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DESIGN OF A THREE HINGED TIMBER PORTAL FRAME BASED ON EUROCODE 5 SAFETY LEVEL INDEX

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ABSTR*ACT*
*A typical 3-hinged timber portal frame is designed to Eurocode 5 design specification and the safety level of the members assessed using the First Order Reliability Method (FORM). The failure modes associated with the members were identified and limit state equations related to each mode of failure is generated and written as subroutines using FORTRAN language in FORM. The safety index (**β**) for each failure mode was computed at varying load ratios, **α** (dead to live load ratio) from 0.2 to 1.0. It t was observed that for the varying load ratio's (**α**), the predominant failure in the rafter member is bending failure with computed safety indices, **β** of 2.089 to 0.661; shear failure was identified for the column member with **β**- values of 2.230 to 0.675; failure mode **(i)** for both eaves and apex joints with **β**-values of 1.791 to 0.294 and -0.453 to -1.996 respectively for the **α** values considered. The results suggest that structural failure is most likely to occur at the apex joint for all possible loading conditions and more attention should be given to it during design.*
Keywords: *Eurocode 5, First order reliability method (FORM), load ratio, Safety level index, Three hinged Portal frame, Timber.*

INTRODUCTION

Portal frames are the most commonly used structural forms for single- storey industrial structures such as warehouses, factories and many other purposes. Portal frames may be a rigid, semi – rigid and have a pinned base (hinged base) construction. The hinged – base portal is the most common type adopted because of the greater economy in foundation design.

A portal frame consists of vertical member called Columns and top member which may be horizontal, curved or pitched. The vertical and top members' built monolithically are considered as rigidly connected. They are used in the construction of large sheds, bridges and viaducts. The base of portal frame may be hinged or fixed. The portal frames are spaced at suitable distance and it supports the slab above the top members.

Portal frame structures are an effective wood construction type for resisting lateral load such as seismic and wind loads (Yeh *et al.*, 2014). In present day engineering practice, portal frames of glued laminated timber are frequently used for industrial buildings, for economic, functional or aesthetic reasons. In these buildings the major structural elements consist of a series of parallel portal frames. Each frame is capable of transmitting loads, which act in the plane of the frame down to the foundations.

The existence of uncertainties in structural engineering has long been recognized and qualitatively accounted for through the use of safety factors in the design of structures. In recent decades, reliability analysis of structures has become a great topic of interest between engineers and designers. This type of analysis inserts the probabilistic uncertainties in load and resistance models of structures that always exist but often neglected during conventional deterministic analysis and design methods (Ocholi, 2000). Structural reliability is therefore carried out to determine the safety levels and probability of failure. These failures can be manifested on the structure at any hierarchic level, in particular, the members, the units, the whole structural system and the connections (Tampone, 2001).

In the analysis of a structure, the goal is to analyze and design the safety margins so that the risk of failure is small. To ensure that each structure will be fit for its intended purpose at any time during its specified design life, an accepted level of probability must be attained. Structural reliability is therefore carried out to determine the safety levels and probabilities of failure as illustrated by Rantu-Manus (2004) and Vilarinho *et al.* (2011) using the practical and theoretical use of reliability analysis of timber structures and the use of probabilistic analysis as provided in JCSS (2000) combined with action and resistance models provided by the Eurocodes was used in checking the safety of traditional timber trusses.

The variability of the uncertainties in timber was further highlighted by Larsen (2001) in which he noted that Structural timber is not a manufactured but a graded natural material, the behavior of which varies not only between members but also within a member. Variability and uncertainties are apparent in the deterministic approach as employed by most design codes. A rational way in evaluating the uncertainties inherent in design is utilizing a probabilistic approach as evident in the safety level evaluations of various structural elements by Au, (2005); Ocholi and Hamza (2012); Ocholi *et al.* (2012) and Behshad and Ghasemi 2013).

Adjanohoun *et al.* (1997) suggested the need to gather scientific information and propose guidelines for the implementation of a fully probabilistic design code of timber structures. They suggested that the method of reliability research and interpretation of different results; importance of the choice of distribution function of the material; Strength dependency of the load history; systems effect and multiple failure modes and other areas should be carefully investigated and analyzed with reliability based methods. Zhong *et al.* (2016) buttressed this fact by carrying out a study which indicated that the reliability index (a measure of safety level) increased none linearly with a decrease in the partial safety factor and live to dead load ratio.

In this study, possible design uncertainties and safety margins of a typical 3 – hinged timber portal frame is assessed using a probabilistic approach of the First Order Reliability Method (FORM), the basic variables that constitute the design parameters for the members of a three-hinged timber portal frame for each specific limit state failure are evaluated and implied safety level associated with all the failure modes computed. The intention is to point out failure modes that are most likely to initiate failure and intimate designers on areas that require careful consideration for a typical three-hinged timber portal frame.

