EUROCODE 5 - BASED RELIABILITY ANALYSIS OF AXIALLY LOADED SOLID TIMBER COLUMNS USING SELECTED NIGERIAN TIMBER SPECIES

By

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DECLARATION

I declare that the work in this thesis entitled "**Eurocode 5-based Reliability Analysis of Axially Loaded Solid Slender Timber Columns Using Selected Nigerian Timber Species**", has been carried out by me in the Department of Civil Engineering. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree or diploma at this or any other Institution.

_____ _____

Mamman ABUBAKAR Date

CERTIFICATION

This thesis entitled "**EUROCODE 5-BASED RELIABILITY ANALYSIS OF AXIALLY LOADED SOLID SLENDER TIMBER COLUMNS USING SELECTED NIGERIAN TIMBER SPECIES**" by Abubakar MAMMAN meets the regulations governing the award of the Degree of Master of Science in Civil Engineering of the Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This piece of work is dedicated to the indelible memory of my late mother Maimunat Shehu and Father Mamman Murtala Danmusa. May Allah make them a party to his blessings, Amin.

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All Praise be to Allah, the beneficent the merciful , Master of the Day of Judgment Who help me in all my undertakings, who in his infinite Mercy made it possible for me to undertake this research work to its successful completion. May the Peace and blessings of Allah be upon His pure, Noble and exalted Prophet Muhammad (S.A.W), his households and those that follow with good deeds to the day of resurrection..

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SYMBOLS AND NOTATIONS

abs	Absolute value
V_m	Applied Force at joint
M	Applied bending Moment
S	action on member
α_1, α_2	Angles between load and timber fibres
A	Area
f_m	Bending strength (N/mm^2)
$f_{m,12\%}$	Bending strength at 12% moisture content
$f_{m,18\%}$	Bending strength at 18% moisture content
$\sigma_{m,o,d}$	Bending stress parallel to grain
$f_{m,d}$	Characteristic bending strength
$f_{c,0,k}$	Characteristic compression parallel to grain
$f_{c,90,k}$	Characteristic compression perpendicular to grain
ρ_k	Characteristic density

$M_{y,Rk}$	Characteristic fastener yield moment
$f_{v,k}$	Characteristic shear strength
$f_{t,0,k}$	Characteristic tension parallel to grain
$f_{t,90,k}$	Characteristic tension perpendicular to grain
$F_{ax,Rk}$	Characteristic withdrawal capacity of fastener
$\sigma_{c,0,d}$	Compressive stress parallel to grain
$\sigma_{m,crit}$	Critical bending stress
F	cumulative distribution being tested
G_k	Dead load
ρ	Density of the test piece in kg/m^3
h	Depth of bending specimen
$f_{m,d}$	Design bending strength
V	Design load
M_d	Design Moment
$f_{v,gd}$	Design shear strength
a	Distance between loading position and the nearest support (mm)

y	Distance to the centroid of section
l_e	Effective length
l_{es}	Effective length for the standard test procedure (in mm)
l_{et}	Effective length for the test (in mm)
$F_{v,Rdc}$	Embedding strength Characteristic value
$f_{h,1,k} f_{h,2,k}$	Embedment strengths
k_v	Factor for adjusting machine grading
k_s	Factor for adjusting number and size of samples
$E_{0.05}$	Fifth- Percentile of modulus of elasticity
ρ_{05}	fifth-percentile value of density for sample j.
$E_{m,g}$	Global modulus of elasticity in bending
h_1	Height of column
H	Height of portal frame
h_2	Height of rafter
R_{EH}	Horizontal force acting at right fixed end
R_{AH}	Horizontal force at left fixed end

Q_K	Imposed load
$F_2 - F_1$	Increment load in Newton on the regression line with a correlation
$W_2 - W_1$	Increment of deformation in millimetres corresponding to F_2-F_1
K_c, K_z	Instability factor
E_i	i^{th} value of Modulus of elasticity
$K - S$	Kolmogorov smirnov
s	length of rafter
F	Load in Newton
F_m	Load of dowel
$F_{N,C}$	Load of dowel in column of the frame due to normal force
F_{vc}	Load of dowel in column of the frame due to shear force
$F_{N,R}$	Load of dowel in rafter of the frame due to normal force
$F_{v,R}$	Load of dowel in rafter of the frame due to shear force
α	Load ratio
m_2	Mass of the oven dry test slice in gram
M	Mass of the test piece in kg

m_1	Mass of the test slice before drying in grams
γ_g	Material factor of safety for dead load
γ_q	material factor of safety for imposed load
$F_{\max}$	Maximum Load (N)
$\overline{f_{05}}$	Mean (in N/mm^2) of the adjusted 5-percentile values
ρ_{mean}	Mean density (kg/m^3)
$E_{0,mean}$	Mean modulus of elasticity parallel to grain
$E_{90,mean}$	Mean modulus of elasticity perpendicular to grain
G_{mean}	Mean shear modulus
$\overline{E_j}$	Mean value of Modulus of Elasticity for sample j
μ	Measured moisture content
MOE	Modulus of Elasticity
$f_{m,12\%}$	Modulus of elasticity at 12% moisture content
$E_{18\%}$	Modulus of elasticity at 18 moisture content
MOR	Modulus of Rupture
ω	Moisture content of the test slice in percentage

M_b	Moment at B
M_D	Moment at D
I_s	Moment of inertia of column
I_0	Moment of inertia of rafter
N_d	Normal force
n_1	Number of bolts (large circle)
n_2	Number of bolts (small circle)
n	Number of specimens
n_j	Number of specimens in sample j.
r_1	Radius of dowel (small circle)
i_y	Radius of gyration along y
i_z	Radius of gyration along z
r_2	Radius of the dowel(large circle)
$\lambda_{rel,z}$	Relative slenderness about z-z axis
$\lambda_{rel,y}$	Relative slenderness of column about y-y axis
R	Resistance of member
K_r	Rotational Stiffness of the connection between beam and column

L	Span of portal frame
β	safety index
$G_{(x)}$	safety Margin
I	Second moment of area
I_Y	second moment of area about y-y axis
I_Z	Second moment of area about z-z axis
Z	Section Modulus
W	Section Modulus (in cubic millimetres)
$F_{v,dc}$	Shear force in column
$F_{v,dR}$	Shear force in rafter
$\tau_{v,c}$	Shear stress of column
λ_z	slenderness ratio about y-y axis
λ_y	Slenderness ratio about z-z axis
K_{ser}	Slip modulus of fastener or Connector for Ultimate limit state
K_u	Slip modulus of fastener or connector
t_1	Smaller of the thickness of the timber, side member

s	Standard deviation of densities of all specimens
D	Statistic of Kolmogorov smirnov
K_{mod}	Strength modification factor
K_{crit}	Takes account of the effect of axial instability reducing the design compression strength of a member.
K_{cz}, K_{cy}	Takes account of the effect of lateral torsional instability, reducing the design bending strength of a member.
$f_{u,k}$	Tensile strength of each screw
t_2	Thickness of the timber, middle member
$F_{d,c}$	Total load of dowel in the axis of column of the frame
$F_{d,r}$	Total load of dowel in the axis of rafter of the frame
V_{dc}	Vertical design force
R_{AV}	Vertical force acting at left fixed end
R_{EV}	Vertical reaction force acting at right fixed end
V	Volume of the test piece in cubic metre
ρ_w	Wet density of timber
V_w	wet volume of timber species
B	Width of column

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ABSTRACT

In this research work, a reliability analysis was carried on three Nigerian timber specie namely: *Mitragyna ciliata* (Abura), *Afromosia elata* (Afromosia) and *Confusa grandiflora* (Berlinia) after classifying the species to international strength classification system using EN 338 (2003) with the aid of statistical package Easyfit (2010) .This was done after establishing the physical and mechanical properties in the laboratory using ASTM D193 (2000) and EN408 (2003). *Mitragyna ciliata* was assigned to strength Class of C_{18} with minimum characteristic values of density, strength and stiffness properties of 466 kg/m^2, 80.2 N/mm^2 and 9.21 kN/mm^2 respectively as contained in EN 338 (2003). *Afromosia elata* and *Confusa grandiflora* were assigned to strength Class D_{30} with characteristic properties of density, strength and stiffness of 480 kg/m^2, 131.2 N/mm^2 and 11.85 kN/mm^2 for *Afromosia elata* and 462 kg/m^{3,} 89.2N/mm^2, 12.74 kN/mm^2 for *Confusa grandiflora*. The reliability analysis carried on the slender timber column was undertaken to check for six failure modes namely: compression, bending, flexural buckling, lateral torsional buckling, and carrying capacity of dowel connection and shear stress capacity of the joint failure modes. Critical mode of failure from the first four failure modes was found to be lateral torsional buckling mode of failure with the lowest β-value of 2.87 for *Mitragyna ciliata*, 3.4 for *Afromosia elata* and 3.52 for Confusa grandiflora, at a load ratio of 0.8 for compression and bending failure , 0.2 for flexural and lateral torsional buckling failure modes. Critical values of safety index obtained are lower than the code value of 3.8 (BS EN 1990:2002). While, the critical mode of failure for dowel connection was found to be in the carrying capacity of Dowel, with safety value of 1.69 at a load ratio of 0.8, also lower than the code value of 3.8 (BS EN 1990:2002).

\CHAPTER ONE

INTRODUCTION

1.1 PREAMBLE

Timber is an efficient building material, not only in regards to its mechanical properties but also because it is a highly sustainable material considering all phases of the life cycle of timber structures; production, use and decommissioning. Due to the low energy use and the low level of pollution associated with the manufacturing of timber structures, the environmental impact of timber structures is much smaller than for structures built using other building materials. Timber is a light material and compared to its weight, the strength is high; the strength to weight ratio is even higher than for steel. It is a unique structural material, fully renewable, lightweight, and potentially very efficient with many environmentally positive attributes (Steiger, et al 2010; Afolayan, 2005).

However, timber is still not utilised to its full potential in the building and construction sector considering its beneficial properties. Many building owners, but also architects and structural engineers, do not consider timber as a competitive building material compared to concrete, steel or masonry. Attributes such as high performance in regard to reliability, serviceability and durability are generally not associated with timber as a building material. One of the main reasons for this is that timber is a highly complex material; it actually requires a significant amount of expertise to fully appreciate the potential of timber as a structural building material. Structurally it displays significant variability in properties, both within and between members (Tord, 2001). Whilst most of the structural materials in use are processed from finite resources requiring large amounts of energy and producing

significant greenhouse emissions, timber is grown using natural solar energy, on natural soil which is fertilised by its own compost, fuelled by carbon dioxide and watered by rain (Da Vinci, 2008a).

A column is a structural member designed to resist compressive forces. It is a vertical member, usually supporting horizontal beams or slabs or other members. There are several structural members that perform the role of columns. These include struts or props, studs, posts or poles and a number of other members. Porteous and Kermani (2007)

Columns are very important since they can transfer loads down to footings and the failure of just one column can result in major and extensive collapse of part or all of a building. The design and construction of these structural members are therefore very important. Porteous and Kermani (2007).

1.2 STATEMENT OF RESEARCH PROBLEM

NCP 2 (1973), which is the timber design code in Nigeria since 1973, is based on permissible stress design philosophy. The code made reference to the CP 112 (1967). The CP 112 (1967) had passed through several revisions from 1967, to BS 5268 (2002). Eurocode 5(2004) (EC5) is a limit state design code for timber structures which replaced the permissible stress code BS 5268 (2002) in 2010 (Tord, 2001; Timber Research and Development Association, TRADA, 2009). The use of limit state design, instead of permissible stresses, enables differentiation of partial safety factors for permanent and variable loads, and makes it possible to reach a more even nominal safety level in all structures (Alpo, 2004).

Most probabilistic assessments are on steel and concrete structures and components. Although little efforts has been made to perform reliability based analysis and design of timber components.

Timber structures have the advantages of low bulk weight, of dry timber and its comparatively high strength along the grain, its low heat conduction across the grain, easy construction that requires less workmanship, coupled with its abundance especially in Nigeria. Another advantage is light weight that signifies low dead load as compared to concrete and steel, which means low construction cost (Salisu, 2007). According to Sousa et al. (2011), the material properties of a timber element vary both in different parts of the same cross-section as well as along the element itself. As such, timber structures are better analysed using probabilistic models. EC5 (2004) provides a wide range of consistent and up to date models and procedures that can be considered for the development of a local code. The options that are provided for and allowed clearly reveal the manner in which provision should be made for local environmental and regulatory conditions.

EC5 does not contain any data on the material properties of Nigerian timber species. According to standards such as probabilistic model code, JCSS(2006) and EN 408 (2003), the basic material properties of any timber species are determined through laboratory experiments, while other material properties can be determined using the JCSS (2006) and EN 408 (2000). This research work therefore has to come up with data through laboratory experiments that was used to develop the strength classes for the three (3) selected Nigerian timber species based on the recommendations of EN 338 (2009), which are: *Mitragyna ciliata* (Abura), *Afromosia elata* (Afromosia) and *Berlinia/confusa grandiflora* (Berlinia). These timber species are common and available in swamp forest like the *Mitragyna ciliata*

and *Afromosia elata* are restricted to Ogoja, Ondo and Ijebu-ode (Ogun state) high forest, *confusa grandiflora* is available in Benin, Ijebu, Akure and Calabar regions.

Thus, the development of strength classes for these Nigerian timber species gives background for structural timber design in accordance with EC5 (2004). The end result of the research finding was used to perform structural reliability analysis using those species.

1.3 AIM AND OBJECTIVES

1.3.1Aim

The aim of this research work is to undertake a reliability analysis of solid slender timber columns in accordance with EC5 (2004) using the experimental results of selected Nigerian timber species: *Mitragyna ciliata* (Abura*)*, *Afromosia elata* (Afromosia*)* and *Berlinia/Confusa grandiflora* (Berlinia*)* ,based on EN 338(2009)

The objectives of the research work are to:

1. Determine the reference and other material properties of the selected timber species.
2. Determine the characteristic values of both the reference and other material properties of such species in accordance with EN 384 (2004) with the aid of data analysis and simulation application, Easy Fit (2010).
3. Perform timber strength grading in order to assign the appropriate strength class to each species in accordance with EN 338 (2009).
4. Determine the limit state function of slender columns based on the requirements of EC 5(2004).
5. Perform reliability analysis on the columns using First Order Reliability Method.

1.4 SCOPE AND LIMITATION OF THE RESEARCH

This research work considered a reliability analysis using EC 5 (2004) of an axially loaded, fixed end solid slender timber column as part of three hinged portal frame transmitting a uniformly distributed loading. The modes of failure from the three classified selected Nigerian timber species were based on EN 338 (2009) recommendations. The basic reference material properties were determined in accordance with EN 408 (2003) while other material properties (derived properties) of those species were obtained based on the empirical relationship as prescribed by EN 408 (2003) and JCSS (2006). These species were classified based on EN 338 (2009) recommendations.

The research work was therefore limited to carrying out a reliability based analysis of the classified timber species considering an axially loaded solid slender timber column as part of three hinged portal frame using EC 5(2004).

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

In Nigeria, majority of timber applications are on roof fabrications as well as in formwork construction in both building and Civil Engineering works. Framed structures in timber are not common unless in some cases for temporary structures such as those at construction sites for offices/stores as well as for scaffolding. However, it is used for other permanent works like for railway sleepers, electric poles etc. It is also rarely used for staircases and footbridges constructions (Porteous and Kermani,2007).

2.2: MATERIAL PROPERTIES OF TIMBER

According to (EN 408, (2003) ; Fuwape, (2000); JCSS, (2006); Abubakar and Mohammed, (2007)) the material properties of any timber specie are the stress and stiffness related properties of the standard test specimen under given (standard) loading and climate conditions and the timber density. However, JCSS (2006) categorised the material properties of timber into two: reference and other properties.

The reference material properties are also known as the basic or grade determining properties which are determined from standard laboratory tests: the bending strength, bending modulus of elasticity in bending and timber density. Other material properties of timber on the other hand, are derivatives of the reference properties and include: tension and compression strengths parallel and perpendicular to grains, shear strength and shear modulus.

EN 384 (2004) has clarified that for the purpose of assigning grades to the strength classes in EN 338 (2003) only three characteristic values, i.e. bending strength, mean modulus of elasticity parallel to grain and timber density need to be determined, other properties may be taken from Table 1 of EN 338 (2009).

2.2.1: Physical properties of timber

Physical properties are the quantitative characteristics of wood and its behaviour to external influences other than applied forces. This include: directional properties, moisture content, dimensional stability, thermal properties, density, electrical, chemical, and decay resistance. Familiarity with physical properties is important because they can significantly influence the performance and strength of wood used in structural applications (Winandy, 1994).

(a) Density and Specific Gravity of Timber: The prediction of strength on the basis of density alone is not always satisfactory (Porteous and Kermani, 2007). Winandy (1994) referred to timber density as its mass per unit volume at some specified condition. The density of a hydroscopic material like wood depends on two factors: the weight of the wood structure and moisture retained in it. Wood density at various moisture contents can vary significantly and must be given relative to a specific condition.

Specific gravity on the other hand, is generally expressed as the ratio of the oven-dry weight of the timber to the weight of an equal volume of water (Winandy, 1994; Porteous and Kermani, (2007). However, Porteous and Kermani (2007) have pointed out that because water volume varies with the moisture content, the specific gravity of timber is

normally expressed at certain moisture content. Hence, basic oven-dry specific gravity of commercial timber ranges from 0.29 to 0.81, most falling between 0.35 and 0.60.

(b) Moisture Content of Timber: Winandy (1994) referred to the moisture content of wood as the weight of water in it, expressed as a percentage of oven dry weight.

