Nigerian Journal of Technology (NIJOTECH)
Vol. 31, No. 3, November, 2012, pp. 336–345.

ISSN 1115-8443

SAFETY OF POST-FIRE STRENGTH CAPACITY PREDICTION IN BOLTED STEEL JOINTS

A.A. Murana[a], O.S. Abejide[b]

Civil Engineering Department, Faculty of Engineering, Ahmadu Bello University, Zaria, Nigeria.
Emails: *[a] fatinoyi2007@yahoo.co.uk; [b] abejideos@yahoo.com*

Abstract

Analytical study of the reliability of a typical bolted double lap joint is presented. Evaluation of the experimental formulation of the strength is carried out using First Order Reliability Method. Results obtained showed that there is a significant capacity reduction of the joint as the residual strength index, RSI, of the connection decreases. Five failure modes which depend on the embedding strength of the steel plates and/or bolts were used to obtain the joint capacities; while the failure due to bearing of plates at the joint was found to be the least reliable. It was observed that increase in member thicknesses, bolt diameters, number of bolts and end/edge distances enhance the safety levels but may not be economical. However, reduction or decrease in any of the above mentioned parameters proved otherwise. More so, safety indices as obtained indicate that steel bolted joints can be rehabilitated and re-used after fire with ease.

Keywords: steel bolts, safety, lap joints after fire, reliability, residual strength index

1. Introduction

Joints are critical parts of steel structures, transmitting forces between structural members. The ultimate behaviour of a structure depends strongly on the structural configuration of the joints [1]. For quite a few decades, riveting was the accepted method used for connecting the members of steel structures. For the last three decades, however, bolting and welding have been the methods used for making structural steel connections, and riveting is almost extinct [1]. Thus, the probabilistic examination of the safety of post-fire strength capacity prediction in bolted steel joints is good benefit to the rehabilitation of existing structures. Bolting of a steel structure is a very rapid field erection process that requires less skilled labour than does riveting and welding. Also, bolted connections are cheaper and have reduced labour and equipment cost; a smaller number of bolts are required to resist the same loads as compared to riveted and welded connections [2].

In order to investigate the fire-resistance capacity of bolted steel joints, an experimental study was performed [3] using a furnace on four full-scale specimens made with H-shaped steel. The failure characteristics and failure modes of the bolted steel joints specimens in fire were obtained from the experiment. The component of a joint are considered to consist of spring components with predefined mechanical properties which are stiffness and strength. The response of the joints subjected to elevated temperatures can be predicted by assembling components, the stiffness and strength of which are assumed to degrade with increasing temperatures [4]. Adequate investigation of the effect of degradation due to fire is structurally important in order to estimate the post-fire use of the steel structure [4].

The essence herein is to establish a level of consistency using the concept of reliability to evaluate the ultimate limit state criteria for a bolted steel joint after exposure to fire. This presentation contributes to the economic design of steel joints in the sense that after fire, the strength of a joint is determined and if adequate, renovation works can be done on the remnants structure instead of demolishing the entire structure. However, the focus is on the safety of the residual strength of bolted double lap joints designed according to British Standard Institute [5] and limited to the use of The First Order Reliability Method (FORM) as suggested by [6].

2. Joints in Bolts

2.1. Structural bolts

The behaviour of bolted steel joints subjected to fire depends on the temperature range and duration of the fire. Thermocouples have been used to monitor the time-temperature rise response in the experiments on bolted steel joints. The validity of the calculation procedure was established by comparing the cal-

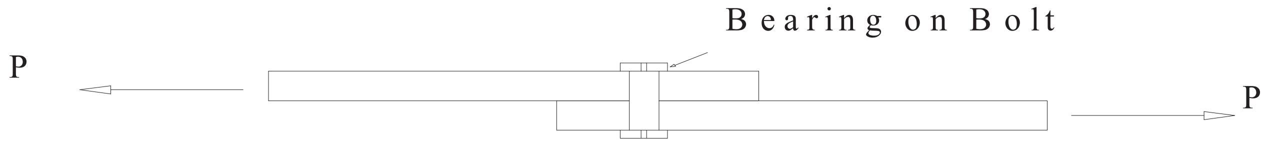

Figure 1: Bolt in single shear in a lap joint.

culated fire resistances with those obtained from fire test on bolted steel joints as expressed by Kodur [7]. He continued that steel bolts subjected to heat treatments that stimulate different potential scenarios in a building exposed to fire have revealed the possibility of significant brittleness. Bolts rapidly quenched from elevated temperatures (900℃) exhibit a brittle fracture under relevant impact testing, whereas bolts allowed to cool slowly from the same temperature exhibit a ductile fracture [8]. It is suggested [8] that this behaviour represents a possible hazard when combined with tensile stresses that are generated within steel structure on cooling from a fire.

Most steel frame structures are assembled using steel bolts and these bolts represent a potential point of weakness for the structure. In a fire, failure of unprotected bolts could result in collapse of the structure before the steel structural members have reached the critical temperature specified in the various building codes [8]. Hong [8] also noted that painting the bolts after assembly is a feasible option that is currently practiced by most of the industries as a means of increasing their fire resistance. But there are various types of bolts in use in the construction industry. Some of these bolts are discussed below in order to appreciate their uses at a glance as in British Standard Institute [5] provisions.

2.1.1. Unfinished bolts

These are also called ordinary or common bolts. They are classified as A307 bolts and are made from carbon steel with strain-stress characteristics very similar to those of A36 steel. They are available in diameters from 16mm to 38mm. Their design strengths are appreciably smaller than those for high-strength bolts. They are primarily used in light structures subjected to static loads for secondary members such as purlins, girt bracing, platforms, small trusses, etc. The analysis and design of A307 or black bolted connections are handled exactly as riveted connections in every way except that the allowable stresses are slightly different [9].

