliability Method (FORM) is one of most common basic techniques and is applicable to all probabilistic problems. It is usually preferred because of its independence on the simulation number (Webster 2001). In order to overcome the invariance problem with the failure function, it was found that it is necessary to transform the basic variables into independent standard normal variables. The First-Order Reliability Method is used by independent standard normal variables of whether transformation of independent variables can be undertaken from the cumulative probability of the distribution that is from the identity. The transformation is generally undertaken automatically within most reliability analysis software packages. However, for non-standard distribution functions, the transformation may need to be undertaken explicitly. First-order second-moment methods, involve estimating the failure probability by *linearizing* the failure surface at the closest point to the origin in standard normal space. It is usually necessary to iterate in order to determine the closest point to the origin, and a number of iterative and optimization techniques are available. The basic variable transformation and first-order reliability estimates are in two basic variables. The distance from the origin to the beta-point is equal to the first-order reliability index. This is sometimes referred to as the geometrical reliability index.

3.2 Limit State Functions

A limit state equation is a state beyond which structures no longer possess at least one of their characteristics that is, the failure at the state region.

3.2.1 Analysis of Structures Based on EC3 (2004)

The limit state equation is as follows

$$\mathrm{G(x)} = \mathrm{R} - \mathrm{S} \tag{14}$$

R = resistance shear force and S = applied shear force.

Hence,

$$\mathrm{G(x)} = F_{V,RD} - F_{V,ED} \tag{15}$$

Failure of the Joint due to Bolt Shearing

For the connection to be safe against shear failure the equation below must be satisfied.

$$F_{V,ED} \leq F_{V,RD} \tag{16}$$

$$F_{V,RD} = \frac{n \propto v f_{ub} \pi d^2}{4\gamma M_2}, \quad n = 4 \tag{17}$$

and

$$F_{V,ED} = \frac{Vl}{2} \tag{18}$$

The maximum design shear load, V, prescribed in relevant codes (API 2000) for offshore structures is given as:

$$\mathrm{V} = (1.35\mathrm{G_k} + 1.7\mathrm{Q_k}) \tag{19}$$

by substituting for V in the above Eq. (18) we obtain

$$\mathrm{V} = \frac{(1.35G_k + 1.7Q_k)l}{2} \tag{20}$$

Let $\alpha = \frac{G_k}{Q_k}$ be the load ratio.

The applied shear force is then

$$F_{V,ED} = \frac{Q_k(1.35 \propto + 1.5)l}{2} \tag{21}$$

Hence, failure due to bolt shear is given by

$$G(x) = 1.26 f_{us} d^2 - Q_k(0.675\alpha + 0.75)l \tag{22}$$

Failure due to bolt bearing

$$F_{b,RD} = \frac{n_{k1} \propto_b f_{ub} td}{4\gamma M2} \tag{23}$$

$$G(X) = f_{ub} td - Q_k(0.675 \propto + 0.75)l \tag{24}$$

The results obtained and discussions there from the first order reliability analysis as coded in FORM 5 are given in Sect. 4.

4 Results and discussion

4.1 Results

The probabilistic model generated in Table 3 were analyzed using the First Order Reliability Method (FORM) to give values of structural safety indices (β) and probability of failure (P_f) in shear and bearing capacity of bolted steel joints in offshore structures. An algorithm developed in FORTRAN module was used for the above failure mode with various ultimate strengths of steel, load ratios, nominal areas of bolts, diameter of bolts and thickness of fasteners.