Wood is a hydroscopic material that absorbs moisture in a humid environment and loses moisture in a dry environment. As a result, the moisture content of wood is a function of atmospheric conditions and depends on the relative humidity and temperature of the surrounding air. Under constant conditions of temperature and humidity, wood reaches equilibrium moisture content (EMC) at which it is neither gaining nor losing moisture. The EMC represents a balance point where the wood is in equilibrium with its environment.

The time required for wood to reach the EMC depends on the size and permeability of the member, the temperature, and the difference between the moisture content of the member and the EMC potential of that environment. Pourteous and Kermani (2007).

Moisture content has a direct effect on the strength and stiffness, swelling or shrinkage of timber. A proper understanding of the physical characteristics of timber enables the building of safe and durable timber structures. The process of drying (seasoning) timber should ideally remove over a third of the moisture from the cell walls. Timber at this stage is referred to as seasoned with a moisture content of between 12 and 25% (depending on the method and duration of drying, i.e. air, kiln, solar, microwave, etc.).Wood changes dimensionally with change in moisture below its fibre saturation point: it shrinks when it loses moisture and swells as it gains moisture. These dimensional changes are mostly in the direction of the annual growth rings (tangentially), with about half as much across the rings

(radially) and as such mainly affect cross-sectional dimensions (perpendicular to the grain) and can result in warping, checking or splitting of wood. Longitudinal shrinkage of wood (shrinkage parallel to the grain) for most species is generally very small. TRADA (2002)

2.2.2 Mechanical Properties of Timber: Mechanical properties are the characteristics of a material in response to externally applied forces. They include elastic properties, which characterize resistance to deformation and distortion, and strength properties, which characterize resistance to applied loads. Mechanical property values are given in terms of stress (force per unit area) and strain (deformation resulting from the applied stress). The mechanical property values of wood are obtained from laboratory tests of lumber of straight-grained clear wood samples (without natural defects that would reduce strength, such as knots, checks, splits, etc.).

(a) Compression : Although wood is an anisotropic material, when used for structural purposes it is assumed to be orthotropic. This means that it is presumed to have directional properties in three mutually perpendicular (orthogonal) axes. The orthogonal axes are aligned with the grain direction (L), the radial direction (R) and the tangential direction (T) as shown in Fig. (2.1)

The properties of the wood along the direction of the L axis are referred to as properties parallel to grain, and because of the direction of its cellular nature, the properties associated with the axis are considerably stronger and stiffer than the equivalent properties associated with the R and T axes.

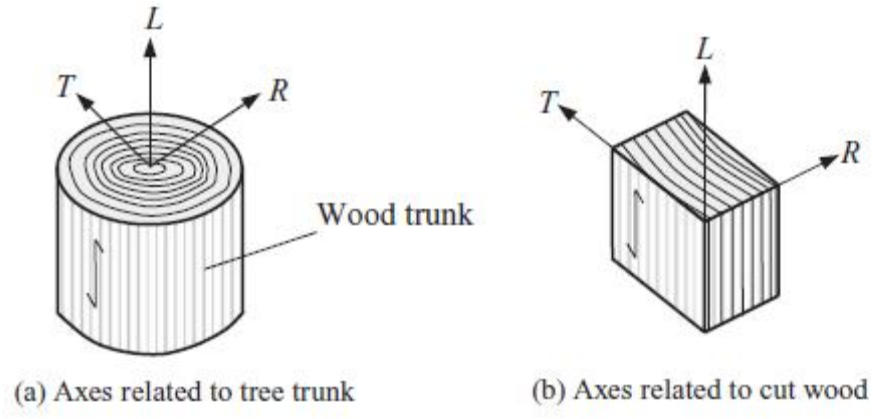


Fig 2.1: Wood as Anisotropic Material

When a compression load is applied perpendicular to grain, it produces stress that deforms the wood cells perpendicular to their length. Once the hollow cell cavities are collapsed, wood is quite strong because no void space exists. In practice, compressive strength of wood perpendicular to grain is usually assumed to be exceeded when deformation exceeds 4% of the proportional limit stress. Compression applied at an angle to the grain produces stresses that act both parallel and perpendicular to grain. The strength at any intermediate angle is intermediate to values of compression parallel and perpendicular to grain and is determined using Hankinson's formula (Porteous and Kermani ,2007).

(b) Tension : Parallel to its grain, wood is very strong in tension. Failure occurs by a complex combination of two modes: cell-to-cell slippage and cell wall failure. Slippage occurs where two adjacent cells slide past one another. Cell wall failure involves rupture within the cell wall with little or no visible deformation prior to complete failure. Tensile strength parallel to grain for clear wood has been historically difficult to obtain; it is often conservatively estimated from bending test values because clear wood normally exhibits initial failure on the face stressed in tension.

In contrast to tension parallel to grain, wood is relatively weak when loaded in tension perpendicular to grain. Stresses in this direction act perpendicular to the cell lengths and produce splitting or cleavage along the grain, which can have a significant effect on structural integrity. Deformations are usually low prior to failure because of the geometry and structure of the cell wall cross-section. Strength in tension perpendicular to grain for clear green samples of Douglas-fir and loblolly pine average 2.1 and 1.8 MPa, respectively. However, because of the excessive variability associated with ultimate stress in tension perpendicular to grain design situations that induce this stress should be avoided.

(c) Bending Strength of Timber: Bending (Flexural) properties are critical and are stresses that were induced when a material is used as a beam, such as in a floor or rafter system. Winandy, (1994) and Mohammad et al, (2011) testified that bending tests are carried out on an isolated simply supported beam according to procedure set forth in MS 544 Part 3(2001) and BS EN 408 (2003). It is determined by performing "**Four Point**" and "**Three Point Bending**" tests.

According to Piazza and Riggio (2008) the strength tests are arranged according to standardised methods (EN 384, 2004; EN 408, 2003) test piece are fist allowed to attain the equilibrium moisture content (EMC) in the standard conditions. (i.e. temperature $T = 20^0 C$ and relative humidity $RH = 65\%$) before testing. At this condition, the EMC for most softwood is about 12%. The Italian standard, UNI 11035(2003) established reference moisture content equal to 20%.

(d) Modulus of Elasticity of Timber: EN 408 states that, the MOE shall be determined by conducting test on a piece having a minimum length of 19 times the depth of the section, and where this aid is not possible, the span of the beam shall be reported.

2.3. Strength Grading System of Timber

Steiger et al (2010) has pointed out that, in Europe, the classification of structural timber is carried out according to a set of three linked standards:

1. Test methods to determine mechanical properties as well as dimensions, moisture content (MC) and wood density of test pieces are specified in the EN 408 (2003)
2. Characteristic values of mechanical properties and density are derived from the test data according to the EN 384 (2004).
3. The timber is finally classified in strength grades according to the EN 338 (2009). This standard defines twelve strength classes for softwoods. A specific sample can be assigned to a certain strength class, if the characteristic values of density and bending strength (both of them are 5th-percentiles) as well as MOE (mean value, usually derived from bending tests) match or exceed the values of the desired class. Additional mechanical properties needed for the design of timber structures are derived from these basic values by empirical relationships.

2.4. DETERMINISTIC AND PROBABILISTIC DESIGN APPROACHES

Deterministic design approach includes the factor of safety method, the load factor method, and the partial factor method (limit state method). The Eurocode 5(2004) employs the limit state approach, in which the material strength are divided by factors of safety and the load

action is multiplied by further factor of safety. This is with the aim of achieving low probability of failure (Melchers, 1999).

Design of engineering involves the consideration of many dependent and independent variable, of which values are associated with varying degree of uncertainties, for example, both the dead and the live loads on structures cannot be estimated with certainty. However, the uncertainties associated with the live load are higher than that of dead load. This is because of high variability associated with the former, even though, this can change. For timber structure owing to the fact that timber is a live material, it is isotropic material. Its properties therefore change with change in environmental condition and load duration. The properties not only vary from specie to specie, but even with a particular specie (Blass et al,1995).Generally, the uncertainties associated with the variable that define resistance and loading in structures invalidates the current deterministic approach.

The Eurocode 5 (2004) uses partial probabilistic approach, being a limit state design code (Stehn and Johnson, 2002). It recognized that there is likelihood, in other words finite probability that a particular limit state may be exceeded. However, there was no attempt to compute such probability (Afolayan, J.O 1992). This therefore signifies the need for the computation of the probability of failure of a structure designed based on Eurocode 5 (2004) requirements. This can be achieved through reliability based analysis (that is fully probabilistic design approach). In probabilistic assessment any uncertainty about a variable is taken in to account (Melchers, 1999).

Previous works carried on this aspect include; (David, 2005) "Structural Reliability Analysis of an Electric Transmission Tower" which used BS 8110 and considers three modes of failure: Tensile, compressive, and joint failure; (Christopher, 2005) "Reliability Analysis of Timber Beams in combined shear and Torsion" which considered Timber beams using EC 5, others include (Salisu,2007) "Reliability based Analysis and Design of Mechanically jointed Timber Beam under Gravity Loads .

2.4.1 Behaviour of columns

Under a concentric load—that is a load that is applied exactly through the centroid of the cross-section of a column—only compressive stresses are generated. A column will experience a very small shortening in length due to this compression.

2.4.2 Eccentricity of loading

A perfectly formed column with a load applied concentrically through the centroid could be made very slender and infinitely long. In practice, however, loads are never perfectly concentrically applied and in addition, columns are never perfectly straight. This results in loads being applied *eccentrically* or not precisely through the centroid of the cross-section. Columns may also be designed to support loads that are intentionally eccentric. Eccentricity of loading leads to bending.

2.4.3 Stability of Slender Columns

When a slender column is loaded axially, there exists a tendency for it to deflect sideways (see Figure 2.2). This type of instability is called flexural buckling. The strength of slender members depends not only on the strength of the material but also on the stiffness, in the

case of timber columns mainly on the bending stiffness. Therefore, apart from the compression and bending strength, the modulus of elasticity is an important material property influencing the load-bearing capacity of slender columns. The additional bending stresses caused by lateral deflections must be taken into account in a stability design.

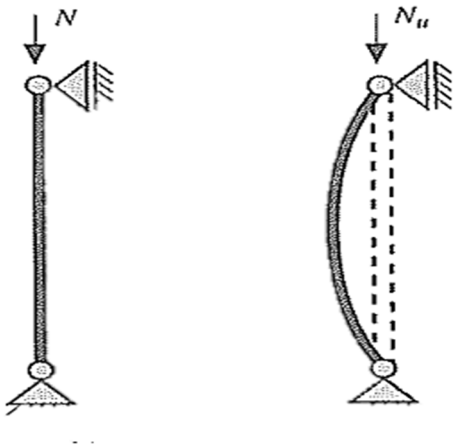


Fig. 2.2: Two hinged columns

If a (geometric) nonlinear analysis, incorporating geometric imperfections (shape errors), is the basis for the design, the additional bending stresses are determined explicitly. In the case of linear static analyses, the higher order effects are accounted for approximately through modification (k) factors. Whenever a slender column is loaded there is a tendency for it to fail by buckling in a more flexible plane by deflecting sideways .

2.5 DESIGN CRITERIA OF TIMBER COLUMN USING EUROCODE 5 (2004)

The Eurocode 5(2004) design criterion for timber column is based on the possible failures of the column as shown below:

2.5.1 Compression failure

The design compression resistance, to resist the compression failure is given by the Eurocode 5 (2004) as:

$$f_{c,o,d} = \left(\frac{k_{\text{mod}} f_{c,o,k}}{\gamma}\right) \tag{2.1}$$

Where k_{mod} is a modification factor to take care of variation in density and moisture content, $f_{c,o,k}$ is the characteristic compressive strength parallel to grain, γ_m is the timber material partial safety factor for strength.

2.5.2 Bending Failure

The design bending resistance provided by EC 5 (2004) against bending failure is given by:

$$f_{m,d} = \left(\frac{k_{\text{mod}} f_{m,d}}{\gamma_m}\right) \tag{2.2}$$

$f_{m,d}$ is the characteristic bending strength

2.5.3 Flexural Buckling failure

The design buckling resistance of a timber member according to EC 5 (2004) is given by

$$\frac{\sigma_{md}}{f_{md}} + \frac{(\sigma_{c,o,d})}{(k_c f_{c,od})} \leq 1 \tag{2.3}$$

Where;

$$k_c = \frac{1}{(k + \sqrt{(k^2 - \lambda_{rel}{}^2)})} \tag{2.4}$$

$$k = 0.5[1 + \beta_c(\lambda_{rel} - 0.3) + \lambda^2{}_{rel}] \tag{2.5}$$

$$\lambda_{rel} = \sqrt{\frac{(f_{c,o,k})}{(\sigma_{c,crit})}} \tag{2.6}$$

$$\sigma_{c,rit} = \pi^2 \frac{E_{0.05}}{\lambda^2} \tag{2.7}$$

Where β_c is 0.2 for solid timber, β_c is 0.1 for glue laminated timber, $\sigma_{c,o,d}$ is design compressive stress, $f_{c,o,d}$ is design compressive strength, σ_{md} is design bending stress, f_{md} is design bending strength, λ_{rel} is slenderness ratio, $E_{0.05}$, is the fifth percentile of the modulus of elasticity parallel to grain, β_c is the factor for members within the straightness limits and k_c is a factor which takes into account lateral instability, λ= bending about either y or x axis.

2.5.4 Lateral Torsional Buckling

The lateral torsional buckling is expressed in EC5 (2004) as:

$$\left(\frac{\sigma_{m,y}}{k_{crit} f_{m,d}}\right)^2 + \frac{\sigma_c}{k_c f_{c,od}} \leq 1 \quad (2.8)$$

where ;

$$k_c = \frac{1}{k + \sqrt{k_y^2 - \lambda_{rel}^2}} \quad (2.9)$$

where k_{crit} is a factor which takes into account the reduced bending strength due to lateral bucking and k is given by Eqn. (2.5)

2.5.5 Carrying capacity of Dowel

$$f_{dc} \leq 2 f_{v,Rdc} \quad (2.10)$$

where

$$f_{dc} = \sqrt{(fm + f_{v,c})^2 + f_{N,C}^2} \quad (2.11)$$

and $$f_{v,Rdc} = \frac{K \bmod, F_{v,RK}}{\gamma_m} \quad (2.12)$$

2.5.6 Shear Capacity Failure

$$\tau_{V,C} = F_{V,gd} \quad (2.13)$$

where $$\tau_{V,C} = \frac{3}{2}\frac{F_{v,dv}}{2b.h} \quad (2.14)$$

and $$f_{v,gd} = \frac{K \bmod, F_{v,k}}{\gamma_M} \quad (2.15)$$

Therefore, $$\frac{3}{2}\frac{F_{v,dv}}{2b.h} \leq \frac{K \bmod, F_{v,k}}{\gamma_M} \quad (2.16)$$

2.6 LIMIT STATE DESIGN

When a structure is loaded, the structure will respond in a manner which depends on the type and magnitude of the load, strength and stiffness of the structure. Whether or not this response is considered satisfactory depends on the requirements which must be satisfied. Such requirements might include safety of the structure against collapse, limitations on damage or on deflections, or any range of other criteria. Each of these requirements may be termed "limit state". The violation of which can then be defined as the attainment of an undesirable condition for the structure, and when the violation of a limit state occurs, the consequences may be extreme. (Melchers, 1999)

Nowak and Collins (2000) defined a limit state as a boundary between desired and undesired performance of a structure.

Mosley et al. (2007) stated that a design is to achieve acceptable probability that a structure will not become unfit for its intended use – that is, it will not reach a limit state.

The two principal types of limit states identified by Mosley et al. (2007) are the ultimate limit state and the serviceability limit state.

2.6.1 Ultimate limit state

This requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed to ensure the safety of the building occupants and/or the safety of the structure itself. Examples include; tipping or sliding, rupture, progressive collapse, plastic mechanism, instability, corrosion, fatigue, deterioration, fire (Melchers, 1999)

2.6.2 Serviceability limit state

Generally, the most important serviceability limit states are:

i. Deflection – the appearance or efficiency of any part of the structure must not be adversely affected by deflections nor should the comfort of the building users be adversely affected.
ii. Cracking – local damage due to cracking and spalling must not affect the appearance, efficiency or durability of the structure.
iii. Durability – this must be considered in terms of the proposed life of the structure and its conditions of exposure.

Methods of reliability analysis depend on the ability to calculate probability of failure.

2.7: RELIABILITY ANALYSIS

Melchers (1999) defines reliability as the systematic calculation and prediction of the probability of limit state violation. Togan & Daloglu (2005) pointed out that structural reliability theory has been examined in a large number of research studies, and now the reliability of a structure could be defined as the ability that the structure fulfils its design purpose for some specified reference period. If the reliability investigation is that of analysis, the method is to find out whether all points fall within the failure surface. However if the problem is one of design, the design parameters should be assigned such that the safe domain contains all the points.

Melchers et al. (1999) observed that structural failure might be considered to be the occurrence of one or more types of undesirable structural responses including the violation of predefined limit states. Thus collapse of all or part of a structure, major cracking and excessive deflection are some possible forms of failure. Fortunately, real structures only rarely fail in a serious manner, but when they do, it is often due to causes not directly related to the predicted nominal loading or strength probability distribution considered. Other factors such as human errors, negligence, poor workmanship or neglected loadings are most often involved (Melchers et al., 1999).

Probabilistic structural analysis is the art of formulating a mathematical model within which one can ask and get answer to questions such as:

- What is the probability that a structure behaves in a specified way when given that one or more of its material properties or geometric dimensions and properties are of a random or incompletely known nature, and/or that the actions on the structure in some respects have random or incompletely known properties?
- How is a structure behaving when its material properties, geometric properties and actions all are uniquely given?
- What dimensions should be assigned to the structure in order that it has optimal properties in a given well-defined sense within the possibilities of the probabilistic model?
- What value should the probability of occurrence of a specified behaviour be in order that the structure in a well-defined sense is optimally designed with respect to this behaviour?