2.1.2. High-strength bolts

These are made from medium carbon heat-treated steel from alloy steel and have tensile strengths two or more times those of ordinary bolts. There are two basic types, the A325 or High Friction Grip (HSF) bolts (made from a heat-treated medium carbon steel) and are the higher strength A490 bolts (also heat-treated but made from an alloy steel). High-strength bolts are used for all types of structures from small buildings to tall structures and bridges. These bolts were developed to overcome the weakness of rivets which primarily has insufficient tension in their shanks after cooling [10]. Fisher et al [10] also noted that the joints obtained using high-strength bolts are superior to riveted joints in performance and economy and are the leading field methods of fastening structural steel members. Among the many other advantages of high-strength bolts is that, it has no fire hazard effect, has higher fatigue strength and good where structures are to be later altered or disassembled; also changes in connections are quite simple because of the ease of bolts removal as noted by Fisher et al [10].

2.1.3. Snug-tight and fully tensioned bolts

High-strength bolts could be either snug-tight or fully tensioned. These terms refers to the degree to which bolts are tightened. For most connections, bolts are tightened only to the snug-tight condition. Snug-tight is the situation existing when all the plies of a connection are in firm contact with each other. It usually means the tightness obtained after a few impacts of an impact wrench. When loads are applied to snug-tight bolts, there may be a little slippage, as holes are little larger than the shanks of the bolts. As a result, the part of the connections may bear against the bolts.

For fatigue situations in cyclic loading, and connections subject to direct tension, it is desirable to use connections that will not slip. These are referred to as slip-critical connections. To achieve this situation, the bolts must be tightened until they reach a fully tensioned condition if they are subjected to extremely large tensile forces.

2.2. Lap joints

The joint shown in figure 1 is referred to as a lap joint. This type of joints has a disadvantage in that the centre of gravity of the force in one member may not be in line with the centre of gravity of the force in the other members. Thus, a couple may be present that causes undesirable bending in the connection. For this reason, lap joints which are usually desirably used only for minor connections, should be designed

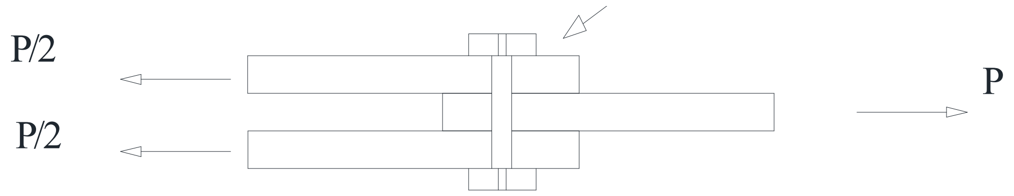

Figure 2: Bolt in double shear in a butt joint.

with at least two fasteners in each line parallel to the length of the member in order to minimize the possibility of bending failure as recommended, for example in [5].

2.3. Butt joint

A butt joint is formed when three members are connected as shown in Figure 2. If the slip resistance between the members is negligible the member will slip a little and tend to shear off the bolts simultaneously on the two planes of contact between the members. Again, the members are bearing against the bolts and the bolts are said to be in double shear and bearing (also called enclosed bearing). The butt joint is more desirable than the lap joint for two main reasons; (1) it provides a more symmetrical loading condition than the lap joint, (2) the members are arranged so that the total shearing force, P is split into two parts.

Double-plane connections are those in which the bolts are subjected to single shear and bearing, but in which bending moment is prevented. This type of connection, subject the bolts to single shear on two different planes.

2.4. Failure of bolted joints

To satisfactorily design bolted joints, it is necessary to understand the possibility or several ways in which failure of the bolted joints can occur. These are described as follows: (1) the possibility of failure in a lap joint by shearing of the bolt on the plane between the members (single shear) as shown in below. (2) the possibility of tension failure of one of the plate through a bolt hole, (3) a possibility failure of the bolts and/or plates by bearing between the two (4) the possibility of failure due to the shearing out of part of the member, (5) the possibility of a shear failure of the bolts along two planes (double shear).

2.5. Spacing and edge distances of bolts

2.5.1. Minimum spacing

Bolts should be placed a sufficient distance apart to permit efficient installation and to prevent bearing failure of the members between fasteners. The Load and Resistance Factor for Reinforced Concrete Design (LRFD) specifications (J3.3) as in [11] provides a minimum centre-to-centre distance for standard, oversized, or slotted fastener holes equal to not less than 2 diameters (with 3 diameters being preferred). Test results have shown that bearing strengths are directly proportional to the 3d centre-to-centre value up to a maximum of 3d, where d is the diameter.

2.5.2. Minimum edge distance

Bolts should not be placed too near the edges of a member for two measure reasons. First, the punching of holes too close to the edges may cause the steel opposite the hole to bulge out or even crack. The second reason applies to the ends of member where there is danger of the fastener tearing through the metal. The usual practice is to place the fastener a minimum distance from the edge of the plates equal to about 1.5 to 2.0 times the fastener diameter so that the metal there will have a shearing strength at least equal to that of the fasteners.

2.5.3. Maximum spacing and edge distances

Structural steel specifications provide maximum edge distances for bolted connections. The purpose of such requirements is to reduce the chances of moisture getting between the parts when fasteners are too far from the edge of the part being connected, the edges may sometimes separate, thus permitting the entrance of moisture. When this happens and there is a failure of the paint, corrosion will developed and accumulate, causing increased separations between the parts. The LRFD maximum permissible edge distance (J3.5) is 12 times the thickness of the connected parts, but not more than 150mm [11].

The LFRD specification (J3.5) states that the maximum spacing of bolts centre-to-centre for painted members or for unpainted members not subject to corrosion is 24 times the thickness of the thinner plate, not to exceed 300mm for unprotected members consisting of weathering steel subject to atmospheric corrosion, the maximum is 14 times the thickness of the thinner plate, not to exceed 175mm.