METHODS

First Order Reliability Method

The first development of First Order Reliability Method also known as FORM took place almost 30 years ago. Since then the method has been refined and extended significantly as one of the most important methods for reliability evaluations in structural reliability theory (Faber, 2007).

FORM has been designed for the approximate computation of general probability integrals over given domains with locally smooth boundaries but especially for probability integrals occurring in structural reliability (Gollwitzer *et al.*, 1988).

In reliability analysis of technical systems and components the main problem is to evaluate the probability of failure corresponding to a specified reference period. However, also other non-failure states of the considered component or system may be of interest, such as excessive damage and unavailability (Faber, 2007).

The general problem to which FORM provides an approximate solution is as follows. The state of a system is a function of many variables some of which are uncertain. These uncertain variables are random with joint distribution function $F_X(X) = P(\bigcap_{i=1}^{n}\{X_i \le \mathrm{X}_i\})$ defining the stochastic model. For FORM, it is required that $F_X(X)$, is at least locally continuously differentiable. The random variables $X = (X_1, \dots X_n)^T$ are called basic variables. The locally sufficiently smooth state function is denoted by $g(\boldsymbol{X})$. It is defined such that;

- $g(\boldsymbol{X}) > 0$, corresponds to favorable(safe, intact, acceptable…) states
- $g(\boldsymbol{X}) = 0$, denotes the so-called limit state or the boundary failure
- $g(\boldsymbol{X}) < 0$, sometimes also $g(\boldsymbol{X}) \le 0$) defines the failure (unacceptable, adverse…) domain.

Among other useful information, FORM produces an approximation to the probability of failure:

$$P_f = P(X \in F) = P(g(X) \le 0) = \int_{g(X)\le 0} F_X(X)\, \mathrm{d} \qquad (1)$$

The complement $1 - P_f$ is the reliability of the structure i.e

$$P_r = 1 - P_f \qquad (2)$$

The corresponding reliability index **"β"** is defined as

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

that is, $P_f = \phi(-\boldsymbol{\beta})$ where ϕ is the standard normal cumulative distribution function

Analysis of frame

A typical 3-himged timber portal is analyzed and designed comprising the following component members: columns, rafter, eaves and apex joint as depicted in Figure 1.The frame is symmetrical so only half of the members will be considered for analysis and design.

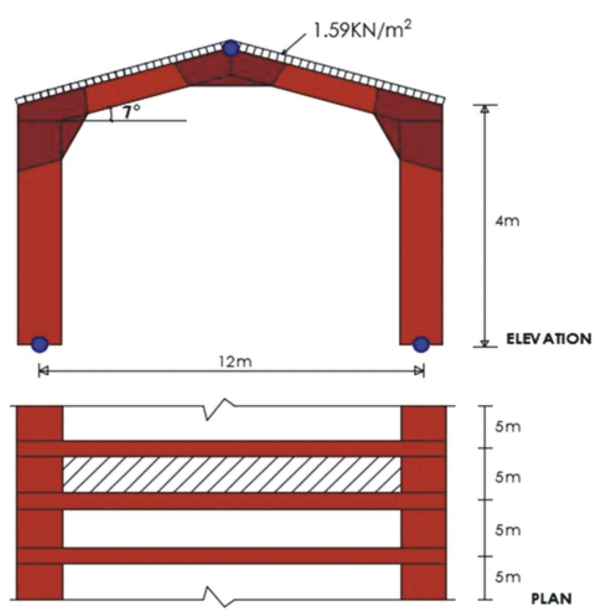

Figure.1: The plan and elevation views of the portal frame under investigation

The estimated loading on the frame and reactive forces acting on the frame are idealized and presented in Figure 2.
Load carried by one frame; $= 1.59 \times 5 = 7.95$ kN/m.

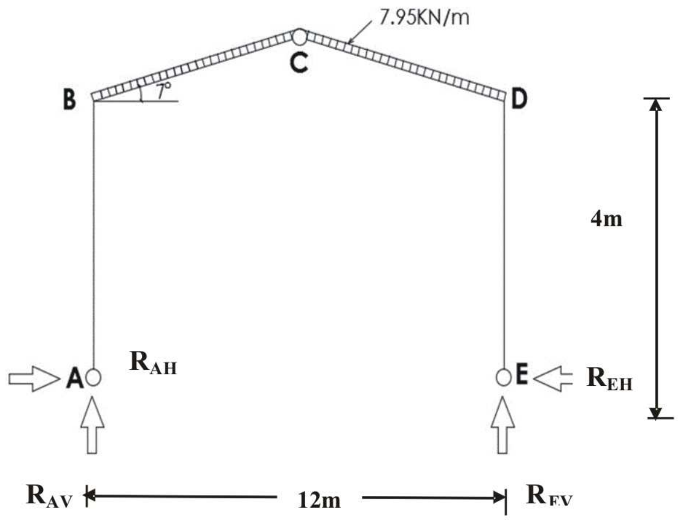

Figure 2: Idealized portal frame with reactive forces

The computed reactive forces, member forces and moments at the joints are calculated and shown in Table 1.