Table 3. Probabilistic parameters of the basic variables under shear mode of failure, bearing mode of failure

S/no	Basic variables	Basic variables Ex	Probability distribution function	Coefficient of variation	Standard deviation Sx
1	Ultimate shear strength of bolt f_{us} (N/mm^2)	600	Log-normal	0.03	18.00
2	Ultimate bearing strength of bolt f_{ub} (N/mm^2)	600	Log-normal	0.03	18.00
3	Imposed load Q_k (KN/m^2)	10.00	Log-normal	0.03	0.30
4	Diameter of the bolt (mm)	25.00	Normal	0.05	1.25
5	Length (mm)	1500	Normal	0.05	75.00
6	Thickness of fasteners t (mm)	40.00	Normal	0.03	1.20
7	N	4.00	Normal	0.05	0.20

4.2 Discussion

The failure due to bolt shearing as illustrated from Figs. 1, 2, 3, 4, 5 and 6; shows that the structural safety indices increases with increases in bolt diameters and bolt strength, while the safety indices decreases with increases in load ratios. It is observed also that for the bolt diameters 20 mm, 22 mm and 24 mm there is significant increases in structural safety levels (that is, the probability that the structure will fail in shear is minimal or greater resistance to applied loads).

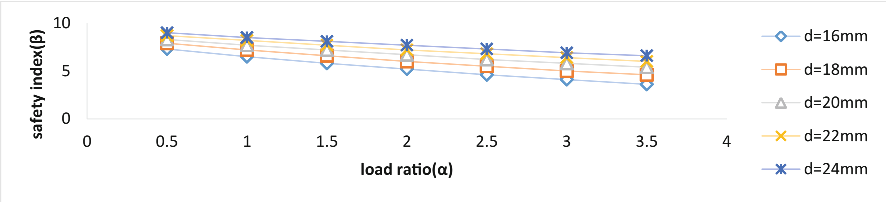

Fig. 1. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 6.8, L = 2000 mm, f_{us} = 600 N/mm^2)

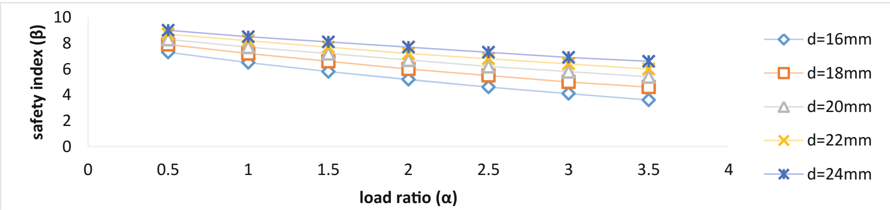

Fig. 2. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 5.6, L = 1500 mm, f_{us} = 500 N/mm^2)

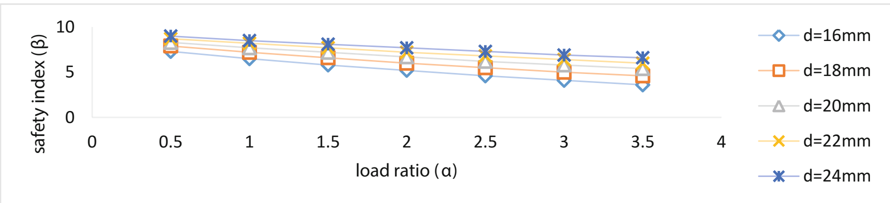

Fig. 3. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 4.6, L = 1000 mm, f_{us} = 400 N/mm^2)

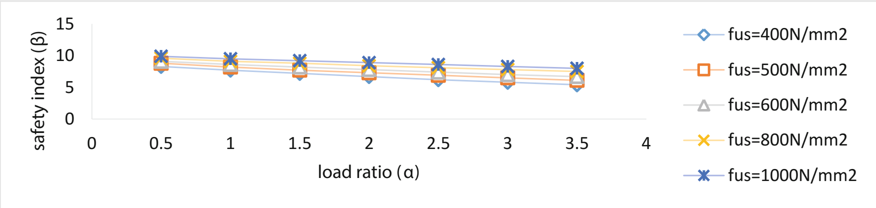

Fig. 4. Relationship of safety index (β) to load ratio (α) and 18 mm bolt diameter, L = 2000 mm and varying f_{us} in shear.