3. Mathematical Modeling of the Bolt Joints

Reliability based design is founded on the concept that one can estimate the probability of an undesirable

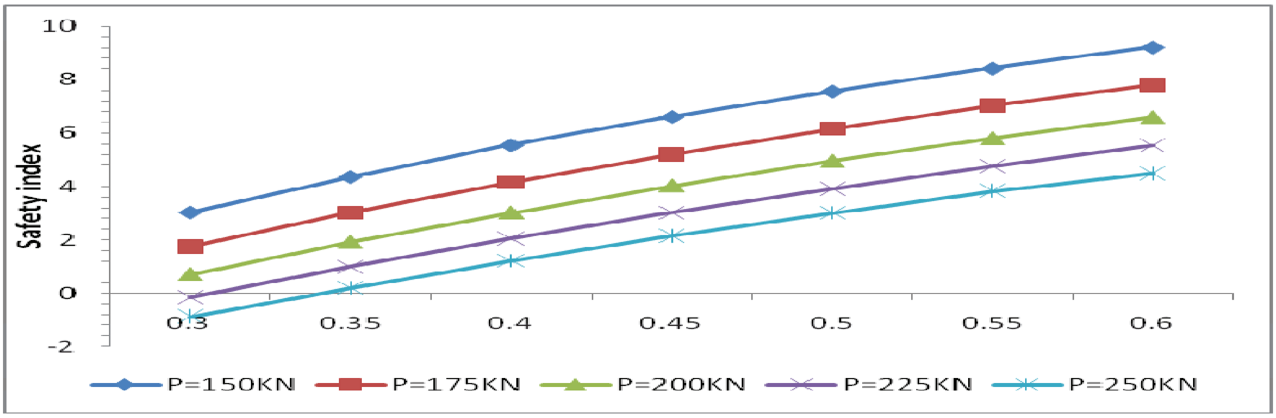

Figure 3: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, n = 4 (Shear Capacity).

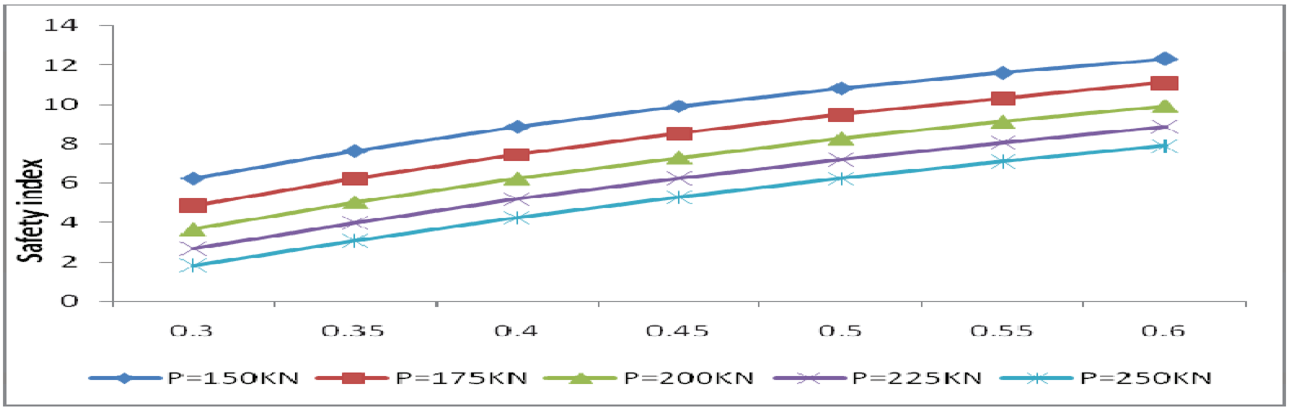

Figure 4: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, n = 6 (Shear Capacity).

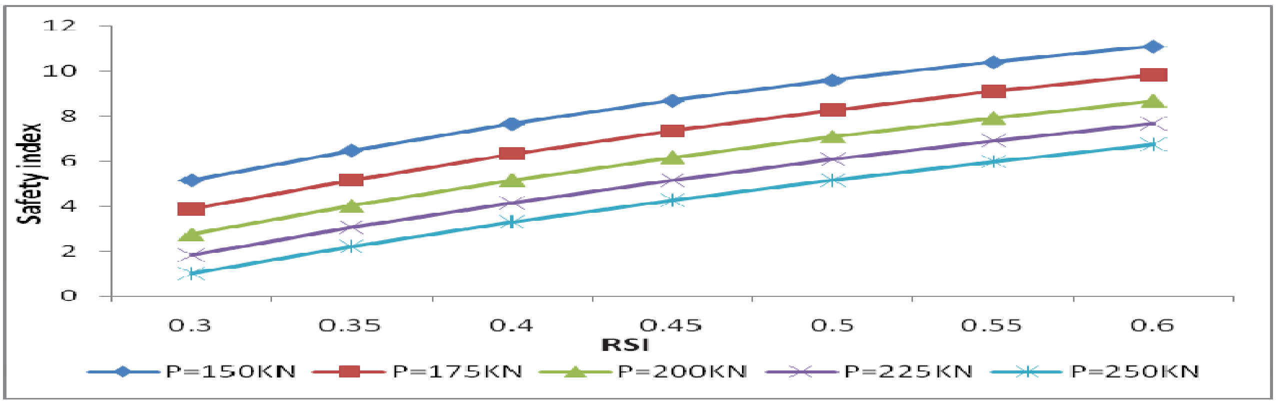

Figure 5: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, n = 4 (Bolt Bearing).

event such as a fracture, occurring over the lifetime of a structure, despite the uncertainties involved [12].