- How much is presently known about the actual safety levels of slender timber columns.
- If anything is known about current safety levels, are these safety levels acceptable?

Reliability studies are basically analytical in approach. The major sources of information for this particular study are secondary and they include text books, research projects, journals, etc. FORM5 (a computer program written in FORTRAN language) will be used to compute the safety indices.

The procedure for this research is summarized below:

1. Determine the limit state function of timber as implied in EC 5(2004).
2. Determine the existing safety values embedded in the EC 5(2004).
3. Conduct a Reliability Analysis on the Limit State Equations using FORM 5.
4. Compare and suggest a new target reliability index, (β).

2.7.2 Reliability Concept

The concept of probability of failure is best described by considering the specific example of supply (X_1) versus demand (X_2). Failure for this problem is defined when demand is greater than or equal to supply, or when $X_1 - X_2 \leq 0$. In a reliability analysis, the variables X_1 and X_2 are treated as random variables with probability density functions. If the supply and demand is statistically independent, then the expression for the probability of failure is (Ang and Tang, 1984):

$$p_f = \int_{-\alpha}^{+\alpha} \left[1 - F_{\chi_2}(x_1)\right] F_{\chi_1}(x_1) dx \tag{2.10}$$

Where F_{x2} (x_1) is the cumulative distribution of X_2 at x_1 and $F_{\chi 1}$ (x_1) is the probability density of X_1 at x_1.

The supply and demand problem may be reformulated in terms of the safety margin (Ang & Tang, 1984).

$$G(X) = X_1 - X_2 \tag{2.11}$$

Where X_1 and X_2 are independent normally distributed random variables.

Failure is defined when

$$G(X) = X_1 - X_2 \leq 0 \tag{2.12}$$

Since the safety margin G(X) is a function of two independent normally distributed random variables (Ang and Tang 1984), G(X) itself is a normally distributed random variable with probability density function, f_G (g). The mean value of G(X) is given by (Ang and Tang, 1984):

$$\mu_G = \mu_{X_1} - \mu_{X_2} \tag{2.13}$$

and the standard deviation of G(X) is

$$\sigma_G = \sqrt{\sigma_{X_1}^2 + \sigma_{X_2}^2} \tag{2.14}$$

The probability of failure is equal to

$$p_f = \int_{-\alpha}^{0} f_G(g) dg = F_G(\beta) \tag{2.15}$$

If β is the number of standard deviations σ_G from the mean value μ_G to the failure region, then failure occurs either when (Ang and Tang, 1984).

$$\mu_G - \beta\sigma_G = 0 \tag{2.16}$$

or when

$$\beta = \frac{\mu_G}{\sigma_G} = \frac{\mu_{X_1} - \mu_{X_2}}{\sqrt{\sigma_{X_1}^2 + \sigma_{X_2}^2}} \tag{2.17}$$

Substituting Eqn. 2.17 into Eqn. 2.15, the probability of failure becomes (Ang and Tang, 1984):

$$p_f = F_G\left(\frac{\mu_G}{\sigma_G}\right) = 1 - \Phi(\beta) \tag{2.18}$$

Where $\Phi(\beta)$ is the standard normal cumulative density function evaluated at β. The quantity β is referred to as the reliability or safety index.

CHAPTER THREE

MATERIALS AND METHODS

3.1 INTRODUCTION

The classification of the three structural timbers was done according to three linked standard procedures as follows:

1. Determination of mechanical properties as well as dimensions, moisture content (MC) and wood density of test specimens as specified by EN 408 (2003).
2. Determination of characteristic values of mechanical properties and density from the data generated in step 1 above in accordance with EN 384 (2004) with the aid of a Statistical Software Package, EasyFit (2010).
3. Classification of timber in strength grades by assigning the appropriate class to those species based on the recommendations of EN 338 (2009).

3.2 MATERIALS

The timber species used for this research work are as follows:

1. *Mitragyna ciliata* (Abura)
2. *Afromosia elata* (Afromosia)
3. *Confusa grandiflora* (Berlinia)

3.2.1 Preparation of Specimens

Eighteen (18) timber planks, 6 planks for each of the three timber species were processed. The dimension of each plank was 50 mm×75 mm×3600 mm. The planks were prepared to obtain a plane surface as well as sharp edges for cross cutting. This was done with the aid

of milling machine and subsequently cross cutting followed with Sawing machine. A total of 60 timber beams, 20 pieces were used for each of the selected species of 50 mm×75 mm×1500 mm were prepared. Sections for moisture content and density determination cut from the test planks were provided. All the specimens were transported to the Heavy Concrete and Materials Laboratory of the Department of Civil Engineering Ahmadu Bello University, Zaria. Forty five (45) slices of full cross section, 15 slices for each species cut from the sections provided for moisture content and density determination were prepared in the Carpentry Workshop of Civil Engineering Department, Ahmadu Bello University, Zaria.

3.3 METHODS

3.3.1 Determination of physical properties of timber

The physical properties determined experimentally include timber density and its related property, i.e. moisture content. They were determined in accordance with the provision of EN 408 (2003) and EN 13183- 1 (2002) respectively.

(a) **Determination of moisture content of test Slices :**Slices of full cross-section cut from the test timber for each of the above mentioned species of 50 mm×75 mm×50 mm free of knots and any other visible defect were provided for moisture content determination as prescribed by standard. The moisture content of each slice was determined by first measuring the initial mass of the slice before drying using AVERY weighing balance (AWB). The test slices were then placed in the oven and the final mass of each slice after oven drying was recorded. The tested slices were dried at a temperature of 105^0C

for 24 hours. The two readings (initial and oven dry) masses were recorded for each slice and the moisture contents were then computed from the relation:

$$\omega = \frac{m_1 - m_2}{m_2} \times 100\% \tag{3.1}$$

Where: m_1 is Mass of the test slice before drying in grams, m_2 is Mass of the oven dried test slice in grams and ω is Moisture content of the test slice in percentage

(b) Determination of density of test slices : Slices for MC determination are those for density calculations. The wet densities of the test slices were determined from the full cross section of the test specimens that are free of any defect. This was done by first measuring the dimensions of cross sections of the test slices before oven drying as well as their masses and then reweighed after oven drying. The densities of the test slices were therefore computed as the ratio of their masses to that of the cross sectional volume.

$$\rho = \frac{M}{V} \tag{3.2}$$

Where M is mass of the test piece in kg, V is Volume of the test piece in cubic metre and ρ is density of the test piece in kg/m^3 .

The wet densities are computed from the respective equations as follows:

$$\rho_w = \frac{m_1}{V_w} \tag{3.3}$$

Where ρ_w is the wet density, V_w wet volume, m_1 and m_2 are as defined above.

(c) Characteristic value of density : The 5-percentile value of density (ρ_{05}) was calculated from the equation:

$$\rho_{05} = \left(\bar{\rho} - 1.65s \right) \ kg/m^3 \tag{3.4}$$

Where: $\bar{\rho}$ is the mean density (in kg/m^3), s is the standard deviation of densities of all specimens

The characteristic density (ρ_k) was calculated from the equation:

$$\rho_k = \frac{\sum \rho_{05} n_j}{\sum n_j} \tag{3.5}$$

Where: n_j is the number of specimens in sample j; ρ_{05} is the 5-percentile value of density for sample j.

The 5th -Percentile value of the density also has to be corrected to 12% Moisture content, the result will be the corrected characteristic values of density to 12% moisture content. This is according to MC adjusted density (ku[illegible]);

$$\rho_{k,12} = \rho_w * \left(1 - \frac{(1-k)*(w-12)}{100} \right) \tag{3.6}$$

ρ_w is density at the MC during the bending test in kg/m^3, k is 0.5 (constant for spruce), w is measured MC, %

In view of the fact that NCP2 (1973) has the strength and stiffness properties based on 18 % MC, values obtained have to be adjusted to 18% MC to meet up with the Nigerian standard.

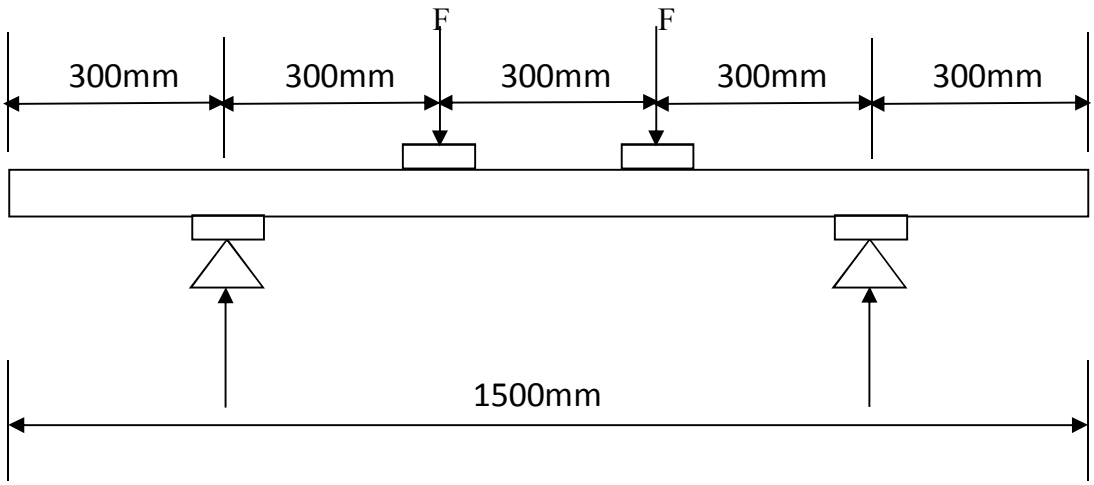
The 18% MC of the density can be obtained from the relation below obtained by analogy from equation 3.6;

$$\rho_{k,18} = \rho_w * \left(1 - \frac{(1-k)*(w-18)}{100}\right) \tag{3.7}$$

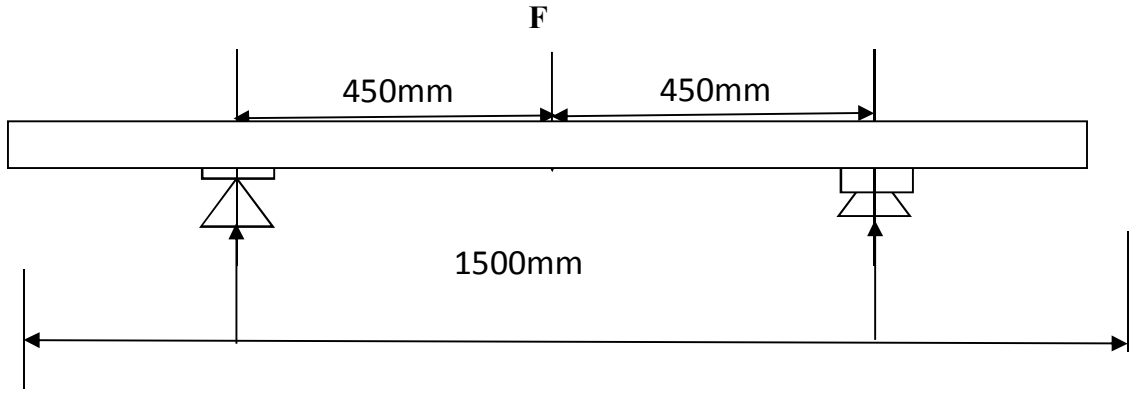
3.3.2 Determination of Mechanical Properties of timber

Forty (40) pieces of 50 mm×75 mm×1500 mm for each of the above mentioned timber species were tested using DENISON Universal Testing Machine (DUTM) to determine their mechanical properties (bending strengths and modulus of elasticity) based on EN 408 (2003).

3.3.2.1 Determination of bending strength of test piece : The bending strengths of the test pieces were determined according to EN 408 (2003) on a specimen having a length of 1500 mm with a cross sectional area of 50 mm×75 mm which agreed with the code requirement that the test piece shall have a minimum length of 19 times the depth of the section. Twenty (20) of the test pieces were symmetrically loaded at two points over a span of 18 times the depth of the section (i.e. 1350mm) using UTM and the other twenty (20) pieces for each specie were loaded at the centre of the beam. Small steel plates were inserted between the test piece as well as the supports before loading in order to minimise the local indentation. The experimental set up for four and three point bending tests is as shown in Fig 3.1



(a) Experimental Set up for Four- Point Bending Test



(b) Experimental Set up for Three -Point Bending Test

Fig. 3.1: Experimental set up for Bending Tests (EN 408 (2003))



Plate 1: Timber Speicies samples in the Laboratory for Bending tests.



Plate 2: Bending Test Experimental setup



Plate 3: Three- Point Bending Test



Plate 4: Four- Point Bending Test

The summaries of four and three Points bending tests are given in Tables A1 to A6 in Appendix A.

The beam deflections for constant load rates of 2000N intervals were considered for all the timber species. The values were computed by multiplying the dial gauge readings by 0.01 to get the actual beam deflection. This is because, the dial gauge calibration used was 0.01

The bending strength was calculated from the equation:

$$f_m = \frac{aF_{\max}}{2W} \tag{3.8}$$

Where

a is the distance between loading position and the nearest support , (in millimetres), $F_{\max}$ is the maximum load (in Newton), $W = \frac{bh^2}{6}$, is the section modulus (in cubic millimetres), f_m is the bending strength (N/mm^2) , b is the width of the beam, h is the depth of the section.

This is for a four- point test. ASTMD193- 94 (2000) provided another formula for computing the bending strength for three- point tests as;

$$f_m = \frac{3F_{\max}.L}{2bh^2} \tag{3.9}$$

L= length between two points of the machine grip.

5th - percentiles of bending and tensile strength shall be adjusted to 150mm depth or width by dividing by:

$$k_h = \left(\frac{150}{h}\right)^{0.2} \tag{3.10}$$

Since the bending test arrangement was not as in EN 408 (2003) (i.e. span, l = 18h and distance between inner load points, a_f = 6h) then the 5 – percentile shall be adjusted by dividing by:

$$k_l = \left(\frac{l_{es}}{l_{et}} \right)^{0.2} \tag{3.11}$$

$l_{es} is$ the effective length for standard test ($l_{es} = 18\text{h}$), l_{et} = effective length for this particular test ($l_{et} = 900\text{mm}$)

After computing the respective bending strength of the three timber species, the fifth percentile of each species is computed using easy-fit statistical software (EasyFit (2010)).

3.3.2.2: Characteristic value of bending strength: From the 5-percentile values (f_{05}) of bending strength computed, the characteristic values of bending strength f_k were calculated from the relation:

$$f_k = \bar{f}_{05} * 1.12 \tag{3.12}$$

Where: $\bar{f}_{05}$ is the mean (in N/mm^2) of the adjusted 5-percentile values of bending strength (Ranta-Maunus et al, 2001).

The computed characteristic bending strength has to be corrected to 12% moisture content (MC) and by analogy to 18% MC (since NCP2 1973 was based on 18% MC) (BostrÖm, 1994; Heikkila and Herajarvi, 2008) respectively.

Using the Eqn. (3.13) as follows:

$$f_{m,12\%} = \frac{f_{m,measured}}{1 + 0.0295 * (12 - \mu)} \quad \text{It follows} \tag{3.13}$$

$$f_{m,18\%} = \frac{f_{m,measured}}{1 + 0.0295 * (18 - \mu)} \tag{3.14}$$

3.3.3: Determination of global modulus of elasticity in bending: The modulus of elasticity test was derived from the bending strength test by using a dial gauge beneath the timber beam to obtain the central deflection of the beam. The load was applied at constant rate. The central deflection as well the load causing it was recorded and tabulated. The global modulus of elasticity was then computed from the expression:

$$E_{m,g} = \frac{l^3 (F_2 - F_1)}{b h^3 (w_2 - w_1)} \left[\left(\frac{3a}{4l} \right) - \left(\frac{a}{l} \right)^3 \right] \tag{3.15}$$

Where; F_2-F_1 is an increment load in Newton on the regression line with a correlation coefficient of 0.99 w_2-w_1 is the increment of deformation in millimetres corresponding to F_2-F_1 as shown in Fig. 3.2.

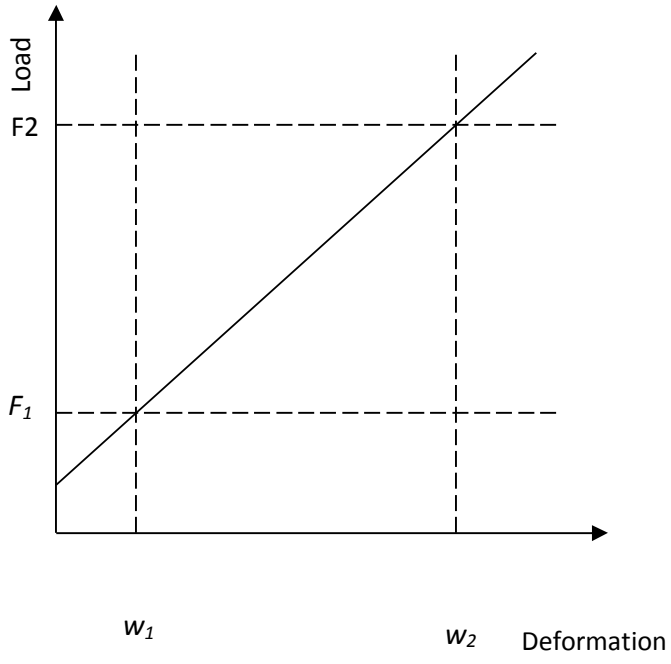


Fig. 3.2: Load-deformation within range of elastic deformation

The formula from equation (3.8) was simplified to; $E_{mg} = \frac{l^3}{4.7bh^3}\left[\frac{F_2 - F_1}{W_2 - W_1}\right]$ for ease of computation , since ***a*** and ***l*** are known as 300 and 900 respectively which are constant.