Table 1: Summary of member forces, reactive forces and moments at the joints

Member Forces (kN)	**Reactive Forces** (kN)	**Moments at Joints** (kNm)
Columns AB = DE = 30.19 Rafters BC = CD = 30.42	R_{AH}= R_{EH} = 30.19 R_{AV}= R_{EV} = 47.70	Joint B - M_{BC} = M_{DC} = 120.60 Joint C, A&E = 0

RESULTS AND DISCUSSION

Geometric Parameter of Frame Members and Joints

The Columns, rafters and joints were designed based on the procedure and specifications of Eurocode 5. A summary of the geometric and timber specification properties of these members are summarized in Table 2.

Table 2: Geometric and timber specification properties of the portal frame

Members	**Geometric Properties**	**Timber Specification Properties**
Columns	Column length, $L = 4.0\ m$; Width $b = 150\ mm$; Depth $h = 300\ mm$	Strength class C22 (BS EN 338:2003, *Table 1*)
Rafters	Breadth, $b = 150\ mm$;Depth,$h = 400\ mm$; Bearing length of the rafter at one end, $l_b = 300mm$; Design span, $l = 6.05\ m$	Strength class C22 (BS EN 338:2003, *Table 1*)
Eaves Joint	Thickness of each plywood gusset plate,$t_g = 40\ mm$; Thickness of the timber, $t_t = 150\ mm$; Width of timber member 1,$w_1 = 400\ mm$; Width of timber member 2,$w_2 = 300\ mm$; Bolt diameter, $d = 25\ mm$;Tensile strength of each bolt, $f_{u,k} = 830\ N/mm^2$	Strength class C22 (BS EN 338:2003, *Table1*) ;BSEN 12369-2:2004) 40mm finish birch plywood
Apex Joint	Thickness of each plywood gusset plate,$t_g = 40\ mm$; Thickness of the timber member , $t_t = 150\ mm$; Width of timber member ,$w = 400\ mm$; Bolt diameter, $d = 25\ mm$;Tensile strength of each bolt, $f_{u,k} = 830\ N/mm^2$	Strength class C22 (BS EN 338:2003, *Table1*) ;BS EN 12369-2:2004) 40mm finish birch plywood

All relevant strength characteristics and modulus of elasticity are derived and specified from the code as stipulated in Table 2. The details of the members and joints areas shown in Figures 2 a, b, c, and d respectively.