Reliability is defined as the probability of a performance function $g(X)$ greater than zero i.e. $P\{g(X) > 0\}$. In other words, reliability is the probability that the random variables $X_i = (X_1, \ldots, X_n)$ are in the safe region that is defined by $g(X) > 0$. The probability of failure is defined as the probability $P\{g(X) > 0\}$. Or it is the probability that the random variables $X_i = (X_1, \ldots, X_n)$ are in the failure region that is defined by $g(X) > 0$. Now, assume that R and S are random variables whose statistical distributions are known very precisely as a result of a very long series of measurements. R is a variable representing the variations in strength between nominally identical structures, whereas S represents the maximum load effects in successive T-yr periods. Then, the probability that the structure will collapse during any reference period of duration T years is given by [13]:

$$P_f = P(R - S \leq) = \int_{-\infty}^{\infty} F_R(x) f_s(x) dx \qquad (1)$$

where, F_R is the probability distribution function of R and f_s the probability density function of S. Note that R and S are statistically independent and must necessarily have the same dimensions.

The reliability of the structure is the probability that it will survive when the load is applied, given by:

$$R = 1 - P_f = 1 - \int_{-\infty}^{\infty} F_R(x) f_s(x) dx \qquad (2)$$

After a few step of mathematical involvement it is obtained that the reliability index is given as:

$$\beta = -\phi^{-1}(P_f) \qquad (3)$$

where ϕ is the standard normal distribution function.

Therefore, if the designed strength of a joint before fire is adequate for the limit state considered, it therefore means that after the onslaught of fire it is envisaged that the strength of the joint cannot increase more than its original designed value. Now, let us measure the strength of the joint after fire by a parameter which we will denote as the Residual Strength Index, RSI. It therefore means that the designed strength of the joint at the limit state considered has a RSI of unity. This strength can only reduce after fire to say zero when all the components of the joint would have melted, that is, it is structurally nonexistent. But if the joint remains, then its strength would vary between 0 and 1. This formulation is used to measure the strength capacity of a steel lap joint after fire in order to estimate its safety and the probable rehabilitation criteria instead of demolishing the joint or entire structure.

4. Derivation Of Limit State Functions

A structure should fundamentally have the following characteristics during its design, construction or service life; safety, performance of its intended use and economy. A limit state is a state beyond which a structure no longer possesses at least one of these characteristics. Hence a limit state can also be defined as a failure surface or line that separates two regions, namely, the failure and the safe regions. The limit state functions for reliability analysis of a bolted double lap joint derived for different failure conditions are presented stepwise below.

Failure of the joint due to shear

For the connection to be safe against shear failure after fire attack, the equation (4) must be satisfied:

$$F_S \leq \mathrm{RSI} * P_S \qquad (4)$$

The limit state function is thus,

$$G(x) = \mathrm{RSI} * P_S - F_S = \mathrm{RSI} * P_S A_S - P/n \qquad (5)$$

where, RSI is the residual strength index; P_S is the shear capacity of the bolt; F_S is the direct shear per bolt; P_S is the shear strength of the bolt; P is the applied axial force; n is the number of bolts.

Failure of joint by bearing of bolts

For the connection to withstand failure due to bolt bearing after fire attack, the equation (6) must be satisfied:

$$F_S \leq \mathrm{RSI} * P_{bb} \qquad (6)$$

The limit state function is thus,

$$G(X) = \mathrm{RSI} * P_{bb} - F_S = \mathrm{RSI} * dt_p P_{bb} - P/n \qquad (7)$$

(7) where, F_S is the direct shear per bolt; P_{bb} is the bearing capacity of bolt; P_{bb} is the bearing strength of bolt; P is the applied axial force; d is the nominal diameter of the bolt; t_p is thickness of the thinner plate; n is the number of bolts.

Failure of joint due to bearing of the plate

For the connection to withstand failure due to bearing of the plate after attack, the equation (8) must be satisfied:

$$F_S \leq \mathrm{RSI} * P_{bs} \leq \mathrm{RSI} * 0.5 K_{bs} e t_p P_{bs} \qquad (8)$$

The limit state function is thus,

$$G(X) = \mathrm{RSI} * P_{bs} - F_S = \mathrm{RSI} * K_{bs} d t_p P_{bs} - P/n \qquad (9)$$

(9) where, P_{bs} is the bearing capacity of the plate; F_S is the direct shear force per bolt; P_{bs} is the bearing strength of the plate; K_{bs} is a coefficient (for bolts in standard clearance holes, $K_{bs} = 1.0$); e is the end distance; t_p, d, and n are defined as before.

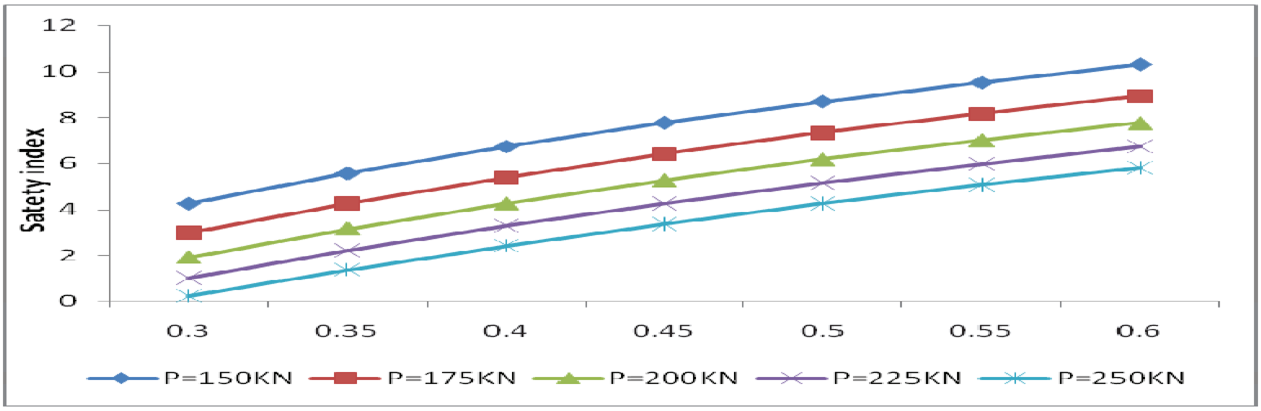

Figure 6: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, n = 6 (Bolt Bearing).