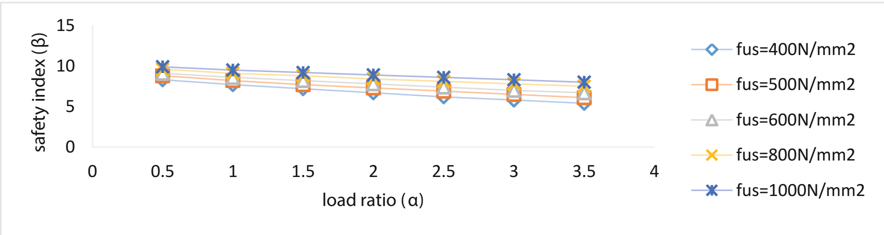

Fig. 5. Relationship of safety index (β) to load ratio (α) and 20 mm bolt diameter, L = 1500 mm and varying f_{us} in shear.

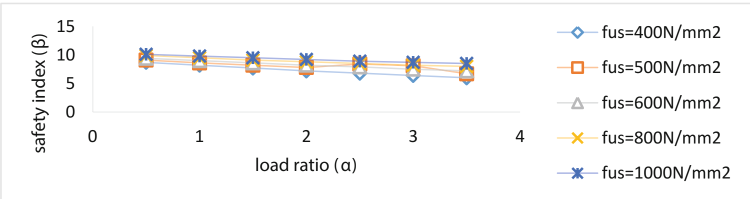

Fig. 6. Relationship of safety index (β) to load ratio (α) and 22 mm bolt diameter, L = 1000 mm and varying f_{us} in shear.

For the failure due to bearing of bolt as illustrated in Fig. 7, 8, 9, 10, 11 and 12, it is observed that the structural safety indices increases with increases in bolt diameters, bearing strengths and thickness of plates and members in joints, while the safety indices decreases with increases in load ratios. It is noted too that for the bolt diameters 20 mm, 22 mm and 24 mm, there are significant increases in structural safety levels; but at 10 mm or lower plate thickness, the safety of the structure is not guaranteed as in the case of other plate thicknesses of 20 mm, 30 mm, 40 mm and 50 mm. This suggests that bolted steel joints of members in offshore should be sufficiently thick; perhaps 16 mm thick plates should be used as minimum.

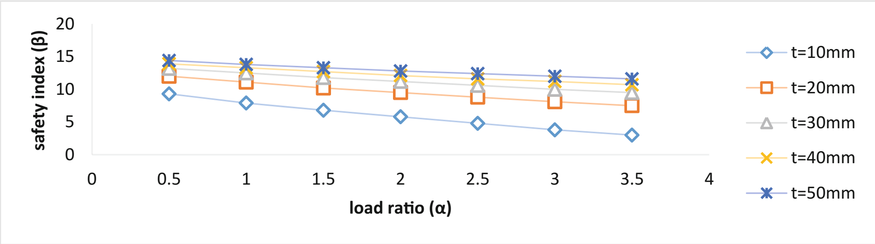

Fig. 7. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 6.8, L = 2000 mm, f_{ub} = 600 N/mm^2)

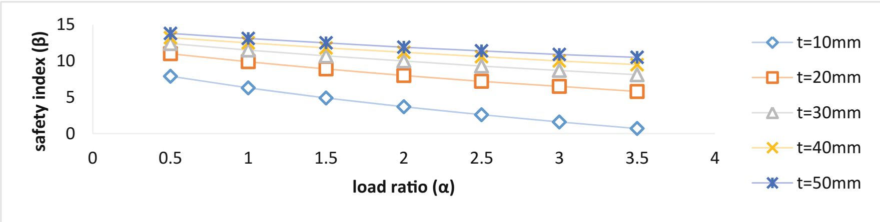

Fig. 8. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 5.6, L = 1500 mm, f_{ub} = 500 N/mm^2)

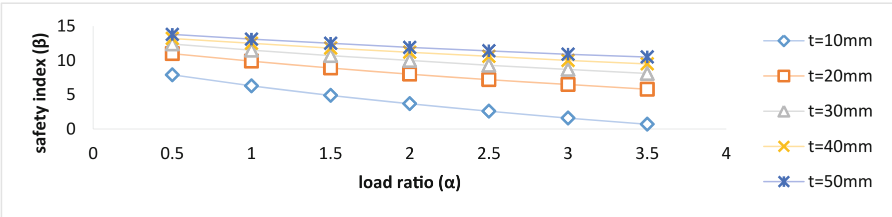

Fig. 9. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 4.6, L = 1000 mm, f_{ub} = 400 N/mm^2)