ASTMD193 has given a formula for calculating the global modulus of elasticity for three point test as follows;

$$E_{mg} = \frac{l^3}{48I}\frac{(f_2 - f_1)}{(w_2 - w_1)} \qquad (3.16)$$

l = length between two points of the machine grip, I= second moment of area = $\frac{bh^3}{12}$

3.3.3.1: Characteristic value of modulus of elasticity: The Characteristic values of modulus of elasticity parallel to grain are given by:

$$E_{0,mean} = \frac{\sum \bar{E}_j n_j}{\sum n_j} \qquad (3.17)$$

$$\bar{E} = \left[\frac{\sum E_i}{n}\right] 1.3 - 2690 \qquad (3.18)$$

for 40 number of species i.e. nj= 40

Therefore $E_{0,mean} = \overline{E}$ (3.19)

This implies that the mean modulus of elasticity is the same as characteristic value of modulus of elasticity as prescribed by EN 384:2004.

The characteristic values of modulus of elasticity were corrected to 12% and 18% moisture content using eqns. (3.26) and (3.21):

$$E_{12\%} = \frac{E}{1+0.0143*(12-\mu)} \tag{3.20}$$

$$E_{18\%} = \frac{E}{1+0.0143*(18-\mu)} \tag{3.21}$$

μ = Measured moisture content

3.4 ALLOCATION TO STRENGTH CLASSES OF SPECIES

EN 338: 2003(E) section 6.2.2 stated that : " A timber population may be assigned to a strength class if its characteristic values of bending strength and density equal or exceed the values for that strength class and its characteristic mean modulus of elasticity in bending equals or exceeds 95% of the value for that strength class given in Table 1((reproduced at Appendix E)."

3.4.1 Tests for Goodness of Fit and probability distribution models

3.4.1.1 Test for Goodness of Fit (GOF) : According to (Ang and Tang, 1975; Mohammed, 2010) the basic procedure involves the comparison between the experimental cumulative frequency (Empirical Cumulative Distribution Function, ECDF) and the assigned theoretical distribution function. If the discrepancy is large with respect to what is normally expected from a given sample size, the theoretical model is rejected. The hypothesis is that "the data follow a specified distribution". Assume a random sample x_1, ... , x_n from some distribution with CDF *F(x)*. The empirical CDF is denoted by:

$$Fn(x) = \frac{1}{n}[\text{Number of Observations} \leq x] \tag{3.22}$$

The Kolmogorov-Smirnov (K-S) statistic (D) is based on the largest vertical difference between the theoretical and the empirical cumulative distribution function:

$$D = \max_{1 \leq x \leq n} \left(F(x_i) - \frac{i-1}{n} \frac{i}{n} - F(x_i) \right) \tag{3.23}$$

Where

D = Statistics of Kolmogorov Smirnov test, F = Cumulative distribution being tested.

The results are presented in the form of interactive table that help decide the model which describes the data in the best way. The hypothesis regarding the distributional form is rejected if the test statistics, D, is greater than the critical value at 0.05 confidence level.

3.5 RELIABILITY ANALYSIS

The reliability - based analysis was carried out with a computer program written in FORTRAN 77. The program was developed using first order reliability algorithm, FORM 5 (Gollwitzer, 1988). The computer program performs reliability- based analysis of axially loaded solid slender column as a member in three hinged portal frame. The design values used in the stochastic parameters were the properties established from the classification of each timber species. Values not obtainable from the classification were obtained from literature.

Six failure modes were considered as follows: compression criterion (failure mode I), bending criterion (failure mode II), flexural - buckling criterion (failure mode III), lateral

torsional -buckling (failure mode IV), carrying capacity of dowel criterion (failure mode V) and shear capacity failure criterion (failure mode VI).

3.5.1: Structural Design Considerations

(a) **Structural Configuration :**In this study, the structural configuration of the axially loaded solid cylinder timber column is developed on the following assumption;

(i) The column is part of three member portal frame with a height of 7.5m and spanning 10m.The frames are at 5m centres

(ii) Material factors of safety are as follows;

γ_q =1.5 for the imposed load, γ_g =1.35 for the dead load

(iii) The frame is a dwelling from service class 1 (EN 14081-1)

(iv) Strength modification factor k_{mod} is 0.6 (EC 5:2004)

(v) Properties generated from grading system of the three timber species were used in establishing the stochastic parameters of basic Variable. Where factors could not be established from the grading analysis, values were obtained from literature. Example modification factor, K_{mod}, and L which is the span of the portal frame , their covariance were obtained from Afolayan (2005).

The pitched portal frame in Fig. 3.4 was used in the design of the slender column considering the modes of failure.

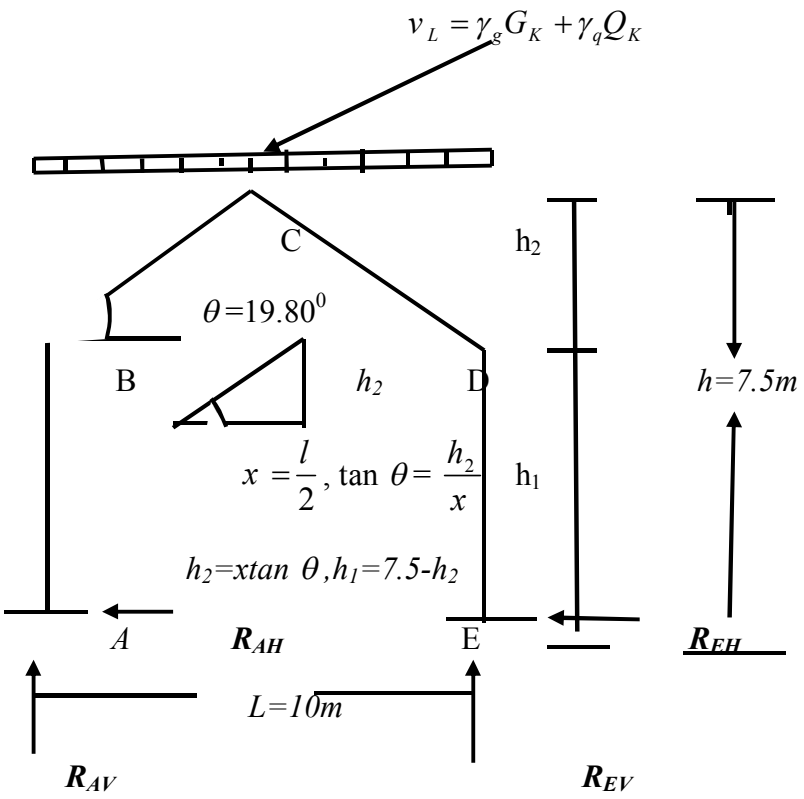
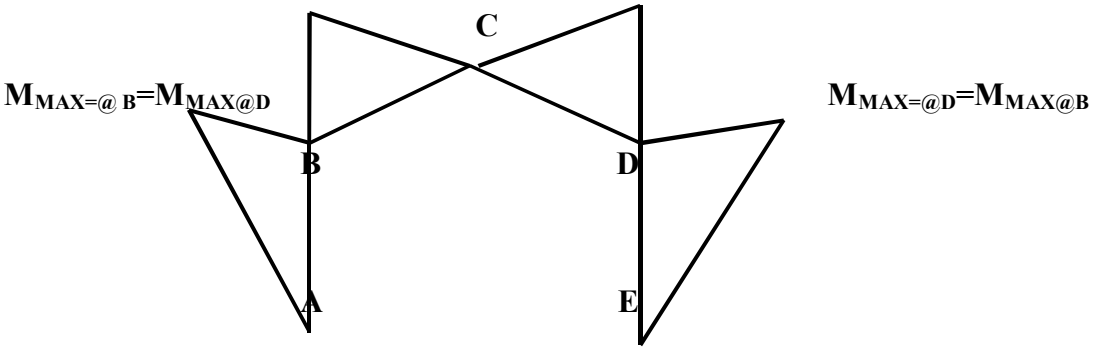


Fig 3.3: Pitched Portal Frame



Moment @ A, C and E=0

Fig. 3.4: Bending Moment Diagram of the Pitched portal frame

Column AB is to be analysed

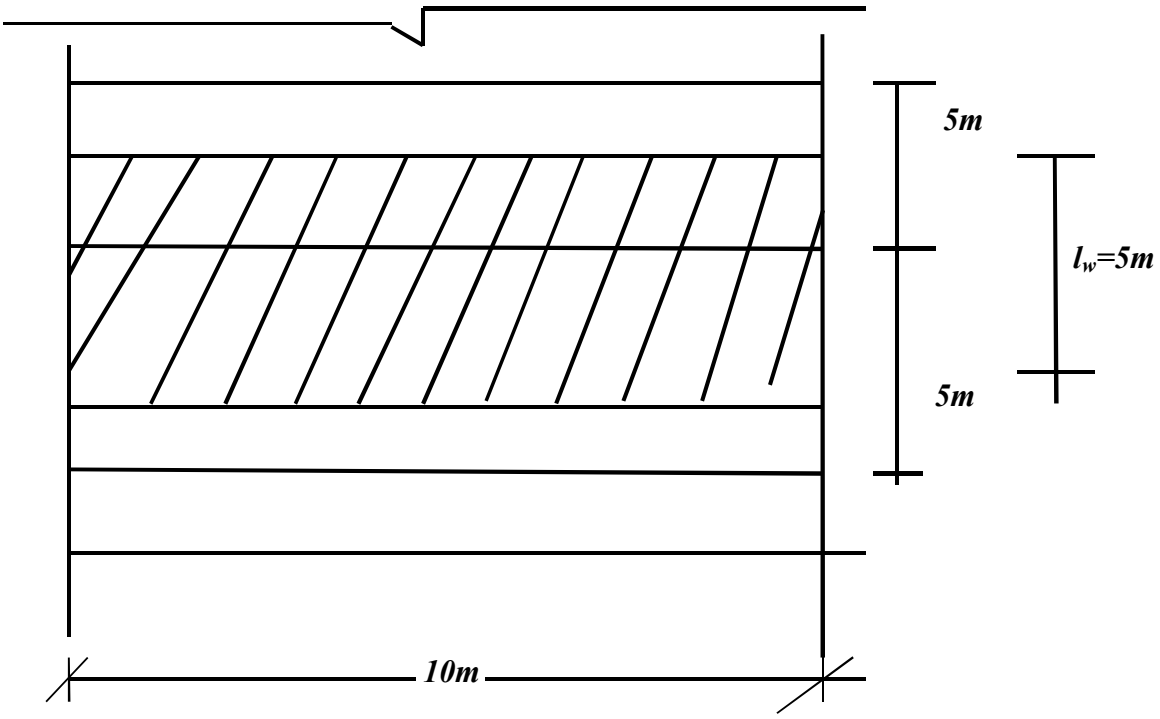


Fig 3.5 plan of the portal frame

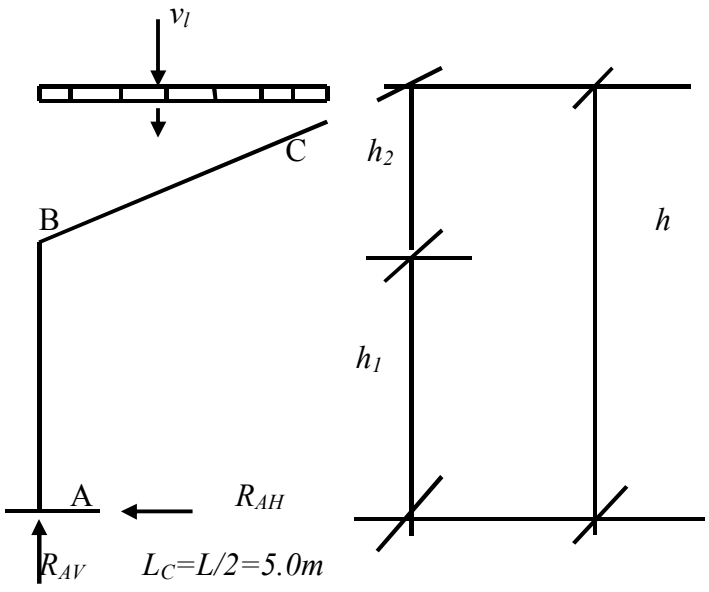
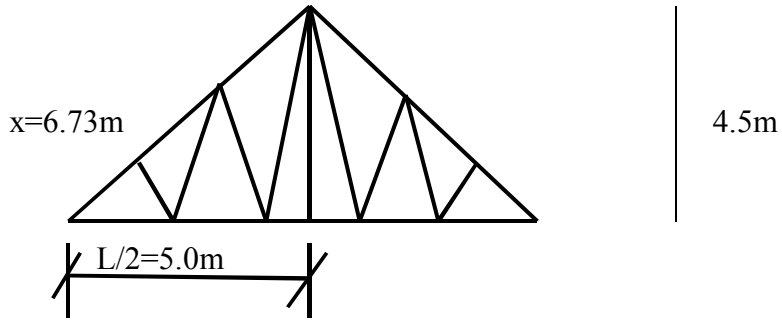


Fig 3.6 Horizontal reactions



From Fig 3.6

$$\sum M_{@c} : R_{AH}(h_1 - h_2) + R_{AV} \cdot \frac{l}{2} = V_l \cdot \frac{l}{2} \cdot \frac{l}{4} \tag{3.24}$$

$$R_{AH}.h + R_{AV}.L_C = V_L \cdot \frac{l^2}{8} \tag{3.25}$$

$$R_{AH} = \left[\left\| \frac{V_l l^2}{8} - \frac{R_{AV} l}{2} \middle/ h \right\| \right] abs \tag{3.26}$$

$$\sum H : R_{AH} + R_{EH}$$
$$R_{AH} = |R_{EH}| abs \tag{3.27}$$

and from Fig 3.3 vertical reactions are;

$$R_{ev} = V_l \cdot \frac{l}{2}$$
$$\sum V : 0 : R_{Av} + R_{ev} = V_l.l.$$
$$R_{AV} = V_l.L - R_{ev}$$

Design Load $v_L = \gamma_g G_K + \gamma_q Q_K$ (3.28)

G_k is obtained as follows;

Dead load measured on the slope;

i. Sheeting and insulation =0.25kN/m^2

ii. Purlines = 0.1kN/m^2

iii. Truss = 0.1kN/m^2

Total = 0.45kN/m^2

v. wall: sheeting, insulation board,

sheeting rails = 0.35kN/m^2

Dead load on the slope length in kN

i. Roof construction members load =0.45* 6.73*5 = 15.14kN

ii. Wall = 0.4*3.0*5 = 6.0kN

iii. Column self weight = 0.4x1.2x3.0x3.8kn/m^3=5.47kN

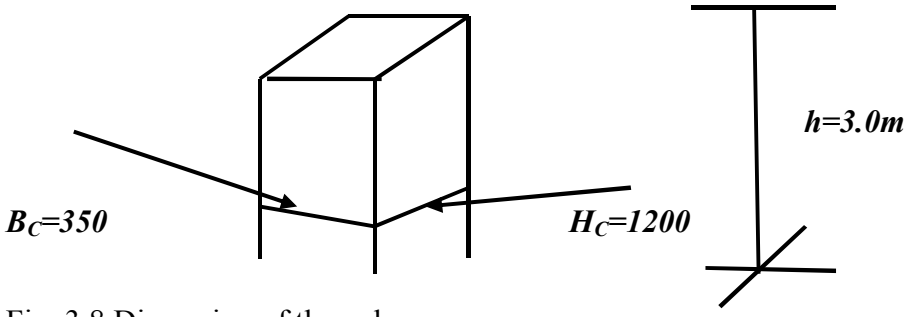


Fig. 3.8 Dimension of the column

Load from self weight in kN=*3.8kN/m^3 x0.48m^2 x3.0m*

$=5.47kN$

Note: Density of C18 class of timber is 3.8kN/m^3(EN1991-1-1-:2002(E))

Therefore total dead load =(15.14+6.0+5.47)

G_k = 26.61kN

Imposed load Q_k =0.5KN/m^2 (average of the range 0.0-1.0) as in EN 1991-1-1:2002(E)

Q_k measured on plan =0.5kN/m^{2}x7.5x5 = **18.75kN**

Wind load: There is no wind action since the height of the structure is<10m (EN 1991-4(wind Action).