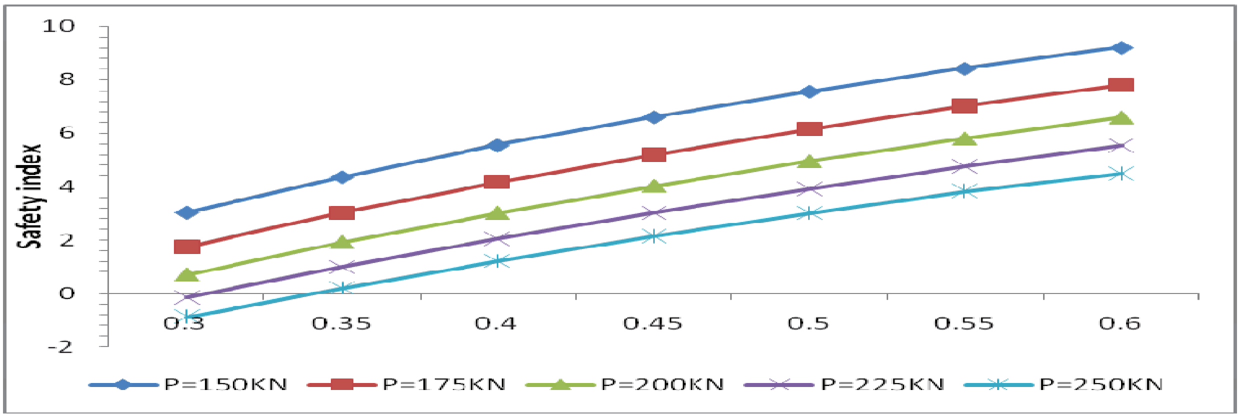

Figure 7: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, t = 8mm (Plate Bearing).

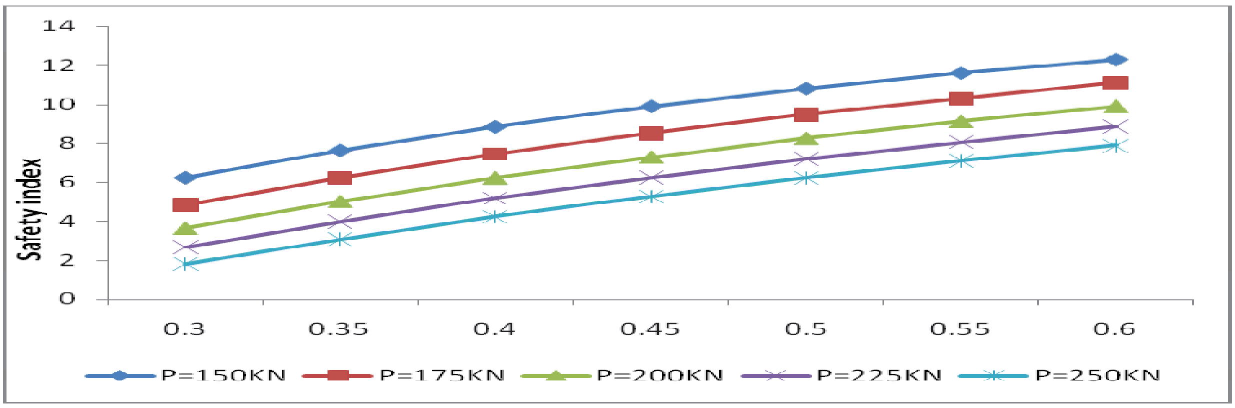

Figure 8: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, t = 10mm (Plate Bearing).

Failure of the joint due to block shear

For the connection to be safe against block shear failure after attack, the condition of equation (10) must be satisfied:

$$F_r \leq \mathrm{RSI} * P_r \tag{10}$$

The limit sate function is thus,

$$G(X) = P_r - \mathrm{RSI}*F_r = \mathrm{RSI}*0.6P_y t(L_v + K_e(L_t k D_t)) - P \tag{11}$$

Where, P_r is the block shear capacity; F_r is the applied force; P_y is the design strength; D_t is the hole size for the tension face; L_v is the length of the shear face; L_t is the length of the tension face; K_e is a coefficient (for two lines of bolts, $K_e = 1.5$); $k = 0.5$, is a coefficient as defined in British Standard Institute [5].

Failure of joint by tension of the plate

For the connection to be safe against tension failure, equation (12) must be satisfied:

$$F_r \leq P_t \tag{12}$$

The limit state function is thus,

$$G(X) = P_t - F_r = P_y A_e - F_r \tag{13}$$

$$G(X) = P_y K_e(B_p - 2D)t_p - F_r \tag{14}$$

where, P_t is the tensile capacity of the plate; A_e is the effective area of the plate; B_p is the width of the plate; D is the hole diameter of the plate; P_y, F_r, K_e, and t_p are all as defined before.

4.1. Computation of safety indices

The First Order Reliability Method (FORM) has been designed to provide approximate solutions of probability integrals occurring in many fields especially structural reliability [13]. FORM as coded in FORM5 [6] was employed in the computation, making use of data obtained from the stochastic modelling [14] with their respective limit state functions. The safety indices were calculated with these data and results based on the bolt sizes and numbers, plate thicknesses are plotted as shown for example in Figures 3 to 14.

4.2. Results of probabilistic evaluation of the residual strength index

The stochastic models generated were analyzed using the FORM to give values of safety indices (β) and probability of failure (P_f) for the bolted double lap joint to British Standard Institution [5], American Institute of Steel Construction [11] and Eurocode [15]. An algorithm developed into FORTRAN module was designed for different failure modes as depicted for shear, bearing and tension with varying residual strength index from 0.10 to 1.0.