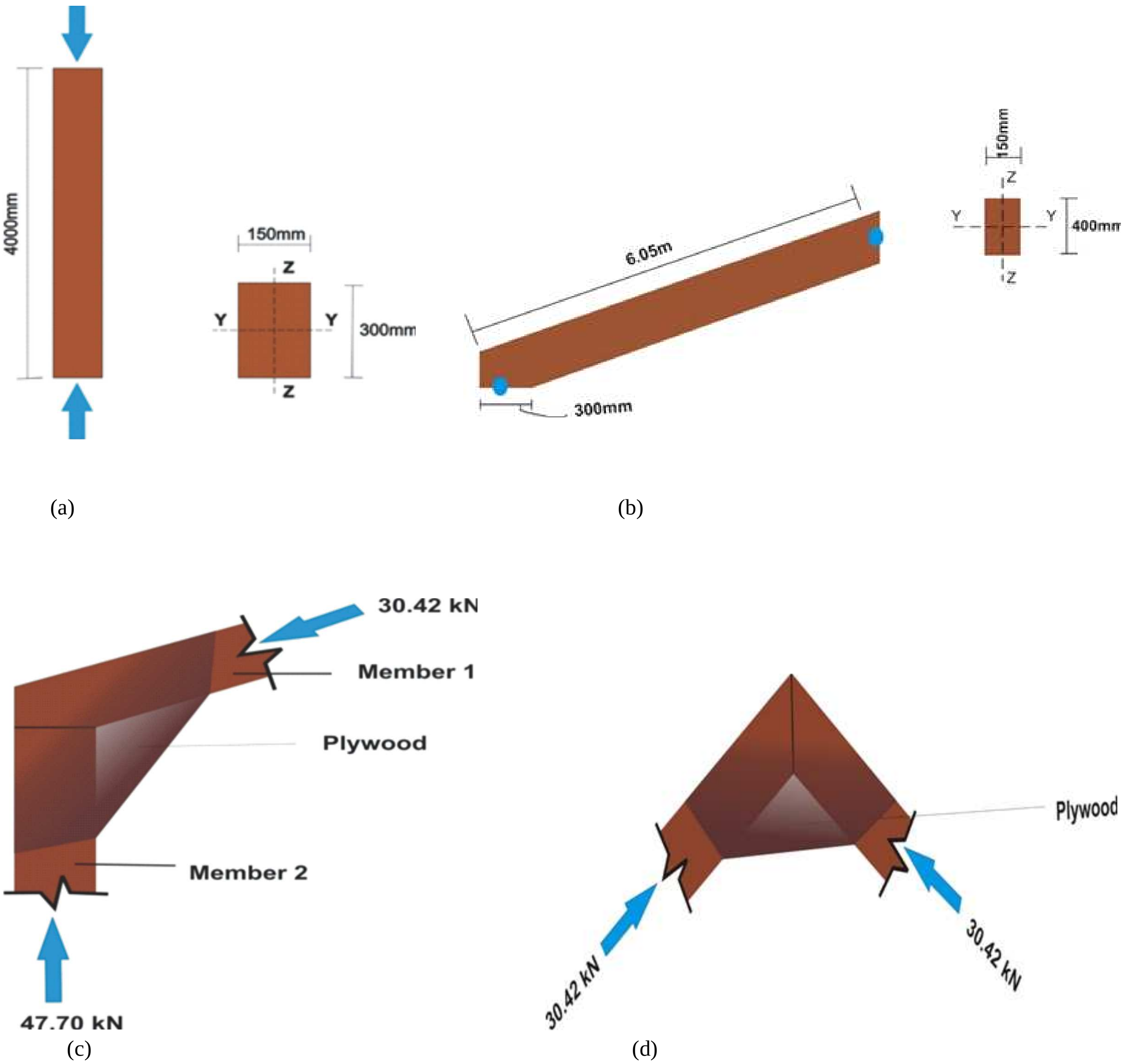

Figure 2: Details of portal frame showing (a) Column, (b) Rafter, (c) Eaves and (d) Apex

Derivation of Limit State Equations

The limit state equation for the failure modes associated with the frame members and joint are derived from the general limit state equation:

$$G = R - S, \quad (4)$$

Where R= resistance and S= load effect.

The limit state equations for the various failure modes associated with the members and joints were derived and listed from Equations (5) to (20):

Columns:

$$G_{CComp} = Compression\ failure = 0.62 f_{c,0.k} - \frac{(Q_k(1.35\ \alpha + 1.5\ ))}{b.h} \quad (5)$$

$$G_{CBuck} = Buckling\ failure = 0.16\ f_{c,0.k} - \frac{(Q_k(1.35\ \alpha + 1.5\ ))}{b.h} \quad (6)$$

$$G_{Cshear} = \text{Shear failure} = 0.62\, f_{v.k} - \frac{1.5\,(Q_k(1.35\,\alpha+1.5\,))}{b.h} \quad (7)$$

Rafters:

$$G_{RBend} = \text{Bending failure} = 0.62\, f_{m.k} - \frac{0.75\,(Q_k(1.35\,\alpha+1.5\,)l^2)}{b.h^2} \quad (8)$$

$$G_{RShear} = \text{Shear failure} = 0.62\, f_{v.k} - \frac{0.75\,(Q_k(1.35\,\alpha+1.5\,)l)}{b.h} \quad (9)$$

$$G_{RComp} = \text{Compression failure} = 0.62\, f_{c,0.k} - \frac{(Q_k(1.35\,\alpha+1.5\,))}{b.h} \quad (10)$$

$$G_{RBuck} = \text{Buckling failure} = 0.07\, f_{c,0.k} - \frac{(Q_k(1.35\,\alpha+1.5\,))}{b.h} \quad (11)$$

$$G_{RBear} = \text{Bearing Failure} = 0.62\, f_{c,90.k} - \frac{0.5\,(Q_k(1.35\,\alpha+1.5\,)l)}{b.l_b} \quad (12)$$

Eaves Joint:

$$G_{Eaves,(g)} = \text{failure mode } (g) = (0.11 - 0.0011d)\rho_{p,k}t_g d - \frac{Q_k(1.35\,\alpha+1.5\,)l}{4} \quad (13)$$

$$G_{Eaves,(h)} = \text{failure mode } (h) = (0.041 - 0.00041d)\rho_k t_t d - \frac{Q_k(1.35\,\alpha+1.5\,)l}{4} \quad (14)$$

$$G_{Eaves,(i)} = \text{Failure mode } (i) = \left[0.438(0.11 - 0.0011d)\rho_{p,k}t_g d\left[\sqrt{1.12 + \frac{1.152\, f_{u,k}d^{2.6}}{(0.11-0.0011d)\rho_{p,k}{t_g}^2}} - 0.4\right]\right] - \frac{Q_k(1.35\,\alpha+1.5\,)l}{4} \quad (15)$$

$$G_{Eaves,(j)} = \text{failure mode } (j) = \left[0.870\sqrt{0.6f_{u,k}d^{2.6}(0.11 - 0.0011d)\rho_{p,k}d}\right] - \frac{Q_k(1.35\,\alpha+1.5\,)l}{4} \quad (16)$$