Load ratio $\frac{G_k}{Q_k} = \frac{26.61}{18.75} = 1.42$

Design load $V = \left(\gamma_g G_k + \gamma_g Q_k\right)$

= (1.35x26.61+1.5x18.75)

= 64.05kN

vertical reactions

$R_{ev} = VL - R_{EV}$

$R_{EV} = \frac{VL}{2}$ (3.29)

$= \frac{64.05x\,10}{2}$

=320.25kN

And R_{AV}=VL-R_{EV} (3.30)

= 64.05x10-320.25

= 320.25kN

horizontal reactions

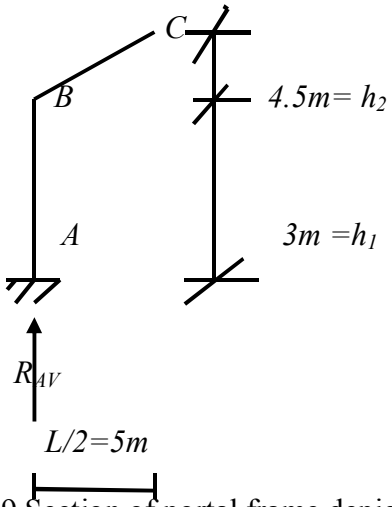


Fig. 3.9 Section of portal frame depicting Horizontal Reactions

From Equation (3.24)

$$\sum M_{@C} = R_{AH}\left(h_1 + h_2\right) = \frac{Vl^2}{8} - R_{AV}\frac{l}{2}$$

$$R_{AH} = \left|\frac{\frac{Vl^2}{8} - \frac{R_{AV}}{2}}{\left(h_1 + h_2\right)}\right| abs$$

$$R_{AH} = \frac{\left|\left(\frac{64.05x\,5.0^2}{8} - \frac{320.25x\,5}{2}\right)\right|}{7.5} abs$$

$$= 80.06kN$$

$$\sum H = 0 : R_{AH} + R_{EH} = 0$$
$$R_{EH} = \left|R_{AH}\right| abs = 80.06kN$$

Moments

$M_{@B}=M_B=$

$$\left|R_{AH}.h_1\right|abs$$
$$=80.06*3$$
$$=240.18kNm$$
$$M_B=M_D=240.18kNm$$

3.6.: EUROCODE 5(2004) PROVISIONS FOR COLUMNS AND LIMIT STATE EQUATIONS

3.6.1 Compression Failure criterion

The design resistance of the timber material needs to be computed using EN1995 (EC5 (2004)) by reducing the ultimate characteristic strength of the material according to the following formula;

$$f_{c,0,d}=\left(\frac{k_{\text{mod}}f_{c,0,k}}{\gamma_m}\right) \tag{3.31}$$

$where, f_{c,0,d}, is$ design compressive strength

Applied stress will thus be; F= $\frac{R_{AV}}{BH}$ (3.32)

B=width of the timber, H=depth of the cross sectional area of the timber

The limit state equation: $G_{(x)}=R-S$ (3.33)

R=Design compressive strength, S= Applied stress

Limit state equation will therefore be $G_{(x)} = \frac{k_{\mathrm{mod},} f_{c,0,k}}{\gamma_m} - \frac{R_{Av}}{BH}$

Where k_{mod} is load duration factor which is 0.6 for permanent action for solid timber service class 1 (EN 14081-1)

$f_{c,0,k}$ = characteristic compressive strength // to grain, γ_m =1.3 for solid timber (EN 1991-1 Table 2.2)

R_{AV} is Axial load in column, *B, H*= width and depth of column respectively.

Therefore, $f_{c,0,d} = \frac{0.6*18}{1.3}$

$= 8.308 \mathrm{N/mm^2}$

Now, $G_{(x)} = \frac{k_{\mathrm{mod},} f_{c,0,k}}{\gamma_m} - \frac{R_{Av}}{BH}$ (3.33a)

R_{AV}= 320kN obtained from eqn. (3.30), B is 400mm (width of section) and H is 1200mm.

Therefore,

$G_{(x)} = 8.308-0.67$

3.6.2 Bending criterion

The bending strength given by EC5 (2004) is

$$f_{m,d} = \left(\frac{k_{\mathrm{mod},} f_{m,k}}{\gamma_m} \right) \quad (3.34)$$

Applied bending stress $= \frac{M}{Z}$ (3.35)

Therefore $G_{(x)} = R - S$

The limit state equation is;

$$G_{(x)} = \frac{k_{\text{mod}} f_{m,k}}{\gamma_m} - \frac{M}{Z} \quad (3.36)$$

Where $M=M_B=M_D$ moment @ B which is maximum

And $Z = \frac{I}{y} = \frac{12}{\frac{H}{2}} = \frac{BH^2}{6}$

Therefore $Z=\frac{BH^2}{6}$

From equation (3.36) the limit state equation is;

$$G_{(X)} = \frac{k_{\text{mod}} f_{m,k}}{\gamma_m} - \frac{6M}{BH^2} \quad (3.37)$$

M is maximum bending moment $M_B=M_D$ 240.18kNm , $f_{m,k}$ = characteristic bending strength which is 18N/mm^2, γ_m =material safety factor

Therefore $G_{(x)}= \frac{0.6*18}{1.3} - \frac{6.0*240.18*10^6}{400*(1200)^2}$

= 8.308-2.50

O.k. since R (8.308) is > S (2.50)

3.6.3 Flexural Buckling Failure

The column AB of Fig. 3.9 was analysed. There is the tendency for the column to buckle in bending due to axial load subjected. EC5 (2004) gives the flexural buckling strength of a timber to satisfy the interactive formula in the following equation;

$$\frac{\sigma_{md}}{f_{m,d}} + \left(\frac{\sigma_{c,0,d}}{k_{cz} f_{c,0d}} \right) \leq 1 \tag{3.38}$$

The limit state equation is given by;

$$G_{(X)} = R - S \tag{3.39}$$

$$G_{(X)} = 1 - \left(\frac{\sigma_{md}}{f_{m,k}} + \frac{\sigma_{c,0,d}}{K_{cz} f_{c,0,d}} \right) \tag{3.40}$$

σ_{md} = Bending stress, $k_{cy} = k_{cz}$ instability facor, $f_{c,0,d}$ = compressive strength, $f_{m,d}$ = bending strength

And

$\sigma_{c,0,d} = \frac{R_{AV}}{BH}$ = compressive stress, $\sigma_{md} = \frac{6M_B}{BH^2}$ = bending stress

Instability factor $k_{c,z}$ from eqn. (3.40) is given as:

$$k_{cz} = \frac{1}{k_{y,z} + \sqrt{k_{y,z}{}^2 - \lambda^2 r_{el},z}} \tag{3.41}$$

Similarly for $k_{cy} == \frac{1}{k_{z,y} + \sqrt{k_{z,y}{}^2 - \lambda^2 r_{el},y}}$

and

$$k_z = 0.5\left(1 + \beta_c(\lambda_{rel},z,-0.5) + \lambda^2{}_{rel},z\right) \tag{3.42}$$

Similarly for $k_y = 0.5\left(1 + \beta_c(\lambda_{rel},y,-0.5) + \lambda^2{}_{rel},y\right)$

$\beta_c = 0.2$ for solid timber

$$\lambda_{rel} = \sqrt{\frac{f_{c,0,k}}{\delta_{c,crity}}} \tag{3.43}$$

$$\sigma_{c,crity} = \pi^2 \frac{E_{0.05}}{\lambda_y{}^2} \tag{3.44}$$

$$\lambda_y = \frac{l_y}{i_y} \tag{3.45}$$

Similarly for λ_z

l_e = Effective length and can be determined as follows;

H

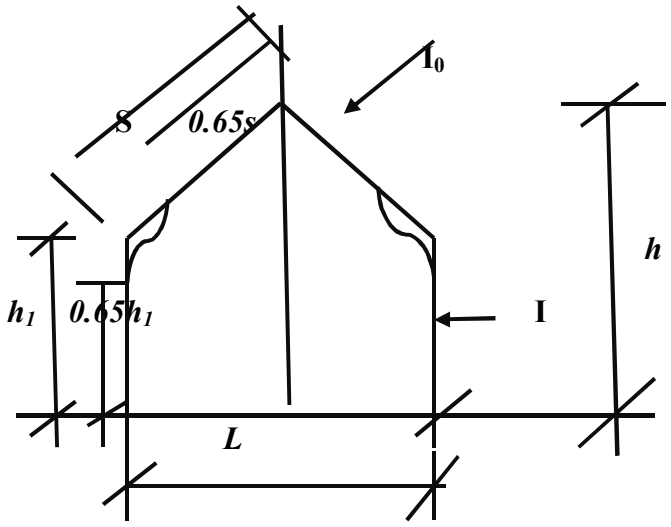


Fig.3.10: Detailed Dimension of Portal Frame

$$l_e = h_1\sqrt{4+3.2\frac{Is}{I_0 h_1}+10\frac{EI}{h_1 k_r}} \tag{3.46}$$

And $\frac{I}{I_0}$=1.0

Assuming that the column and rafter are of the same cross section

where; I= moment of inertia of column, I_0= moment of inertia of rafter

Therefore;

$$l_e = h_1\sqrt{4+\frac{3.2s}{h_1}+10\frac{EI}{h_1 k_r}} \tag{3.47}$$

s= length of rafter, h_1=length of the column, E=five percentile of the column Elastic modulus, k_r= rotational stiffness of the connection between beam and column.

$$k_r = 2ku\left[n_1 r_1^2 + n_2 . r_2^2\right] \tag{3.48}$$

$$k_u = 2\frac{k_{ser}}{3} \tag{3.49}$$

and $$k_{ser} = \rho_k^{1.5} . \frac{d}{20} \tag{3.50}$$

d= diameter of bolts

$$k_r = \frac{2k_{ser}}{3}\left[n_1 r_1^2 + n_2 . r_2^2\right] \tag{3.51}$$

n_1 is number of bolts (large circle)

n_2 is number of bolts (small circle)

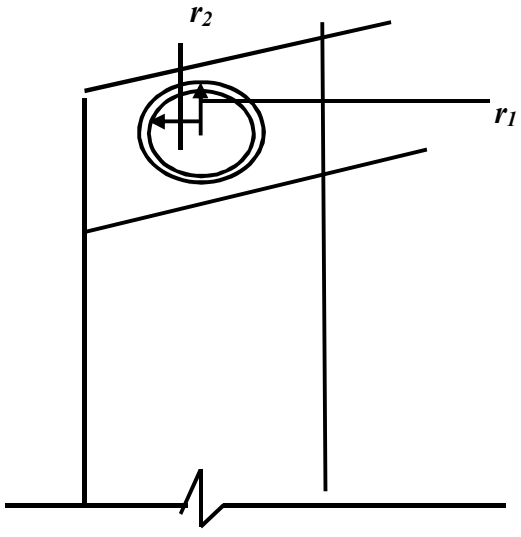


Fig. 3.11: Detail of Dowel Connection

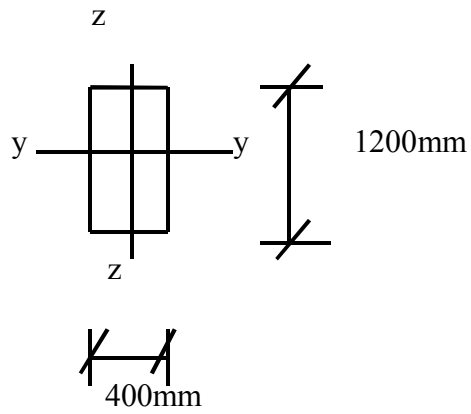


Fig.3.12: Section of the Column

Area A=bh =400mmx1200=480x103mm^2, Mean modulus of Elasticity E= 9kN/mm^{2,} 5th percentile of ME, $E_{0.05}$=6KN/mm^2

ρ_k =charcteristic density=320kg/m^3

$\rho_{m=}$mean density=380kg/m^3

$\sigma_{c,0,d} = \frac{R_{AV}}{BH}$ =0.67N/mm2

$\sigma_{m,0d} = \frac{6MB}{BH^2}$ =2.50N/mm^2

$f_{m,d}$ =Bending-strength=8.31N/mm^2

$f_{c,0,d}$ =compressive strength $= \frac{k \bmod f_{c,0,k}}{\gamma}$ =8.31N/mm^2 ,

I_y= second moment of area about y-y axis= bh^3/12=400 $* \frac{1200^3}{12}$

= 57.6x10^{9}mm^4

Radius of gyration along y-y axis;

$$i_y = \sqrt{\frac{Iy}{A}} = \sqrt{\frac{57.6*10^9}{480*10^3}} = 346.41mm$$

Radius of gyration along z-z axis;

$$I_z = \frac{hb^3}{12} = 1200*400^2/12 = 6.4x10^9 mm^4$$

$$i_z = \sqrt{\frac{I_z}{A}} = \sqrt{\frac{6.4*10^9}{480*10^3}} = 115.47mm$$

Slenderness ratio about y-y axis

$$\lambda_y = \frac{l_{ey}}{i_y}$$

$$l_{ey} = effective - length = h_1\sqrt{4 + \frac{3.2S}{h_1}\frac{10EI}{h_1 k_r}}$$

As $I/I_o = 1$(same cross section for the column and rafter)

I_y=57.6x109mm4, S=length of rafter=6.73m, h_1=3.0m, height of column, k_r=2ku

From eqn.3.51

$$kr = \frac{2k_{ser}}{3}\left[n_1 r_1^2 + n_2 . r_2^2\right]$$

$$r \le 0.5h - 4d$$
$$\le 0.5*1200 - 4*24 = 504mm$$
$$d = diameter - of - dowel$$
$$r_2 \le r_1 - 5d$$
$$= 504 - 5*24 = 384mm$$
$$no\,of bolts$$
$$= n_1 \le \frac{2\pi r_1}{6d} = \frac{2\pi * 504}{6*24} = 21.99 \approx 22no. \qquad (3.52)$$

$$= n_2 \le \frac{2\pi r_2}{6d} = \frac{2\pi * 384}{6*24} = 16.7 = 17no.$$

from equation 3.50;

Therefore $k_{ser} = \rho_m{}^{1.5} \frac{d}{23} = (380\frac{m^3}{kg})^{1.5}.\frac{24mm^{-1}}{23}$

$$= 7.73x10^3 N/mm$$

And

$$k_u = \frac{2*7.73*10^3}{3}\left(22*504^2 + 17*384^2\right)$$
$$= 4.17*10^{10}$$
$$k_r = 2ku = 2*4.17*10^{10}$$
$$= 8.34*10^{10}$$

Therefore $l_{ey=}$

$$300\sqrt{4 + \frac{3.2*6730}{3000}} + \sqrt{\frac{10*9000*57.6*10^9}{3000*8.34*10^{10}}}$$
$$= 3000\sqrt{4 + 7.18 + 20.72}$$
$$l_{ey} = 16,944.03mm$$

Therefore slenderness ratio about y-y axis is

$$\lambda_y = \frac{l_{ey}}{i_y} = \frac{16,944.03}{346.41} = 48.91$$

Now slenderness ratio along z-z axis

$$\lambda_z = \frac{l_{ez}}{i_z}$$

$$l_{z=}h_1\sqrt{4+\frac{3.2S+}{h_1}}+\sqrt{\frac{10EI_2}{h_1k_r}}$$

$I_z = 6.4\text{x}10^9\text{mm}^4$

Therefore

$$l_{ez} = 3000\sqrt{4+\frac{3.2*6730}{3000}}+\sqrt{\frac{10*9000*6.4*10^9}{3000*8.34*10^{10}}}$$

=11,014.53mm

And

$$\lambda_z = \frac{l_{ez}}{i_z} = \frac{11,014.53}{115.47} = 95.39$$

Buckling Resistance

Relative slenderness about y-y axis

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \qquad (3.53)$$

$$= \frac{48.91}{\pi}\sqrt{\frac{18}{6000}}$$

$$= 0.85$$

Relative slenderness about z-z axis

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \qquad (3.54)$$

$$= \frac{95.39}{\pi}\sqrt{\frac{18}{6000}}$$

$$= 1.66$$

Both the two slenderness ratios are > 0.3

Therefore EC5, 6.3.2(3) applies

Maximum slenderness ratio of the column is; $\lambda_{rel,z} = 1.66$, $\beta_c = 0.2$ for solid timber as in Eqn. (6.29) of EC5 (2004))

$$k_z = 0.5\left[1 + \beta_c(\lambda_{relz} - 0.3) + \lambda_{rel,z}{}^2\right] \quad \text{as in eqn. (6.26) of EC 5 (2004))} \qquad (3.55)$$

$=0.5[1+0.2(1.66-0.3)+1.66^2]$

$K_z = 2.01$

$$k_{cz} = \frac{1}{k_z + \sqrt{k^2{}_z - \lambda_{rel,z}{}^2}} \quad , \quad \text{instability factor as in eqn. (6.26) of EC5 (2004)} \qquad (3.57)$$

$$k_{cz} = \frac{1}{2.01 + \sqrt{2.01^2 - 1.66^2}}$$

$$= 0.32$$

Therefore for flexural buckling failure to be satisfied, the column must satisfy eqn. (3.38) as follows:

$$\frac{\delta_{md}}{f_{m,d}} + \left(\frac{\delta_{c,0,d}}{k_{cz} f_{c,0d}} \right) \leq 1$$

$$\left(\frac{2.50}{8.31} + \frac{0.67}{0.32x\ 8.31} \right) \leq 1$$
$$= (0.30 + 0.25) \prec 1$$
$$= 0.55 \prec 1 - o.k$$

3.6.4 Lateral Torsional Buckling

The equation below is for lateral torsional buckling as provided by EC 5 (2004)

$$\left(\frac{\sigma_{m,y}}{k_{crit} f_{m,d}} \right)^2 + \frac{\sigma_c}{k_{c,z} f_{c,0,d}} \leq 1 \tag{3.56}$$

$k_{c,y}$ = is related to flexural buckling about the y- axis, like wise $k_{c,z}$ is about z- axis.

The limit state equation is;

$$G_{(x)} = R - S \tag{3.57}$$

All the expressions in the equation 3.56 are defined under flexural buckling.

$$k_{crit} = \begin{cases} 1 & \lambda_{\text{rel,m}\leq 0.75} \\ 1.56 - 0.75\lambda_{rel,m} & \text{for } 0.75 < \lambda_{\text{rel,m}} < 1.4 \\ \dfrac{l}{\lambda_{rel,m}{}^2} & \text{otherwise} \end{cases} \tag{3.58}$$

Lateral instability factor is given as (EC 5(2004));

$$k_{cy} = \frac{1}{k_y + \sqrt{k^2{}_y - \lambda_{rel,y}{}^2}} \tag{3.59}$$

Similarly for $k_{cz} = \frac{1}{k_z + \sqrt{k^2{}_z - \lambda_{rel,z}{}^2}}$

And $\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$ (3.60)

$\sigma_{m,crit}$ = is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values.