The results obtained from the FORM program show that the strength capacity of the joint will start to experience failure usually as the RSI decreases at varying elevated temperatures. In other words, the safety index values decrease significantly at low RSI values indicating that the probability of failure of the structure is high.

The failure due to shear is illustrated from figures 3 and 4. It is observed that the reliability index, β increases with increasing RSI, bolt diameter, and number of bolts connected to the joint.

In failure due to bearing of bolts as illustrated in Figures 5 to 6, it is also observed that the safety of the joint as expressed in terms of the reliability index, β, increases with increasing RSI, bolt diameter and number of bolts and plate thickness under the applied load considered. This type of failure was noticed to be safe even when the bolt sizes decreases as long as the bolt did not fail in shear.

Observing the mode of failure due to bearing of plate indicates that there is a significant capacity reduction of the joint for the RSI of less than 0.70 and for plate thicknesses, t, of 10mm and 8mm as most of the safety indices have values less that the acceptable as opined by Ellingwood [16], Augusti et al [17] and Ditlevsen and Madsen [18]. But the safety index value increases as the plate thickness, t increases as shown in Figures 7 to 9. Hence, steel joints after fire may not be safe enough to guarantee the safety of rehabilitation of the structure except bearing of the bolts on plates are safe enough.

In the modelling of the joints after fire in block shear, it was observed as represented in Figures 10 and 11, which the safety of the steel joints increases as the RSI and member or plate thicknesses increases; but reduces as the applied load increases. Thus adequate precaution should be taken to rehabilitate joints in order to prevent block failure. Also the failure of the joint due to tension in the plate was modelled. Results presented in Figures 12 to 14 indicates that the safety level decreases with decreasing RSI and increases with increase in end distances and bolt spacing under the applied loads considered. Furthermore, the safety levels of the joint was observed to be adequate as shown in Figures 12 to 14 for end distances, e, of 40mm and 50mm respectively. Therefore, it is clear that the joint structure is safer and more reliable when end and edge distances are adequate and not less than recommended in British Standard Institution [5], American Institute of Steel Construction [11] and Eurocode [15].

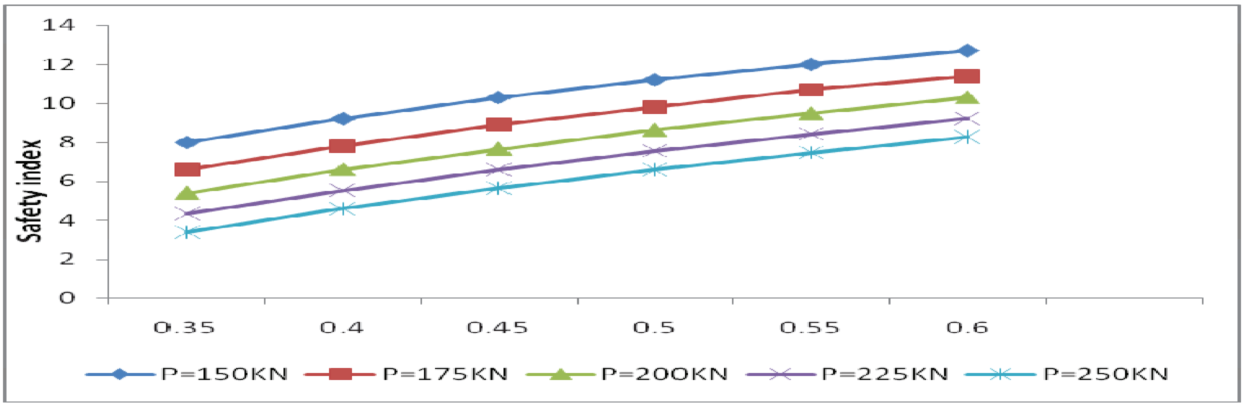

Figure 9: Variation of Safety Index with Residual Strength Index (RSI) for ϕ = 20mm, t = 12mm (Plate Bearing).

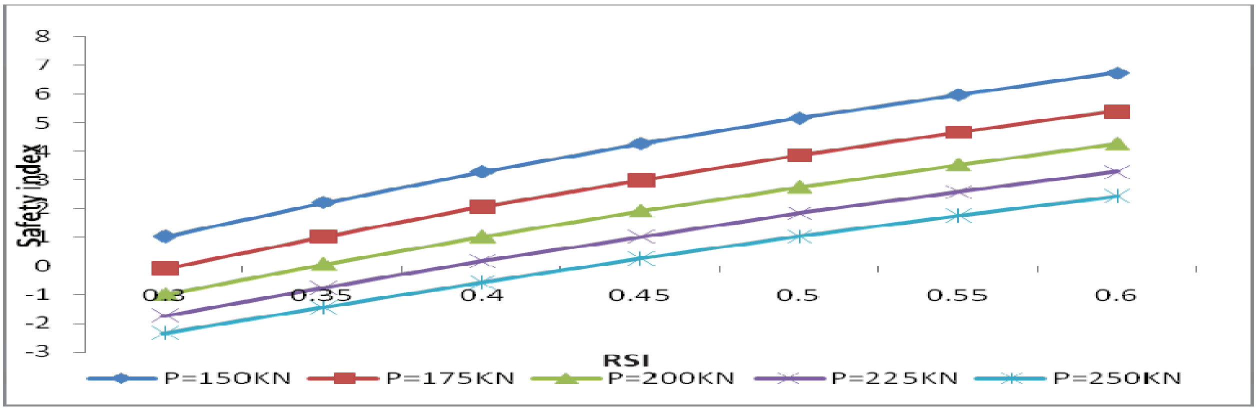

Figure 10: Variation of Safety Index with Residual Strength Index (RSI) for t = 10mm (Block Shear).