Apex Joint:

$$G_{Apex,(g)} = \text{failure mode}(g) = (0.11 - 0.0011d)\rho_{p,k}t_g d - \frac{Q_k(1.35\,\alpha+1.5\,)l}{2} \quad (17)$$

$$G_{Apex,(h)} = \text{failure mode}(h) = (0.041 - 0.00041d)\rho_k t_t d - \frac{Q_k(1.35\,\alpha+1.5\,)l}{2} \quad (18)$$

$$G_{Apex,(i)} = \text{Failure mode } (i) = \left[0.438(0.11 - 0.0011d)\rho_{p,k}t_g d\left[\sqrt{1.12 + \frac{1.152\, f_{u,k}d^{2.6}}{(0.11-0.0011d)\rho_{p,k}{t_g}^2}} - 0.4\right]\right] - \frac{Q_k(1.35\,\alpha+1.5\,)l}{4} \quad (19)$$

$$G_{Apex,(j)} = \text{failure mode}(j) = \left[0.870\sqrt{0.6f_{u,k}d^{2.6}(0.11 - 0.0011d)\rho_{p,k}d}\right] - \frac{Q_k(1.35\,\alpha+1.5\,)l}{2} \quad (20)$$

Where all the Symbols and Abbreviations from Equations (5) to (20) are defined in BS EN 338(2003), BS EN 12369-2(2004) and BS EN 1995(2002): Eurocode 5

Computation of Safety Indices

Based on the parameters from the limit state equations for Column, Rafter, Eaves joint and Apex joint , the stochastic models of the basic variables for the frame using the initial design dimensions (Table 2) for each failure mode for the members and joints are presented in Tables 3,4, and 5.

Table 3: Stochastic model parameters for the failure modes of the columns

S/No.	Basic variables		Distribution Type	Expected Value (E(X))	Coefficient of Variation	Standard Deviation (S(X))
1	Characteristic compression Strength parallel to grain ($f_{c,0.k}$)(N/mm^2)		Log-normal	20	0.05	1.00
2	Characteristic shear Strength ($f_{v.k}$)(N/mm^2)		Log-normal	2.4	0.05	0.12
3	Imposed Load (Q_k)	Compression Buckling failure (N)	Log-normal	21000	0.3	6300
		Shear failure(N)	Log-normal	13284	0.3	3985.2
4	Breadth (b)(mm)		Normal	150	0.05	7.50
5	depth (h)(mm)		Normal	300	0.05	15.00
6	Length (L)(mm)		Normal	4000	0.05	200

(Source: EN 338, JCSS 2006; Afolayan 2005)

Table 4: Stochastic model parameters for the failure modes of the eaves and apex joints

S/No.	Basic Variables		Distribution Type	Expected Value (E(X))	Coefficient of Variation	Standard Deviation (S(X))
1	Characteristic Bending Strength $(f_{m.k})$(N/mm^2)		Log-normal	22	0.05	1.10
2	Characteristic shear Strength $(f_{v.k})$(N/mm^2)		Log-normal	2.4	0.05	0.12
3	Characteristic compression Strength perpendicular to grain $(f_{c,90.k})$ (N/mm^2)		Log-normal	2.4	0.05	0.12
4	Characteristic compression Strength parallel to grain $(f_{c,0.k})$ (N/mm^2)		Log-normal	20	0.05	1.00
5	Imposed Load (Q_k)	Bending and shear failure (N/mm^2)	Log-normal	3.5	0.3	1.05
		Compression and Buckling failure (N)	Log-normal	13385	0.3	4016
6	Length (L)(mm)		Normal	6050	0.05	302.50
7	Breadth (b)(mm)		Normal	150	0.05	7.50
8	Depth (h)(mm)		normal	400	0.05	20.00
9	Bearing length (l_b)(mm)		Normal	400	0.05	20.00

(Source: EN 338, JCSS 2006; Afolayan 2005)

Table 5: Stochastic model parameters for the failure modes of the rafters

S/No.	Basic Variables	Distribution Type	Expected Value (E(X))	Coefficient of Variation	Standard Deviation (S(X))
1	Characteristic density of the plywood$(\rho_{p,k})$(kg/m^3)	Log-normal	630	0.05	31.50
2	Characteristic density of the timber(ρ_k) (kg/m^3)	Log-normal	340	0.05	17.00
3	Imposed Load (Q_k)(N/mm)	Normal	3.5	0.3	1.05
4	Thickness of plywood gusset plate (t_g) (mm)	Normal	40	0.05	2.00
5	Length (L)(mm)	Normal	12000	0.05	600
6	Diameter of Bolt(d) (mm)	Normal	25	0.05	1.25
7	Thickness of plywood timber(inner member) (t_t)(mm)	Normal	150	0.05	7.50
8	Tensile strength of bolt $(f_{u,k})$(N/mm^2)	Log-normal	830	0.05	41.50

(Source: EN 338, JCSS 2006; Afolayan 2005)

The stochastic parameters of the basic variables were used in writing subroutines of the failure modes in FORM5, using FORTRAN programming language. The safety indices ,'**β**'-values, which indicate the level of safety or otherwise for each limit state equation is computed using FORM5 computer programs for all members and joints. The load ratio '**α**' which is the ratio of dead to imposed load ($\propto = 0.57$ is the design ratio) is varied from 0.2 to 1.0 for each of the corresponding modes of failures for the Columns , Rafters, Eaves and Apex joints with the safety levels (**β**) computed and the results are presented in Figures1,2,3 and 4 respectively.