For softwood, the critical bending stress $\sigma_{m,crit}$ with solid rectangular cross section may be taken as; (EC5, 2004)

$$\sigma_{m,crit} = \frac{0.78\ Bc^2 E_{0.05}}{hl_{ef}} \quad (3.63)$$

$$\frac{0.78 * 400^2 * 6000}{3000 * 6730}$$
$$= 37.09 N/mm^2$$

Therefore;

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{mcrit}}}$$
$$= \sqrt{\frac{18}{37.09}}$$
$$= 0.69 < 0.75$$

Therefore;

$k_{crit} = 1$

Now, from eqn.(3.56)

$$\left(\frac{\sigma_{m,y}}{k_{crit}f_{m,d}}\right)^2+\frac{\sigma_c}{k_{c,z}f_{c,0,d}}\le 1$$

$$\left[\left(\frac{2.50}{1.0x\,8.31}\right)^2+\left(\frac{0.67}{0.32x\,8.31}\right)\right]<1$$

$$=(0.072+0.26)$$

$$=0.33<1-ok$$

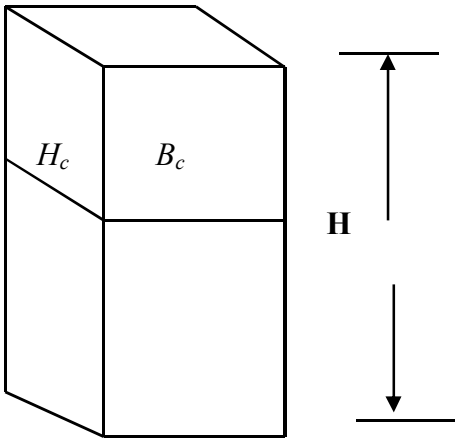


Fig. 3.13: Column section

3.6.5 Design of Moment Resisting Joint

Characteristic Bending Strength = f_{mk}=18N/mm^2

Characteristic Shear strength = $f_{v,k}$ =2.0N/mm^2

Characteristic Density = ρ_k =320 kg/m^3

Therefore, design bending strength = $f_{m,d} = k_{\mathrm{mod.}} \frac{f_{mk}}{\gamma_m} = \frac{0.6*18}{1.3} = 8.31N/mm^2$ (3.61)

Design shear Strength = $f_{v,gd} = k_{\mathrm{mod}} . \frac{f_{v,k}}{\gamma_m} = \frac{0.6*2}{1.3} = 0.92N/mm^2$ (3.62)

Steel grade S 235 ϕ=24mm , Tensile Strength of Each screw = f_{vk} =*400N/mm*^{2}, Vertical design force = V_{dc} = 62.12KN, design moment =M_d =232.95kNm, normal force N_d= 310.6kN, r_1= 504mm,

$r_{2=}$ 384mm, n_1 =22 no., n_2=17no.

Load of dowels in column and rafter of the frame due to BM,

$$FM = Md \frac{r_1}{n_1 r_1^2 + n_2 r^2} = 232.95*10^6 x \frac{504}{22x\,504^2 + 17*384^2}$$
$$= 14.5*10^3 N$$
(3.63)

Load of dowel in column of the frame due to shear and normal force,

$$F_{v,c} = \frac{V_{d,c}}{n_1 + n_2} = \frac{62.12*10^3}{22+17} = 1.59x\,10^3 \quad (3.64)$$

$$F_{N,c} = \frac{N_d}{n_1 + n_2} = \frac{310.6*10^3}{22+17} = 7.89x\,10^3 \quad (3.65)$$

load of dowel in rafter due to shear and normal force,

$F_{v,R} = F_{v,c}$= 1.59x10^{3}N

$F_{N,R} = F_{N,C}$ = 7.89x10^3

Total load of dowel in the axis of the rafter and column of the frame;

$$F_{d,c} = \sqrt{(F_M + F_{v,c})^2 + F_{N,c}{}^2}$$
$$= \sqrt{(14.5*10^3 + 1.59*10^3) + (7.89*10^3)^2}$$
$$= 17.92*10^3 N$$
$$F_{d,c} = F_{d,r} = 17.92*10^3 \quad (3.66)$$

Shear force in column and rafter joint;

$$V_M = \left(\frac{M_d}{\pi} \frac{n_1 r_1 + n_2 r_2}{n_1 r_1^2 + n_2 r_2^2} \right) \quad (3.67)$$
$$= \left(\frac{232.95*10^6}{\pi} . \frac{22x\,504 + 17*384}{22x\,504^2 + 17*384^2} \right)$$
$$= 161.36*10^3 N$$

$$F_{vdc} = V_M - \frac{V_{dc}}{2} \quad (3.68)$$

$$F_{vdc} = 161.36*10^3 - \frac{62.12*10^3}{2}$$
$$= 130.3*10^3 N$$
$$F_{vdc} = F_{VdR} = 130.3*10^3 N$$

3.6.5.1 The mechanical properties of Dowels

Embedding strength in Fibres (characteristic values);

3.6.5.1(a) Carrying Capacity of dowel in column axis

Angle between load and timber fibres

$$\alpha_1 = \arctan\left(\frac{F_M + F_{VC}}{F_{NC}} \right) \quad (3.69)$$

$$\alpha_2 = \alpha - \left(\frac{\pi}{2} - \alpha_1\right) \tag{3.70}$$

$$\alpha_1 = \arctan\left(\frac{14.5*10^3 + 1.59*10^3}{7.89x\,10^3}\right)$$

$$= 63.88^0$$

$$\alpha_2 = \alpha - \left(\frac{\pi}{2} - \alpha 1\right)$$

$$= 19.8 - \left(\frac{\pi}{2} - 63.88^0\right)$$

$$= 19.8 - \left(90 - 63.88^0\right)$$

$$= -6.32^0$$

Embedding strength (characteristic value)

K_{90}= 1.35+0.015d = 1.35 +0.015x2 (3.71)

=1.71

$$f_{h,1,k} = \frac{f_{h,0,k}}{k_{90}.\sin^2\alpha_1 + \cos^2\alpha_1} = \frac{19.9}{1.71\sin^2 63.88 + \cos^2 63.88} \tag{3.72}$$

$$= 12.66N/mm^2$$

$$f_{h,2,k} = \frac{f_{h,0,k}}{k_{90}.\sin^2\alpha_2 + \cos^2\alpha_2} = \frac{19.9}{1.71\sin^2(-6.32) + \cos^2(-6.32)} \tag{3.73}$$

$$= 19.73N/mm^2$$

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} = \frac{19.73}{12.66}$$

$$= 1.558 \qquad (3.77)$$

β is the embedding strength

Yield moment (characteristic value);

Characteristic fastener yield moment;

$$M_{y,Rk} = 0.3 fu,kd^{2.6} \qquad (3.75)$$

$$M_{y,Rk} = 0.3x\ 400x\ 24^{2.6}$$

$$= 465.3x\ 10^{3}\ Nmm$$

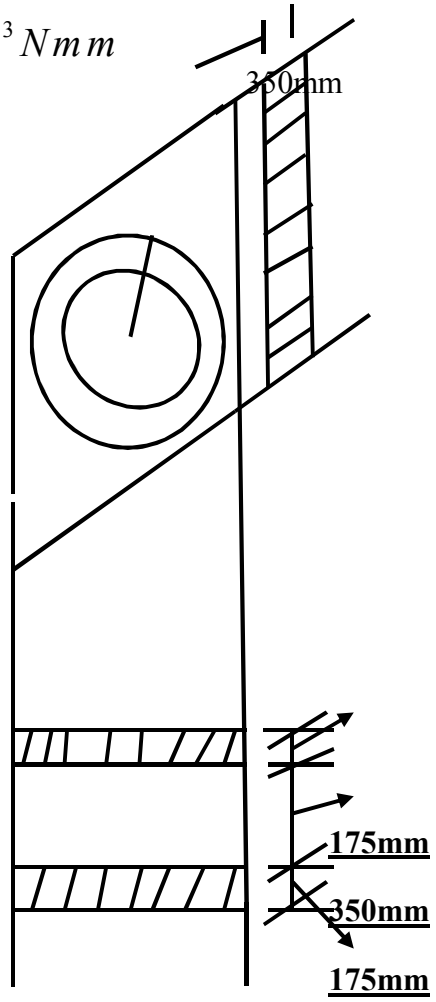


Fig. 3.13a: Column / rafter dowel connection

$f_{u,k}$=tensile strength of each screw

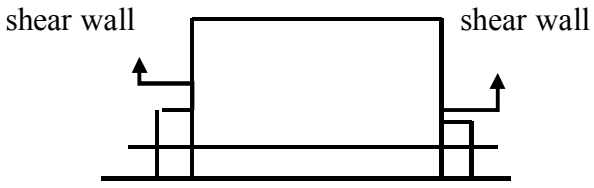
with of column =2x175=350mm

width of rafter = 350mm

t_1=the smaller of the thickness of the timber side member or the penetration depth = 175mm

t_2= is the thickness of the timber, middle member =350mm

For the fastener in double shear, the figure below illustrate the shear wall;



Double shear: 2 shear plane per fastener

Embedding strength characteristic value; $f_{V,RK}$ =

$$F_{v,Rk} = Min\begin{cases} f_{h,1,k}\, t_1 = 12.66*175x\,24 = 53.172*10^3 N \\ 0.5 f_{h,2,k}\, t_2 d = 0.5*19.73*35*24 = 82.866*10^3 N \\ 1.05\dfrac{f_{h,1,k}\, t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}\, d\, t_1^{\,2}}}-\beta\right]+\dfrac{[FaxRk]}{4} \\ 1.05.\dfrac{53.17*10^3}{3.558}\left[\sqrt{3.116(3.558)+\dfrac{19.42*465.3*10^3}{12.66*24*175^2}}-1.558\right]+0 = 30.04*10^3 \\ 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2My,Rk\, f_{h,1,k}\, d}+\left[\dfrac{Fax.Rk}{4}\right] \\ 1.15\sqrt{\dfrac{2*1.558}{1+1.558}}.\sqrt{2*465.3*10^3*12.66*24} = 21.34*10^3 N \\ *Fax.Rk = 0 \end{cases} \quad (3.76)$$

Therefore, $F_{V,RK}$ =21.34*10^{3}N being the minimum

Now, embedding strength design value is given by;

$$\Rightarrow F_{v,Rd,c} = \frac{K_{\mathrm{mod}}.F_{v,Rk}}{\gamma_m} = \frac{0.6*21.34*10^3}{1.3} = 9.85x\,10^3 N \quad (3.77)$$

Check

$F_{d,c}$ = Total load of dowel in the rafter in the axis of column;

from eqn.(3.66);

$$= \sqrt{\left(F_M + F_{V,C}\right)^2 + F^2{}_{N,c}}$$

$F_{V,R,dc}$ =Embedding strength of the dowel in axis of column

$$F_{d,c} \le 2F_{V,R,dc} \tag{3.78}$$

$$F_{d,c} = 17.92*10^3$$

$$\Rightarrow 17.92*10^3 < 2*9.85*10^3 N$$

$$17.92*10^3 < 19.7*10^3 N$$

okay.

3.6.5.1 (b) Carrying capacity of dowel in rafter axis: This is the same as in column axis

since $F_{v,c}=F_{vR}$ = load of dowel in column and in rafter respectively due to shear force

$F_{N,C}=F_{NR}$ = load of dowel in column and in rafter respectively due to normal force

3.6.5.2 Shear stress in frame column and rafter assessment

3.6.5.2 (a) column

$\tau_{v,c}$ = shear stress of column

$$\tau_{v,c} = 3/2\frac{Fv,dv}{2b.h} = \frac{3*130.3*10^3}{2*2*175*1200} = 0.47N/mm^2 \tag{3.79}$$

Check

From eqn.3.62;

$f_{v,g,d}$ = design shear strength = $\frac{k \bmod f_{v,K}}{\gamma_m}$

$F_{v,k=}2$= shear strength, characteristic value

$$\tau_{v,c} \leq f_{v,g,d} \tag{3.80}$$

$\Rightarrow 0.47 < 0.92$

okay

3.6.5.2 (b) <u>rafter</u>

$\tau_{v,c} \leq \tau_{v,R}$

okay

3.7. LIMIT STATE EQUATIONS FOR THE SIX FAILURE MODES

3.7.1 Compression Criterion

$$G_{(x)} = f_{c,0,d} - \sigma_{c,0d}$$

$$G_{(x)} = \frac{k_{\text{mod}} f_{c,0,d}}{\gamma} - \frac{R_{AV}}{BH} \tag{3.81}$$

$$but\ R_{AV} = Vl - R_{EV} \tag{3.82}$$

$$and\ \ VL = (1.35g_k + 1.5q_k)l$$

and

$$R_{EV} = \frac{Vl}{2} = \frac{l}{2}(1.35g_k + 1.5q_k) \tag{3.83}$$

$$Therefore\ \ R_{AV} = (1.35g_k + 1.5q_k) - \frac{l}{2}(1.35g_k + 1.5q_k) \tag{3.84}$$

$$\alpha = \frac{g_k}{qk} = 1.42$$

Multiply equation 3.84 through by $\frac{qk}{qk}$=1; to establish the load ratio $\alpha = \frac{q_k}{g_k}$ thereby simplifying the equation

$$\Rightarrow R_{AV} = \frac{q_k}{q_k}\left[(1.35g_k + 1.5q_k)l - \frac{l}{2}(1.35g_k + 1.5q_k)\right]$$

$$= \frac{q_k l}{2}\left[2(1.35\alpha + 1.5) - (1.35\alpha + 1.5)\right]$$

$$R_{AV} = \frac{q_k l}{2}(1.35\alpha + 1.5) \tag{3.85}$$

Therefore;

$$G_{(X)} = \frac{k_{\text{mod}} f_{c,0,k}}{\gamma_m} - \frac{Q_k l}{2BH}\left[(1.35\alpha + 1.5)\right] \tag{3.86}$$

i. Bending Criterion

$$G_{(x)} = f_{m,0,d} - \sigma_{m,0,d} \tag{3.87}$$

$$G_{(x)} = \frac{k_{\text{mod}} f_{m,k}}{\gamma} - \frac{6M_B}{BH^2}$$

$$But \quad M = R_{AH}.h_1 \tag{3.88}$$

$$R_{AH} = \left(\frac{Vl^2}{8} - \frac{R_{AV} l}{2}\right)\Big/ l \tag{3.89}$$

$$Vl^2 = (1.35g_k + 1.5q_k)l^2 \tag{3.90}$$

$$\therefore \frac{Vl^2}{8} = \frac{l^2}{8}(1.35g_k + 1.5q_k) \tag{3.91}$$

$$and \quad R_{AV} = \frac{lq_k}{2}(1.35\alpha + 1.5)$$

$$\text{therefore} \quad \frac{R_{AV} l}{2} = \frac{l^2}{4} q_k (1.35\alpha + 1.5) \tag{3.92}$$

Substituting eqns. (3.92) and (3.91) into eqn. (3.89)

$$R_{AH} = \frac{\frac{l^2}{8}(1.35g_k + 1.5) - \frac{l^2 q_k}{4}(1.35\alpha + 1.5)}{l}$$

$$= (\frac{l^2}{8} q_k (1.35\alpha + 1.5) - \frac{l^2 q_k}{4}(1.35\alpha + 1.5)) / l$$

$$= \frac{l^2 q_k}{8}[(1.35\alpha + 1.5) - 2(1.35\alpha + 1.5)] / l$$

$$R_{AH} = \frac{-l q_K h_1}{8}[(1.35\alpha + 1.5)]$$

From eqn. (3.88) $M_B = R_{AH} * h_1$

Therefore $M_B = \frac{-q_k l^2 h_1((1.35\alpha + 1.5))}{8l}$

Therefore $\frac{6M_B}{BH^2}$ as it appear in the eqn. $G_{(x)} = \frac{k_{\text{mod}} f_{m,k}}{\gamma} - \frac{6M_B}{BH^2}$ is given as:

$$= -\frac{6}{BH^2} \cdot \frac{l q_k h_1 \left[(1.35\alpha + 1.5)\right]}{8}$$

$$= \frac{-3L q_K h_1}{4BH^2}(1.35\alpha + 1.5)$$

$$\therefore G_{(X)} = \frac{K_{\text{mod}} f_{m,k}}{\gamma} + \frac{3LQ_K h_1}{4BH^2}(1.35\alpha + 1.5) \quad (3.93)$$

3.6.3 Flexural Buckling Criterion

$$G_{(X)} = 1 - \left(\frac{\sigma_{md}}{f_{m,k}} + \frac{\sigma_{c,0,d}}{K_c f_{c,0,d}} \right) \quad (3.94)$$

$$\sigma_{md} = \frac{6M_B}{BH^2} = -\frac{3Lq_K h_1}{4BH^2}(1.35\alpha + 1.5)$$

$$\sigma_{c,0,d} = \frac{R_{AV}}{BH} = \frac{q_K l}{2BH}(1.35\alpha + 1.5)$$

$$\therefore G_{(X)} = [(\frac{3Lq_K h_1}{4BH^2, f_{m,d}}(1.35\alpha + 1.5)) + (\frac{q_K L}{2BH, K_{cz}, f_{c,0,d}}(1.35\alpha + 1.5)] \quad (3.95)$$

$$G_{(x)} = 1 - \left(\frac{\left(q_k l \left(1.35\alpha + 1.5\right)\right)}{\left(2BH.K_{cy}, f_{c,o,d}\right)} - \frac{\left(3q_k lh_1(1.35\alpha + 1.5)\right)}{\left(4BH^2 f_{m,d}\right)} \right)$$