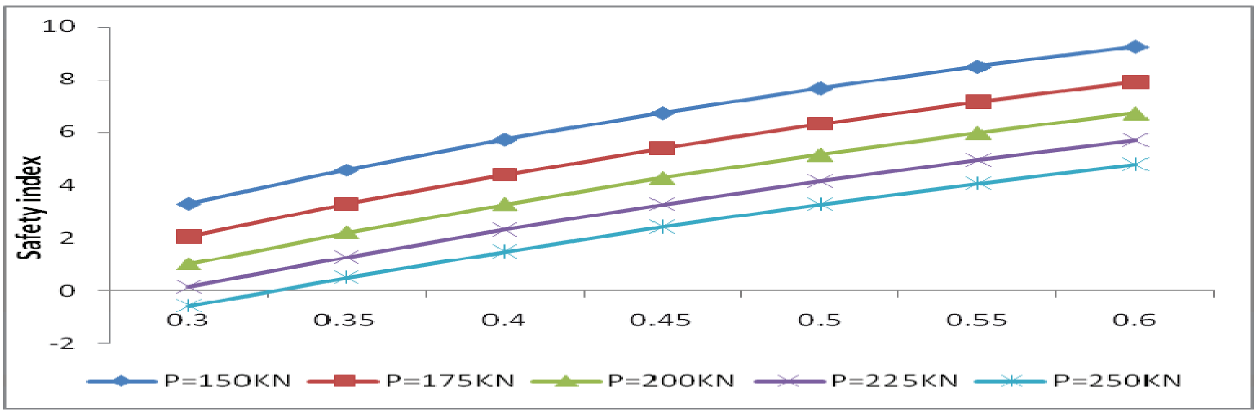

Figure 11: Variation of Safety Index with Residual Strength Index (RSI) for t = 14mm (Block Shear).

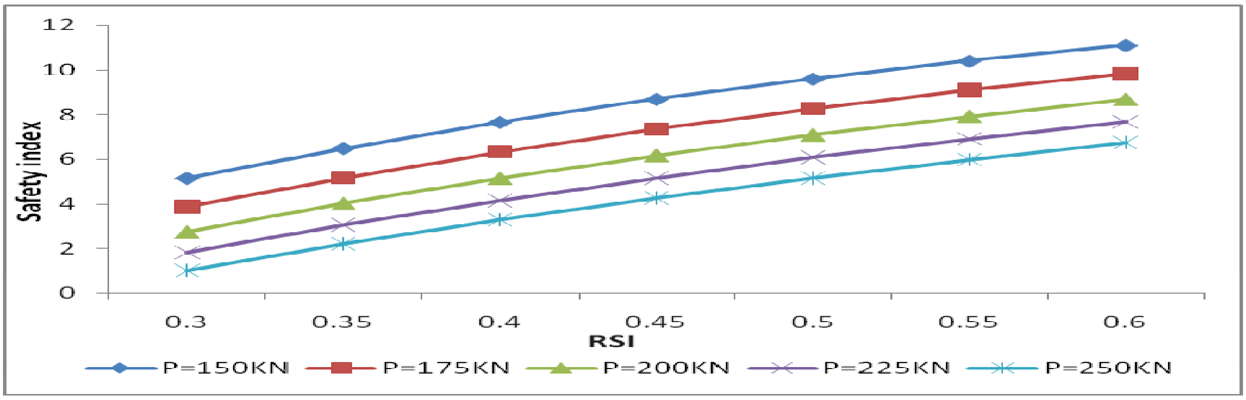

Figure 12: Variation of Safety Index with Residual Strength Index (RSI) for e = 40mm, s = 50mm (Tension Capacity).

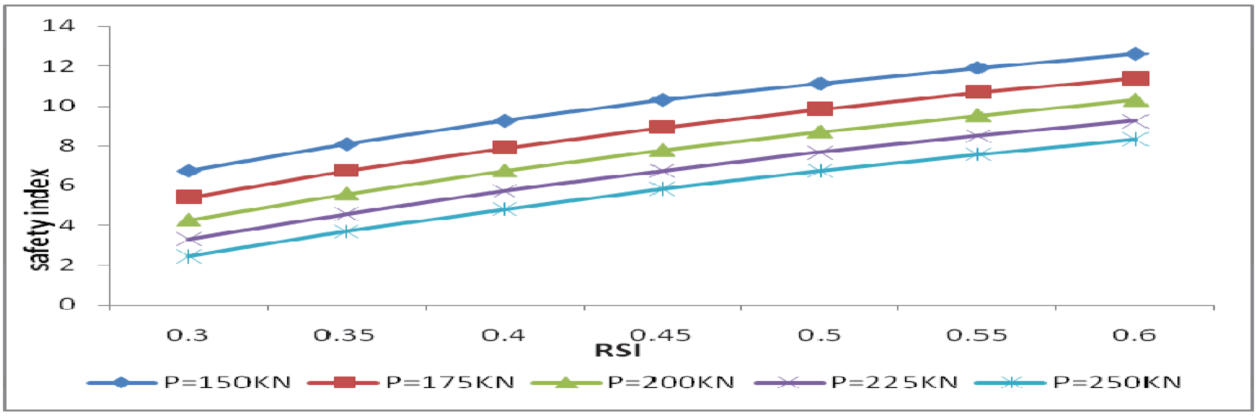

Figure 13: Variation of Safety Index with Residual Strength Index (RSI) for e = 50mm, s = 50mm (Tension Capacity).