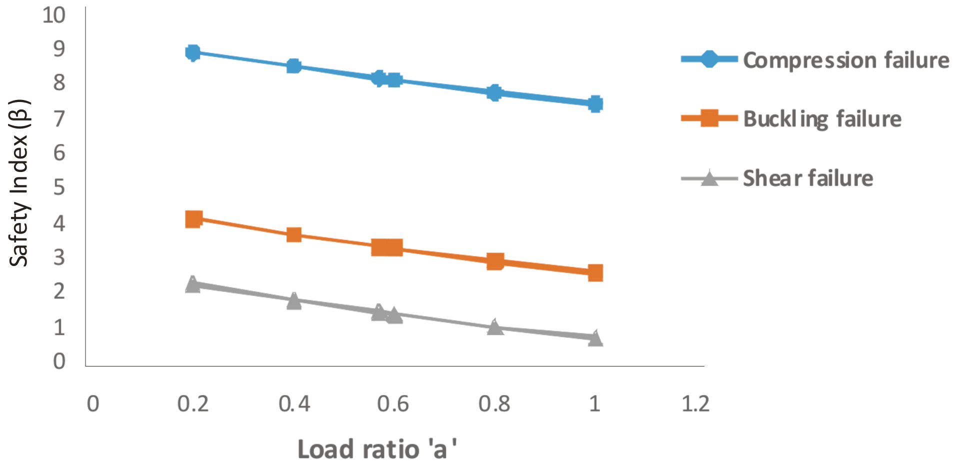

Figure 1: Safety level variations for column member failure modes

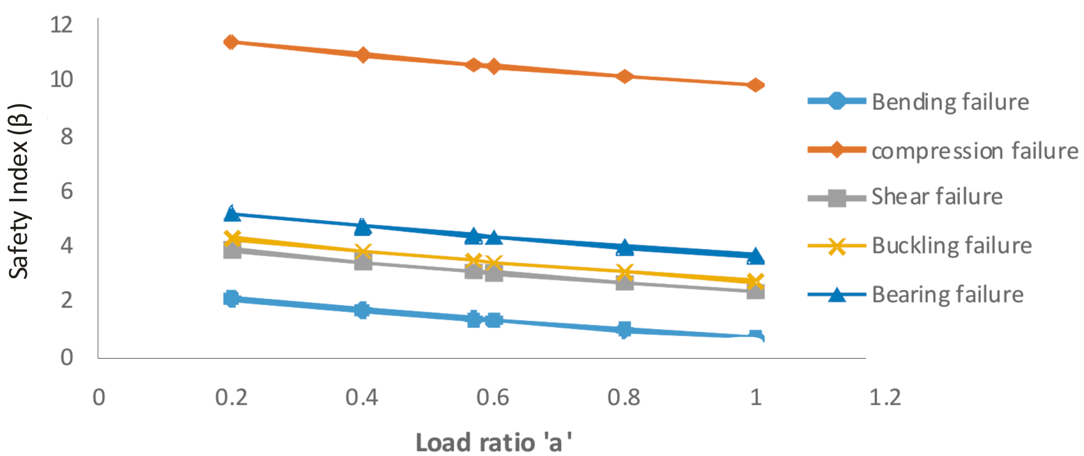

Figure 2: Safety level variation of the rafter member failure modes

Figure 1 shows the safety level variations of the column member failure mode at varying load ratios "**α**". From the plot there is a general decrease in the safety level "**β**" for all the failure modes identified as the **α**-value increases. This may be due to a decrease in the axial carrying capacity of the column section. The graph also shows that for the column member failure mode the most likely to exhibit failure is the shear failure mode as indicated by the **β**-values ranging from 2.2 to 0.7 for all the load ratios considered. These values are much less than the **β**-value of 3.3 recommended in BS EN 1990(2002) for a reliability class 3 (RC3) consequence class1 (CC1) structure. It is clear the level of safety with regards to the column failure is in the order: Shear failure - Buckling failure –Compression failure. The compression failure mode being the safest (Figure 1) as depicted by the high **β**-values of 8.9 and 7.4 for load ratio's (**α**) 0.2 and 1.0.

Figure 2 depicts the safety level variation of the rafter members' failure modes identified at varying load ratios ranging from 0.2 to 1.0. The graph shows a general trend of decreasing value of β for all failure modes as the load ratio is increasing. The decrease in the safety level may be attributed to the reduction in the flexural resistance of the rafters as a result in a decrease in the stiffness of the members. The plot also suggest that the bending failure mode shows the highest possibility for initiating failure, with the **β**-values for this failure mode ranging from 2.1 to 0.7 at load ratios of 0.2 to 1.0. The level of safety with regards to the Rafter member is in the order of: Bending failure – Shear failure – Buckling failure – Bearing failure – Compression failure.