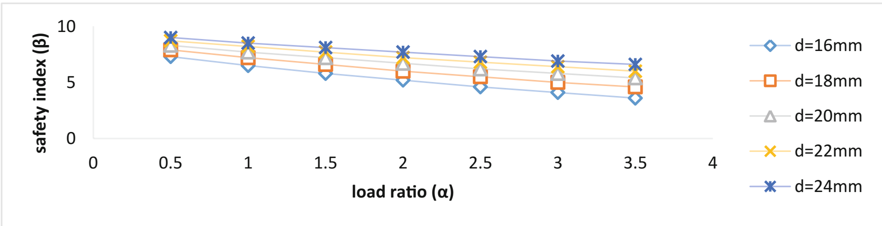

Fig. 10. Relationship of safety index (β) to load ratio (α) and bolt diameter (bolt grade 4.6, L = 1000 mm, f_{ub} = 400 N/mm^2)

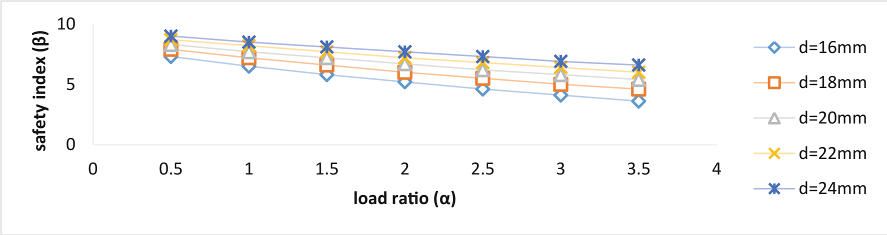

Fig. 11. Relationship of safety index (β) to load ratio (α) and 30 mm bolt thickness L = 1500 mm, f_{ub} = 500 N/mm^2 and varying diameter in bearing

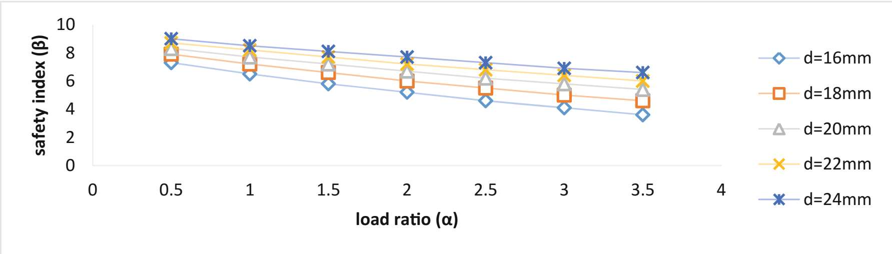

Fig. 12. Relationship of safety index (β) to load ratio (α) and 20 mm bolt thickness L = 1000 mm, f_{ub} = 400 N/mm^2 and varying diameter in bearing

5 Conclusion and Recommendation

This study presents the results of investigation of the structural safety levels of steel joints for two failure modes for bolted joints using probabilistic assessments. The level of failure which is associated with each of the failure modes were computed using the First Order Reliability Method (FORM). It was observed that the safety indices decreases with increases in load ratios which is an indication that the structural safety of steel joints is dependent on the bolt diameter, bolt strength, thickness of fasteners and the joint load. However, the formulations as evaluated is robustly specified or configured as the safety indices are well above the minimum for safety and economy as recommended by the JCSS (2005). Thus, economy commensurate with safety may still be achieved when prompt and adequate maintenance of the existing structure is carried out and for upgrade for increased structural capacity.

In order to obtain an optimum design life, as the recommendations of EC3 (2003) and JCSS (2005). It is recommended herein that the use of high-strength bolt of grade 6.8 and 24 mm diameter; and with a fastener thickness of 40 mm which not only have high resistance but also for its safety and economy is essentially structurally safe for joints in offshore steel structures.

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