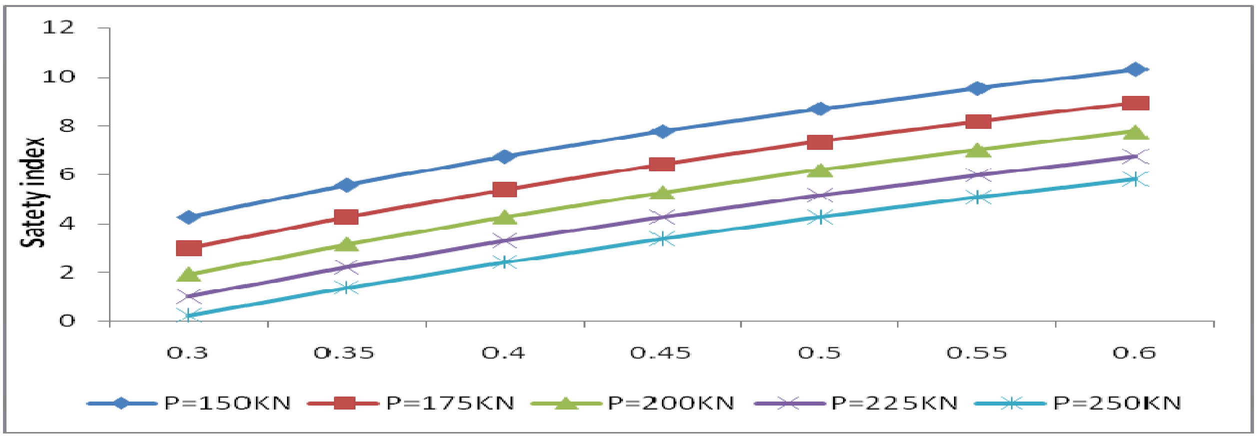

Figure 14: Variation of Safety Index with Residual Strength Index (RSI) for e = 40mm, s = 60mm (Tension Capacity).

5. Conclusion

The limit state functions for the various failure modes for bolted double lap joints were suitably considered for probabilistic assessments. The level of safety which is associated with each of the modes of failure was computed using the First Order Reliability Method (FORM). It was observed that the residual strength index (RSI) is proportional to the safety of the joint under applied loads. However, the safety of these steel bolted joints increases with increases in bolt sizes or diameters, member thicknesses, number of bolts and end / edge distances. The essence of the consideration of Residual Strength Index (RSI) is to provide results based on adequate engineering judgement in order to determine the safety of a structure after the onslaught of fire. As a result, the safety indices obtained suggest the consideration and type of rehabilitation of the joint and entire structure so as to achieve the optimum design for the joint and subsequently the entire structure. Thus it was observed that high strength bolts (or grade 8.8) of 20mm diameter have high resistance after fire or do not significantly lose their strength after fire, and hence can be considered safe and economical in steel structures where fire may readily occur.

References

1. Wei-Yong W. Experimental Study and Spring- component Modelling of Extended End-plate Joint in Fire. *Journal of Construction Steel Research*, Vol.63, August 2007, pp.1227 – 1137.
2. MacGinley T.J. *Structural Steelwork: Design to Limit State Theory.* Biddles Publishers, Guildford and Kings Lynn, Great Britain, 1998.
3. Turvey G. J. Thermal Preconditioning Study for Bolted Tension Joints in Pultruded GRP Plate. *Composite Structures*, Vol.77, February 2007, pp. 509 – 513.
4. Newman, G.M., Robinson, J.T. and Bailey, C.G. *Fire Safe Design: A New Approach to Multi-story Steel-framed Buildings.* Steel Construction Institute, Berkshire, U.K, 2000.
5. British Standard Institution. *The Structural Use of Steel in Buildings.* Her Majesty's Stationery Office, London, 2000.
6. Gollwittzer, S., Abdo, T. and Rackwitz, R. *First Order Reliability Method, FORM Manual.* RCP Gmbh, Nymphenburger Str. 134, MUNCHEN, Germany, 1988.
7. Kodur, V.K.R. Design Equations for Evaluating Fire Resistance of SRFC-filled Steel columns. *Journal of Structural Engineering*, Vol. 124, No.6, 1998, pp. 671 677.
8. Hong P.J. Susceptibility of Unprotected Steel Bolted Connection to Embrittlement after Fire. *Composite Structures*, Vol. 65, 1994, pp.507 – 512.
9. Davidson, B. and Owens, G.W. *Steel Designers Manual.* 6th Edition, Blackwell Publishing Inc, Oxford U.K, 2005, pp: 671-1265.
10. Fisher, J.W. and Struik, J.H.A. *Guide to Design Criteria for Bolted and Riveted Joints.* Wiley Inter-Science Publishers, Fritz Engineering Laboratory, Lehigh University Bethlehem Pennsylvania, 1974, pp: 1 – 69.
11. American Institute of Steel Construction. *Specification for Structural Steel Buildings.* ANSI/AISC, One East Walker Drive, Suite 700, Chicago, Illinois U.S.A., 2005, pp: 343 -350.
12. Abejide, O. S. Appraisal and Reliability Of Variable Engagement Model Prediction For Fibre Reinforced Concrete. *Nigerian Journal of Technology*, Vol. 27 No.2, 2008, pp 78 – 92.
13. Shema, M. A and Abejide, O. S. Safety of Premature Loading on Reinforced Concrete Slabs. *Nigerian Journal of Technology*, Vol. 28 No.1, 2009, pp 5 – 15.
14. Fiorato, A.E. *Geometric Imperfections in Concrete Structures.* National Building Research report, D5, Stockholm, Sweden, 1973.
15. Eurocode 3 *Design of Steel Structures*, (ENV 1993-1-1: 1992). Edited Approved Drafts, April 1996, pp: 116 – 122.
16. Ellingwood, B.R. *Reliability Bases of Load and Resistance Factor for Reinforced Concrete Design.* National Bureau of standards, Building Science Series 110, Washington, D.C., USA, 1978.
17. Augusti, G., Baratta, A. and Casciati, F. *Probabilistic Methods in Structural Engineering.* Hapman and Hall, New York, 1984.
18. Ditlevsen, O. and Madsen, H.O. *Structural Reliability Method*. First edition published by John Wiley & Sons Ltd, Chichester, 1996, pp. 279 – 280.