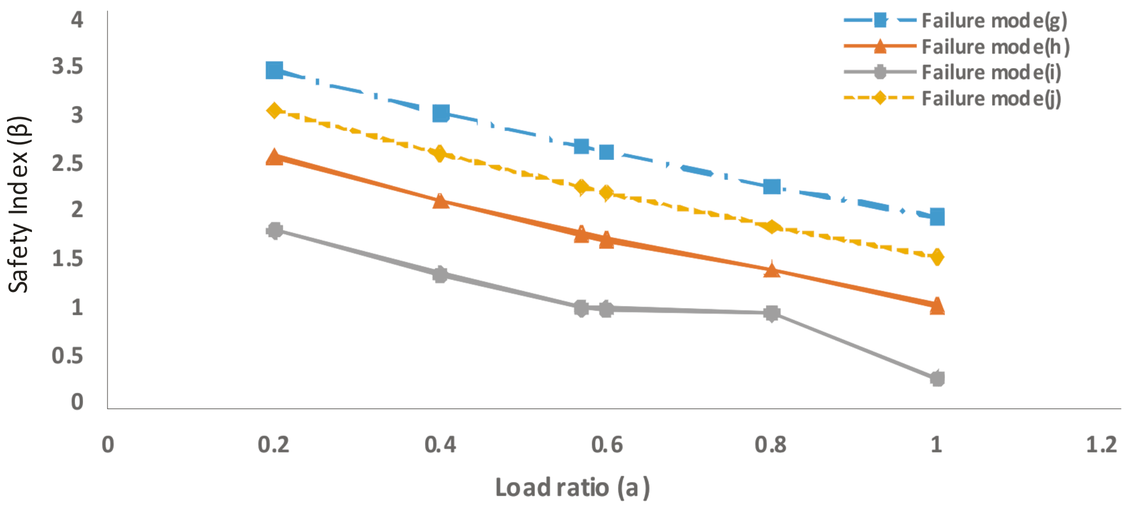

Figure 3: Safety level variations of the eaves joint failure modes

The safety level variation of the Eaves joint (column to rafter connection) failure modes are shown in Figure 3. From the plot a general decrease in the safety index values is observed for all the possible failure modes identified. This trend may be as a result in the reduction of shearing and bearing resisting capacities of the connector bolts as the loading is increasing by the increase in the **α**-value. Failure mode (i) is the most likely mode of failure to initiate failure due to it having the least **β**-values ranging from -0.5 to -1.1 for **α**-values of 0.2 to 1.0 respectively. These results are in agreement with what was observed by Ocholi *et al.* (2010). The negative sign is a strong indication of failure. The failure progression in terms of their computed safety levels is in the order: Failure mode (**i**) – Failure mode (**h**) – Failure mode (**j**) – Failure mode (**g**). The values obtained fall short of even the target serviceability **β** value of 1.5 stipulated by BS EN 1990(2002) for this class of structure.

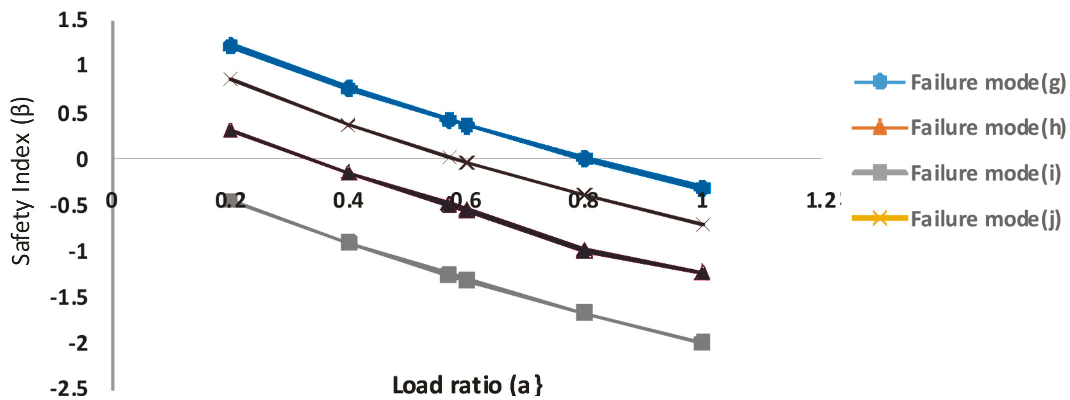

Figure 4: Safety level variations of the apex joint failure modes

N.B. The negative values are an indication of failure.

Figure 4 depicts the safety level variation of the Apex joint (Rafter to Rafter connection) failure modes as the **α**-value is increasing. The observed trend show a general decrease in the **β**-values as the **α**-values increases for all the failure mode associated with the Apex joint identified. This decrease in safety level may be due to a reduction of shearing and bearing resisting capacities of the connector bolts. Failure mode (i) is the most likely mode of failure to initiate failure due to it having the least **β**-values ranging from 1.8 to 0.25. This is similar to what was observed for the Eaves joint.