3.6.4 Lateral Torsional Buckling

$$G_{(X)} = 1 - \left((\frac{\sigma_{md}}{f_{crit}, f_{m,d}})^2 + \frac{\sigma_{c,0,d}}{K_{cz} f_{c,0,d}} \right)$$

$$\sigma_{md} = \frac{6M_B}{BH^2} = \frac{3LQ_K h_1}{4BH^2}(1.35\alpha + 1.5)$$

$$\sigma_{c,0,d} = \frac{R_{AV}}{BH} = \frac{Q_K L}{2BH}(1.35\alpha + 1.5)$$

$$\therefore G_{(X)} = [(\frac{3LQ_K h_1}{(4BH^2, k_{crit}, f_{m,d})}(1.35\alpha + 1.5))^2 + (\frac{Q_K L}{(2BH, K_{cz}, f_{c,0,d})}(1.35\alpha + 1.5)]$$

$$\therefore G_{(X)} = 1 - [(\frac{3LQ_K h_1}{(4BH^2, k_{crit}, f_{m,d})}(1.35\alpha + 1.5))^2 + (\frac{Q_K L}{(2BH, K_{cz}, f_{c,0,d})}(1.35\alpha + 1.5)] \quad (3.96)$$

3.8.: LIMIT STATE EQUATIONS FOR CONNECTIONS

3.8.1 Carrying capacity of Dowels

G(x) = R-S

From eqn.3.78

$$2F_{v,Rdc} - F_{d,c} \leq 0$$

$$\Rightarrow G_{(x)} = \frac{2k_{\text{mod}}.F_{V,Rk}}{\gamma_m} - \sqrt{\left(F_M + F_{V,c}\right)^2 + F^2{}_{N,C}} \quad \leq 0$$

And from eqn. (.3.77) $F_{v,Rdc}$ is given as:

From Eqns. (3.63) and (3.66), it follows that

$$M_d = M_B = \frac{LQ_K h_1\left((1.35\alpha + 1.5)\right)}{8} \tag{3.97}$$

$$\Rightarrow F_M = \frac{LQ_K h_1}{8}\left[\left(1.35\alpha + 1.5\right)\right].\frac{r_1}{n_1 r_1^{\,2} + n_2 r_2^{\,2}} \tag{3.98}$$

$$F_{V,c} = \frac{V_{d,c}}{n_1 + n_2} \tag{3.99}$$

$$V_{d,c} = Q_K\,(1.35\alpha + 1.5)1000 \tag{3.100}$$

$$\Rightarrow F_{v,c} = \frac{Q_K\,(1.35\alpha + 1.5)1000}{n_1 + n_2} \tag{3.101}$$

From Eqn. (3.65), it follows that:

$$N_d = R_{AV} = \frac{Q_k L\left(1.35\alpha + 1.5\right)}{2} \tag{3.102}$$

And,

$$\Rightarrow F_{N,C} = \frac{Q_k L\left(1.35\alpha + 1.5\right)}{2\left(n_1 + n_2\right)} \tag{3.103}$$

Therefore;

$$F_{d,c} = \sqrt{\left[\frac{LQ_K h_1\left(1.35\alpha + 1.5\right)r_1}{8\left(n_1 r^2{}_1 + n_2 r_2^{\,2}\right)} + \frac{1000Q_K\left(1.35 + 1.5\right)}{\left(n_1 + n_2\right)}\right]^2 + \left[\frac{LQ_K\left(1.35\alpha + 1.5\right)}{2\left(n_1 + n_2\right)}\right]^2}$$

$$= \left[\left[\frac{LQ_K h_1\left(1.35\alpha + 1.5\right)r_1}{8\left(n_1 r^2{}_1 + n_2 r_2^{\,2}\right)} + \frac{1000Q_K\left(1.35 + 1.5\right)}{\left(n_1 + n_2\right)}\right]^2 + \left[\frac{LQ_K\left(1.35\alpha + 1.5\right)}{2\left(n_1 + n_2\right)}\right]^2\right]^{\frac{1}{2}} \tag{3.104}$$

$$\therefore G(x) = \frac{2k_{\text{mod}}.F_{v,Rk}}{\gamma_m} -$$

$$\left[\left[\frac{LQ_K h_1(1.35\alpha+1.5)r_1}{8\left(n_1 r^2{}_1 + n_2 r_2{}^2\right)} + \frac{1000Q_K(1.35+1.5)}{(n_1+n_2)}\right]^2 + \left[\frac{LQ_K(1.35\alpha+1.5)}{2(n_1+n_2)}\right]^2\right]^{\frac{1}{2}} \quad (3.105)$$

i. Shear stress Failure Criterion

The shear stress failure criterion was determined using eqns. (3.62), (3.68) and (3.80)

$$G(x) = R - S$$

$$= \frac{K_{\text{mod}} f_{v,k}}{\gamma_m} - 3/2\left[\frac{2Vm - \left(q_k\ (1.35\alpha+1.5)1000\right)}{2bh}\right]$$

In eqn. (3.106),: the 1000 was introduced to convert the imposed load q_k from kN to N. Therefore,

$$G(x) = \frac{K_{\text{mod}} f_{v,k}}{\gamma_m} - 3/2\left[\frac{2Vm - (Qk\,(1.35\alpha+1.5)1000}{4bh}\right] \quad (3.107)$$

3.9: PROGRAM FLOWCHART

After establishing the limit state equations for the six failure modes, the FORTRAN programming was used to establish the safety indices for all the criteria using a special package called FORM5(1988) . The flowchart for the programming is shown in Fig. (3.14)

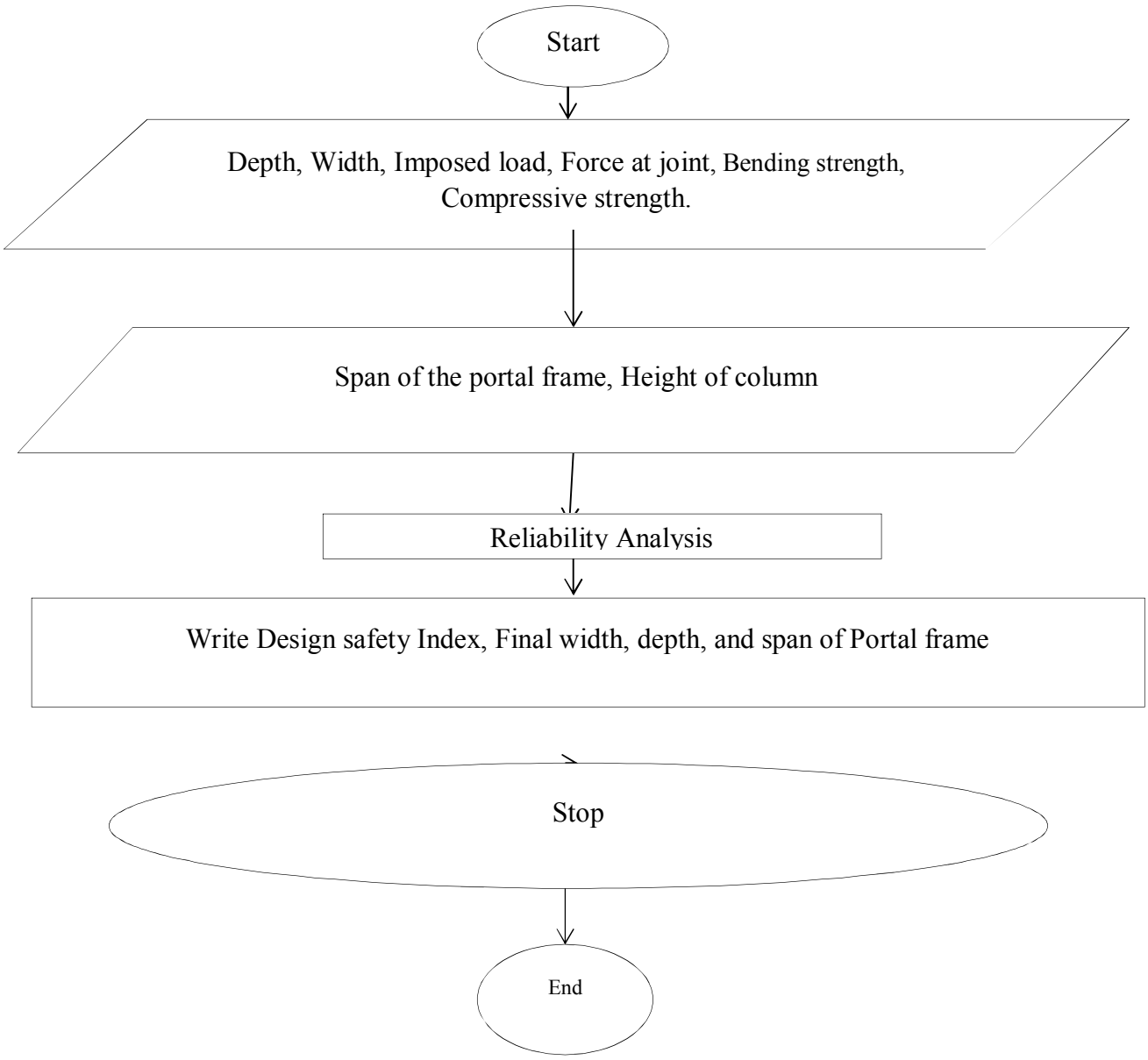


Fig. 3.14: Program Flowchart

The program starts by asking the user to input basic design variables such as, depth, width of the timber column, span of the portal frame, imposed load, height of the column and force applied at joint. The program then carries out reliability analysis of the section using Gollwitzer, et al. (1988). The implied safety index of the column is given as the output of the program, including the final width and depth of the section, as well as the span.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 RESULTS OF MOISTURE CONTENTS

Tables (4.1) to (4.3) show the results of the parameters for moisture contents determination for *Mitragyna ciliata* (Abura), *Afromosia elata* (Afromosia), and *Confusa grandiflora* (Berlinia) timber species respectively.

Table 4.1: Moisture Content for Abura Timber Species

Test Slice	Mass of moist slice M_1 (g)	Mass of Oven Dried slice M_2 (g)	mass of moisture (M_1-M_2) (g)	Moisture content W (%)
ABR01	88	68	20	29.41
ABR02	74	67	7	10.45
ABR03	89	70	19	27.14
ABR04	80	66	14	21.21
ABR05	84	70	14	20.00
ABR06	86	67	19	28.36
ABR07	76	67	9	13.43
ABR08	77	68	9	13.24
ABR09	89	69	20	28.99
ABR10	84	65	19	29.23

Test Slice	Mass of moist slice M_1 (g)	Mass of Oven Dried slice M_2 (g)	mass of moisture (M_1-M_2) (g)	Moisture content W (%)
ABR 11	81	71	10	14.08
ABR 12	78	68	10	14.71
ABR 13	79	65	14	21.54
ABR14	82	65	17	26.15
ABR 15	81	66	15	22.73
AVERAGE				21.38

Table 4.2: Moisture Content for Afromosia Timber Species

Test Slice	Mass of moist slice M_1 (g)	Mass of Oven Dried slice M_2 (g)	mass of moisture (M_1-M_2) (g)	Moisture content W (%)
AFR01	82	59	23	38.98
AFR02	74	60	14	23.33
AFR03	83	64	19	29.69
AFR04	80	66	14	21.21

Test slice	Mass of Moist slice M_1(g)	Mass of oven dried slice M_2(g)	Mass of Moisture(M_1-M_2) (g)	Moistre conteant W(%)
AFR05	84	70	14	20.00
AFR06	86	67	19	28.36
AFR07	80	67	13	19.40
AFR08	78	66	12	18.18
AFR09	89	69	20	28.99
AFR10	84	65	19	29.23
AFR11	81	65	16	24.62
AFR12	80	68	12	17.65
AFR13	83	68	15	22.06
AFR14	82	65	17	26.15
AFR15	81	66	15	22.7
AVERAGE				24.71

Table 4.3: Moisture Content for Berlinia Timber Spicies

Test Slice	Mass of moist slice M_1 (g)	Mass of Oven Dried slice M_2 (g)	mass of moisture (M_1-M_2) (g)	Moisture content W (%)
BERLI01	77	50	27	54.00
BERLI02	75	59	16	27.12
BERLI03	80	64	16	25.00
BERLI04	80	55	25	45.45
BERLI05	78	60	18	30.00
BERLI06	80	63	17	26.98
BERLI07	81	56	25	44.64
BERLI08	79	67	12	17.91
BERLI09	80	68	12	17.65
BERLI10	76	64	12	18.75
BERLI11	80	66	14	21.21
BERLI12	81	69	12	17.39
BERLI13	79	67	12	17.91
BERLI14	80	64	16	25.00
BERLI15	78	65	13	20.00
AVERAGE				27.27

The terms ABR, AFR and BERLI as used in Table (4.1), (4.2) and (4.3) are respectively Abura, Afromosia and Berlinia. The numbers 01 to 15 used in the Tables are sample no.1 up to sample no.15.

The moisture content was computed as follows:

Moisture content for ABR01: from eqn. 3.1

$$\omega = \frac{m_1 - m_2}{m_2} \times 100\%$$

$$= \frac{88 - 68}{68} = \frac{20}{68} = 29.4\%$$

The calculations for other slices were done in the same way as above.

Similarly,

The Average MC for Abura = **21.38%**

The Average MC for Afromosia = **24.71%**

The Average MC for Berlinia = **27.27%**

The summary of the calculated moisture contents for the three (3) timber species are given in Table 4.4.

Table 4.4: Summary of Moisture Content Results

Timber Species	Moisture Content, MC (%)		Coefficient of Variation
	Mean	Standard Deviation	
Abura *(Mitragyna ciliata*)	21.38	6.76	0.3161
Afromosia (*Afromosia elata*)	24.71	5.67	0.230
Berlinia(*Berlinia/confusa grandiflora*)	27.27	11.63	0.426

4.1.1 Analysis of Variance for Moisture content of the selected species

Two- way analysis of variance (2-way ANOVA) was employed here. The test examines the influence of different categorical independent variables on one dependent variable. It is thus used here to compare the means of more than two different samples. The different types of samples were considered to be the three selected timber species. Therefore, the mean values of moisture contents were compared within and between group of species. Thus, this is a simple experiment with two factors; the variability of MC within and between groups.

The results of the two-way ANOVA for moisture content of the selected species obtained are shown in the Tables 4.5 to 4.7

Table 4.5: ANOVA Results for Moisture content of the Selected Timber Species (within group)

Test slice No	No. of sample within group, n	Mean (%)	Standard error (SE) (%)	Standard deviation(SD) (%)
01	3	40.797	7.1564	12.395
02	3	20.300	5.0451	8.738
03	3	27.277	1.3556	2.348
04	3	29.290	8.0800	13.995
05	3	23.333	3.3333	5.774
06	3	27.900	0.4600	0.797
07	3	25.823	9.5649	16.567
08	3	16.443	1.6036	2.777
09	3	25.210	3.7800	6.547
10	3	25.737	3.4933	6.051
11	3	19.970	3.1052	5.378
12	3	16.583	0.9397	1.628
13	3	20.503	1.3053	2.261
14	3	25.767	0.3833	0.664
15	3	21.810	0.9050	1.568

Table 4.6: ANOVA Results for Moisture content of the selected Timber Specie (between groups)

Groups	No. of sample between Groups, n	Mean Value (%)	Standard error (%)	Standard deviation (%)
Mitragyna ciliata	15	21.378	1.7446	6.757
Afromosia elata	15	24.703	1.4649	5.674
Confusa grandiflora	15	27.267	3.0017	11.626

Table 4.7: ANOVA Results for Moisture content of the selected Timber species (within and between groups)

Source of Variation	Sum of Squares (SS) (%)	Degree of Freedom (DF)	Mean square (%)	F-statistic	P-value
Slice No.	1510.384	14	107.885	2.05	0.0512
Groups	261.581	2	130.790	2.49	0.1012
Residual	1471.570	28	52.556		
Total	3243.534	44			

The variability of the selected timber species within each specie group as well as between groups are shown in Tables 4.5 and 4.6 which are the results of two-way ANOVA performed. On the other hand, the results of the mean square values of MCs for within and between groups of species are presented in Table 4.7. The table gave the sample variance which is the ratio of SS and DF. The SE presented in Tables 4.5 and 4.6 measure the variability of the mean. A hypothesis or statement describing a situation where no effect is observed is referred to as null hypothesis. This, therefore, implies that the value of 0.0512 which is the p-value in Table 4.7 and as well the probability of F-statistic within the species group since it was found to be greater than 0.05, that is , the confidence level, the null hypothesis is acceptable for the variability within the species group. The same applies to the p-value of 0.1012; the null hypothesis is accepted since the value is greater than 0.05 confidence level for variability between groups of species.

4. 2 RESULTS OF WET DENSITY

The summary of the timber densities for the three (3) selected timber species are given in Table 4.8. The detailed calculations on density are given Appendix A.

Table 4.8 Summary of Wet Density Results

Timber Species	Timber Density (kg/m^3)		Coefficient of Variation
	Mean	Standard Deviation	
Abura *(Mitragyna ciliata*)	544.73	33.68	0.062
Afromosia (*Afromosia elata*)	544.27	24.84	0.047
Berlinia(*Berlinia/confusa grandiflora*)	525.71	15.28	0.029

The characteristic wet density values obtained using eqn. (3.4) for the three timber specie are given in Table 4.9

Table 4.9 Characteristic Wet Density of the Timber Specie

S/ NO	SPECIES	MEAN DENSITY ($\bar{\rho}$) (kg/m^3)	S. DEVIATION (s)(kg/m^3)	5th PERCENTILE $\rho_{05} = \left(\bar{\rho} - 1.65s\right)$ (kg/m^3)
1	Abura *(Mitragyna ciliata*)	544.73	33.68	489.2
2	Afromosia (*Afromosia elata*)	544.27	24.84	503.3
3	Berlinia(*Berlinia/confusa grandiflora*)	525.71	15.28	500.5

The corrected wet density values were obtained using eqn. (3.6) and are given in Table 4.10

Table 4.10: Corrected Characteristic Wet Densities to 12% and 18% Moisture Contents.