The sequence of structural failure with respect to the level of safety where high values of **β** are associated with a safe structure and low values of **β** indicate an unsafe structure with high possibility of failure when negative values are observed is in the order: apex connection failure-eaves connection failure -rafter failure-column failure. This implies that failure alternates between members and joints of the portal frame. The sequence of failure at a particular load ratio (say design load ratio $\propto = 0.57$) is presented in Figure 5.

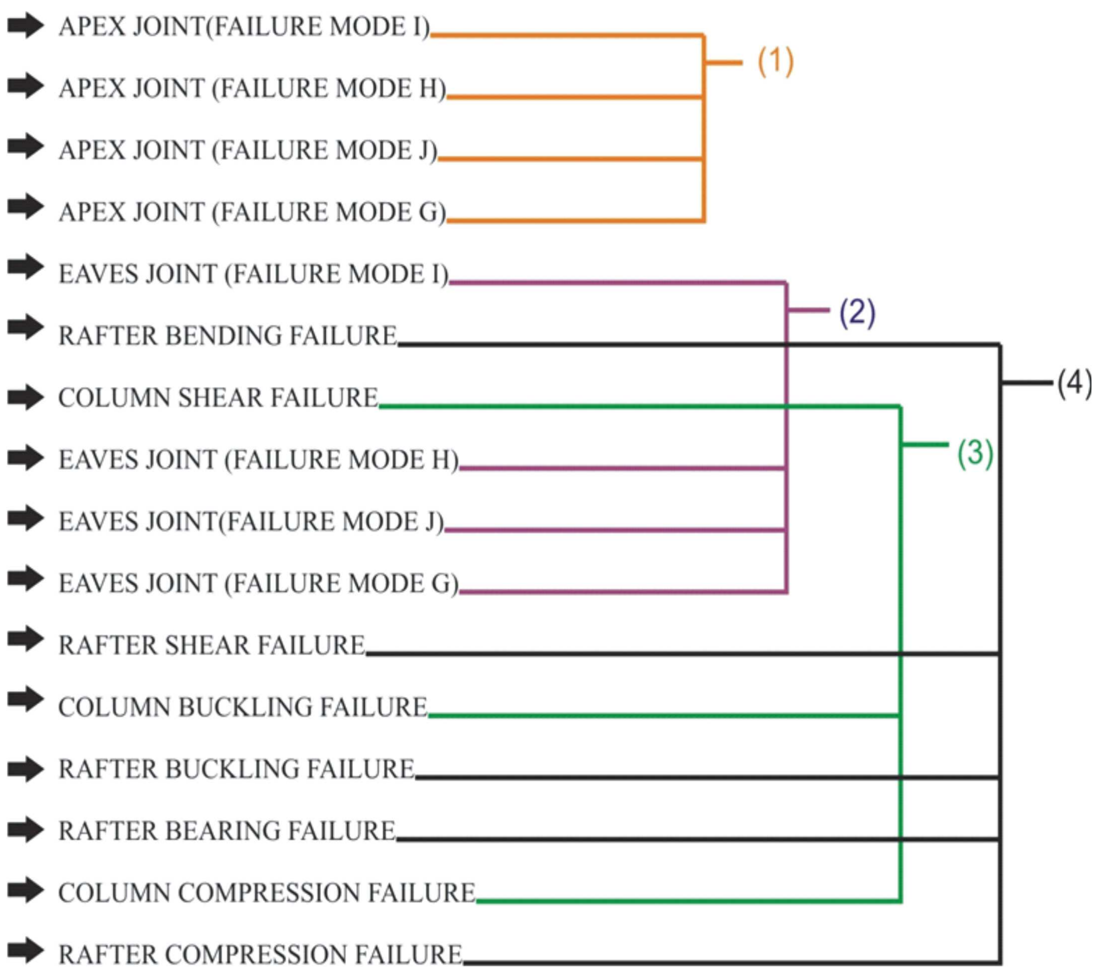

Figure 5: Sequence of failures for the portal frame at a design load ratio$\propto = 0.57$

Figure 5 shows that at the design load ratio, structural failure begins from the Bolted apex connection-Eaves connection-Column-Rafter.

CONCLUSIONS

The safety level assessment of a typical three-hinged timber portal frame was evaluated using FORM5 and the following conclusions were deduced. The column members are most likely to fail due to shear failure. The Rafter members are most likely to fail under bending failure. The Apex and Eaves joints are most likely to fail by Failure mode (i). The sequence of structural failure for a three hinged timber portal frame suggest that the Rafter to Rafter Connection (Apex joint) as the most likely point of failure initiation. It is therefore recommended that when designing a typical three-hinged timber portal frame to Eurocode 5 specification careful attention should be given to the Apex joint as indicated by its high likelihood to failure.

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