Species	ρ_w (kg/m^3)	$W(\%)$	$\rho_{k,12}=\rho_w*\left(1-\frac{(1-k)*(w-12)}{100}\right)$ (kg/m^3)	$\rho_{k,18}=\rho_w*\left(1-\frac{(1-k)*(w-18)}{100}\right)$ (kg/m^3)
Abura *(Nitragyra ciliata*)	489.2	21.38	466	481
Afromosia (*Afromosia elata*)	503.3	24.71	480	486
Berlinia(*Berlinia/confusa grandiflora*)	500.5	27.27	462	477

It is observed from Table 4.10 that the characteristic values of density decrease with decrease in moisture content.

4.2.1 Analysis of Variance for Density of Timber Specie (within group)

Table 4.11 to 4.13 present the ANOVA results for Density for the selected timber specie.

Table 4.11: ANOVA Results for Density of the Selected Timber Species (within group)

Test slice No	No. of sample within group, n	Mean(kg/m^3)	SE(kg/m^3)	SD(kg/m^3)
01	3	569.0	21.96	38.0
02	3	495.3	2.33	4.0
03	3	557.7	17.64	30.6
04	3	513.0	0.00	0.0
05	3	525.3	12.67	21.9
06	3	569.3	13.67	23.7

Test slice No.	No. of sample within group, n	Mean (kg/m^3)	SE(kg/m^3)	SD(kg/m^3)
07	3	526.7	10.04	17.4
08	3	532.0	4.04	7.0
09	3	566.3	19.67	34.1
10	3	540.3	17.67	30.6
11	3	537.7	2.33	4.0
12	3	531.0	5.86	10.1
13	3	546.3	9.33	16.2
14	3	529.7	4.33	7.5
15	3	531.3	6.67	11.5

Table 4.12: ANOVA Results for Density of the selected Timber Specie (between groups)

Groups	No. of sample between Groups, n	Mean Value(kg/m^3)	SE(kg/m^3)	SD(kg/m^3)
Mitragyna Ciliata	15	544.7	8.70	33.7
Afromosia Elata	15	544.3	6.41	24.8
Confusa Grandiflora	15	525.2	3.84	14.9

Table 4.13: ANOVA Results for Density of the selected Timber species (within and between groups)

Source of Variation	Sum of Squares (SS) (kg/m^3)	Degree of Freedom (DF)	Mean square (kg/m^3)	F-statistic	P-value
Slice No.	18421.5	14	1315.8	4.01	0.0009
Groups	3726.5	2	1863.3	5.68	0.0085
Residual	9186.8	28	328.1		
Total	31334.8	44			

Since p-value of 0.0009 for variability within specie group is less than the confidence level of 0.05, then the null hypothesis is rejected in this case. Again, p-value of 0.0085 is also

less than the confidence level and this lead to the rejection of the null hypothesis for the variability within groups of the timber species.

4.3 RESULTS OF BENDING STRENGTH OF TEST SPECIE

Detailed results of bending strengths for the three timber species used for both four and three point bending tests are given in Appendix B. The summary the descriptive statistics for the bending strength is given in Table 4.14. The 5^{th} percentiles of the bending strength for the timber species were obtained using equations (3.8, 3.9 and 3.10) and are given in Table 4.15

Table 4.14: Descriptive Statistic Results for Bending strength tests

Timber specie / Bending method		Bending strength(N/mm^2)			Coefficient of variation
		Mean	S.D (s)	Fractile f_{05}	
Mitragyna ciliata	4 Point bending	69.64	2.73	64.2	0.03926
	3 point bending	92.28	3.01	86.65	0.03263
Afromosia elata	4 point bending	97.51	3.75	90.9	0.03844
	3 point bending	129.31	5.35	119.09	0.04136
Confusa grandiflora	4 point bending	58.51	2.11	54.3	0.03605
	3 point bending	70.71	2.91	66.33	0.04114

Table 4.15: 5th Percentile of the Bending Strength of the three Timber Species

S/NO	SPECIE OF TIMBER	5th PERCENTILE VALUE f_{05} (N/mm^2)
1	*Mitragyna ciliata* 4 point test	64.2
2	*Mitragyna* 3 point test	86.7
3	*Afromosia elata* 4 point test	90.9
4	*Afromosia elata* 3 point test	119.2
5	*Confusa grandiflora* 4 point test	54.3
6	*Confusa grandiflora* 3 point test	66.3

From Table (4.14) the mean values of bending strengths for *Mitragyna ciliata*, *Afromosia elata* and *Confusa grandiflora* corresponding to four point bending test are 69.64 N/mm^2, 97.51 N/mm^2 and 58.51 N/mm^2 respectively. While the corresponding coefficient of variations are 0.03926, 0.03263 and 0.03605 respectively. Considering three -point bending test, the corresponding mean values of bending strengths for the three timber species are: 92.28 N/mm^2 *for Mitragyna ciliata*, 129.31 N/mm^2 for *Afromosia elata* and 70.71 N/mm^2 for *Confusa grandiflora*. The corresponding coefficients of variations are: 0.03263, 0.04136 and 0.04114 respectively.

Table 4.16 The results of adjustments for bending strength of the three timber species using Eqn. (3.11)

Table 4.16: Adjustment of the Bending Strength for the 3 Timber Specie

Timber specie	Average h(mm)	5th percentile of bending (N/mm^2)	$l_{es} = 18h$	$l_{et} = 900$	$k_h = \left(\frac{150}{h}\right)^{0.2}$	$k_l = \left(\frac{l_{es}}{l_{et}}\right)^{0.2}$	Corrected bending strength (N/mm^2)
Mitragyna ciliata 4 point test	74.6	64.2	1342.3	900	1.15	1.08	51.5
Mitragyna ciliata 3 point pt	74.6	86.7	1342.3	900	1.15	1.08	69.6

Timber species	Average h (mm)	5^{th} percentile of bending	$l_{es} = 18h$	$l_{et} = 900$	$k_h = \left(\frac{150}{h}\right)^{0.2}$	$k_l = \left(\frac{l_{es}}{l_{et}}\right)^{0.2}$	Corrected bending strength (N/mm^2)
Afromosia elata 4 point	74.6	90.9	1342.3	900	1.15	1.08	73
Afromosia elata 3 point	74.6	119.2	1342.3	900	1.15	1.08	95.6
Confusa grandiflora 4 point	74.7	54.3	1342.3	900	1.15	1.08	43.6
Confusa grandiflora 3 point	74.6	66.3	1342.3	900	1.15	1.08	53.3

Table (4.17) shows the values of computed characteristic bending strength using Eqn. (3.12)

Table 4.17: Computed Characteristic Bending Strength

PROPERTY	*Mitragyna ciliata*		*Afromosia elata*		*Confusa grandiflora*	
	4 POINT	3 POINT	4 POINT	3 POINT	4 POINT	3 POINT
CORRECTED BENDING STRENGTH $\bar{f}_{05}$ (N/mm^2)	51.50	69.6	73	95.6	43.6	53.3
CHARACTERISTIC BENDING STRENGTH $f_k = \bar{f}_{05} x\, 1.12$ (N/mm^2)	58	78	82	107	49	60

The characteristic bending strength for the three timber specie at various moisture contents are as given in Table (4.18).

Table 4.18: Characteristic Bending strength Result

TIMBER PROPERTY	*Mitragyna ciliata*		*Afromosia elata*		*Confusa grandiflora*	
	MC (μ =21.38)		MC (μ =24.71)		MC (μ =27.27)	
	4 POINT	3POINT	4 POINT	3 POINT	4 POINT	3 POINT
$f_{m,k}$ (N/mm^2)	58	78	82.2	107	49	60
$f_{m,,12\%}$ (N/mm^2)	80.2	107.8	131.2	171.2	89.2	109.2
$f_{m,18\%}$ (N/mm^2)	64.42	56.65	102.24	88.52	67.44	53.68

4.3.1 Analysis of Variance for Bending Strength of Timber Species (within group)

Tables 4.19 to 4.21 present the ANOVA results for bending strength for the selected timber species

Table 4.19: ANOVA Results for Bending Strength of the Selected Timber Species (within group)

Test slice No	No. of sample within group, n	Mean (N/mm^2)	SE(N/mm^2)	SD(N/mm^2)
01	3	80.310	11.0050	19.061
02	3	87.280	14.1400	24.491
03	3	87.140	13.1200	22.725
04	3	78.693	12.1967	21.125
05	3	83.273	13.4867	23.360
06	3	85.107	12.7033	22.003
07	3	82.920	12.9100	22.361
08	3	85.673	12.9367	22.407
09	3	79.040	12.2200	21.166
10	3	85.400	13.4000	23.209
11	3	84.633	12.9667	22.459
12	3	85.273	12.9367	22.407
13	3	88.060	13.6800	23.694
14	3	80.860	12.6300	21.876
15	3	83.733	12.6667	21.939
16	3	88.647	14.0733	24.376
17	3	83.873	12.7867	22.147
18	3	86.073	13.3367	23.100

Test slice No.	No. of sample within group,n	Mean(N/mm^2)	SE(N/mm^2)	SD(N/mm^2)
19	3	89.627	13.8133	23.925
20	3	82.280	13.3400	23.106

Table 4.20: ANOVA Results for Bending Strength of the selected Timber Specie (between groups)

Groups	No. of sample between Groups, n	Mean Value (N/mm^2)	SE (N/mm^2)	SD (N/mm^2)
Mitragyna Ciliata	20	97.413	0.8211	3.672
Afromosia Elata	20	97.412	0.8213	3.673
Confusa Grandiflora	20	58.360	0.4814	2.153

Table 4.21: ANOVA Results for Bending Strength of the selected Timber species (within and between groups)

Source of Variation	Sum of Squares (SS) (N/mm^2)	Degree of Freedom (DF)	Mean square (N/mm^2)	F-statistic	P-value
Slice No.	541.556	19	28.503	18.34	<0.0001
Groups	20334.376	2	10167.188	6542.44	<0.0001
Residual	59.053	38	1.554		
Total	20934.985	59			

Since p-value of <0.0001 for variability within specie group is less than the confidence level, then the null hypothesis is rejected in this case. Again, p-value of <0.0001 is also less

than the confidence level (0.005) and this leads to the rejection of the null hypothesis for the variability within groups of the timber species.

4.4: RESULTS AND DISCUSSION OF GLOBAL MODULUS OF ELASTICITY IN BENDING

The results of characteristic MOE was based on the MC 12 and 18% as shown in Table (4.22) for both three- and four -point tests.

Table 4.22:Global Modulus of Elasticity Results for the Timber Species (4- point test)

Species	*L (mm)*	L^3 *(10^6)*	*b(mm)*	*h(mm)*	h^3 *(mm^3)*	$4.7bh^3$ (mm^4)	$\frac{(f_2-f_1}{(w_2-w_1)}$ *(N/mm^2)*	$\frac{L^3}{4.7bh^3}$ (m)	$E_{m,g}$ *(kN/mm^2)*
Mitragyana cilita 4 pt	900	729	40.9	74.6	415160.9	79806387	897.90	9.135	8,202
Afromosia elata 4pt	900	729	40.9	74.6	415160.9	79806387	1,043.00	9.135	9,527
Confusa grandiflora 4 pt.	900	729	40.85	74.7	416832.7	80029799	1,068.00	9.109	9,729

The global MOE for the three timber species are given in Table (4.23)

Table 4.23: Global Modulus of Elasticity Results for the Timber Specie (3 point test)

specie	*L(mm)*	*$L^3(mm^3)$*	$\frac{(f_2-f_1}{(w_2-w_1)}$ *(N/mm^2)*	*b (mm)*	*h (mm)*	*$48I(mm^4)$*	$\frac{l^3}{48I}$ (m^{-1})	$E_{mg}=\frac{l^3}{48I}\frac{(f_2-f_1)}{(w_2-w_1)}$ *(kN/mm^2)*
Mitragyana ciliata 3PT	900	729000000	984.41	40.9	74.6	67920329	10.733	10,566
Afromosia elata 3PT	900	729000000	980.35	40.9	74.6	67920329	10.733	10,522
Confusa grandiflora 3PT	900	729000000	980.86	41	74.6	68086394	10.707	10,502

The mean values of global MOE considering both three- and four- point test are given in Table (4.24)

Table 4.24: Mean Value of Global Modulus of Elasticity

S/NO	SPECIE	Emg Value	MEAN VALUE OF Emg $\overline{E}=\left[\sum E_{i/n}\right]1.3-2690$
1.00	ABORA 4 PT	8202	7973
2.00	ABORA 3 PT	10566	11046
3.00	AFRO 4 PT	9527	9695
4.00	AFRO 3 PT	10522	10,989
5.00	BERNI 4 PT	9729	9958
6.00	BERNI 3 PT	10502	10963

The characteristic MOE at 12% and 18% moisture contents using Eqn. (3.20) and (3.21) are given in Table 4.25

Table 4.25 Characteristic Modulus of Elasticity at 12 and 18% Moisture Contents

SPECIE	*Mitragyna ciliata*		*Afromosia elata*		*Confusa grandiflora*	
	4 POINT	3 POINT	4 POINT	3 POINT	4 POINT	3 POINT
$E_{m,k}\ (kN/mm^2)$	7.973	11.046	9.695	10.989	9.958	10.963
μ (%)	21.38	21.38	24.71	24.71	27.27	27.27
$E_{m,12\%} = \frac{E}{1+0.0143(12-\mu)}(kN/mm^2)$	9.208	12.757	11.849	13.429	12.740	14.026
$E_{m,18\%} = \frac{E}{1+0.0143(18-\mu)}(kN/mm^2)$	8.378	11.607	10.724	13.414	11.480	8.263

The tabulated results show that as the moisture content increased the MOE in bending decreased, likewise when the moisture content decreased the MOE in bending increased. This implies that timber attains its full state of stiffness when it is dry.

The descriptive statistic values for MOE obtained from both four- and three -Point Bending test are shown in Table 4.26.

Table 4.26: Mean, Standard Deviation and Coefficient of Variation for MOE

Timber specie	Test Method	MOE(N/mm^2)		Coefficient of Variation
		Mean	Standard Deviation	
Mitragyna Ciliata	4 point	7973	104.54	0.0126
	3 point	11046	25.86	0.00246
Afromosia elata	4 point	9695	64.06	0.00672
	3 point	10989	34.72	0.00331
Confusa grandiflora	4 point	9958	54.63	0.00561
	3 point	10963	88.12	0.0084

4.4.1 Analysis of Variance for MOE of Timber Specie (within group)

Tables 4.27 to 4.29 present the ANOVA results for MOE for the selected timber species

Table: 4.27: ANOVA Results for MOE of the Selected Timber Species (within group)

Test slice No	No. of sample within group, n	Mean (kN/mm^2)	SE (kN/mm^2)	SD (kN/mm^2)
01	3	9.2377	0.49075	0.8500
02	3	9.1627	0.49876	0.8639
03	3	9.1989	0.49481	0.8570
04	3	9.1320	0.50213	0.8697
05	3	9.1847	0.49660	0.8601
06	3	9.2230	0.49258	0.8532
07	3	9.1397	0.50147	0.8686
08	3	9.2360	0.49096	0.8504
09	3	9.1497	0.49557	0.8584
10	3	9.1313	0.50234	0.8701
11	3	9.1473	0.50071	0.8673
12	3	9.2933	0.40004	0.6929
13	3	9.1687	0.49822	0.8629
14	3	9.1223	0.50342	0.8720
15	3	9.1130	0.50429	0.8735

Test slice No	No. of sample within group, n	Mean (kN/mm^2)	SE (kN/mm^2)	SD (kN/mm^2)
16	3	9.1647	0.49876	0.8639
17	3	9.2267	0.49183	0.8519
18	3	9.1987	0.49507	0.8575
19	3	9.1487	0.50039	0.8667
20	3	9.1163	0.50396	0.8729

Table 4.28: ANOVA Results for MOE of the selected Timber Specie (between groups)

Groups	**No. of sample between Groups, n**	**Mean Value** (kN/mm^2)	**SE** (kN/mm^2)	**SD** (kN/mm^2)
Mitragyna Ciliata	20	8.2026	0.01941	0.0868
Afromosia Elata	20	9.5169	0.00912	0.0408
Confusa Grandiflora	20	9.8048	0.00732	0.0328

Table 4.29: ANOVA Results for Modulus of Elasticity of the selected Timber species (within and between groups)

Source of Variation	Sum of Squares (SS) (kN/mm^2)	Degree of Freedom (DF)	Mean square (kN/mm^2)	F-statistic	P-value
Slice No.	0.1359	19	0.0072	4.58	<0.0001
Groups	29.1793	2	14.5897	9348.40	<0.0001
Residual	0.0593	38	0.0016		
Total	29.3745	59			

Since p-value of <0.0001 for variability within species group is less than the confidence level (0.005), then the null hypothesis is rejected in this case. Again, p-value of <0.0001 is also less than the confidence level (0.005) and this leads to the rejection of the null hypothesis for the variability within groups of the timber species.

4.5 ALLOCATION OF TIMBER SPECIES TO STRENGTH CLASSES

From the results presented in Table 4.30, lower values of density, bending strength and MOE were considered as stipulated by EN-338 (2009) for consideration in the allocation of strength classes. Therefore, four -point test results were found to be relatively lower than the corresponding three - point test results, as such, allocation of characteristic values for the three species was based on the four - point test results.

Based on the requirements of EN 338:2009(E), the three timber species can be allocated to a strength class based on the results obtained. Table 4.30 give the values of the grade determining properties which can be used to determine the grade of each of the three timber specie.

Table 4.30 Grade Determining Properties at 12% Moisture content

Timber Species	Density (kg/m^3)	Modulus of Elasticity (kN/mm^2)		Bending Strength (N/mm^2)	
		4-point Bending	3-point Bending	4-point Bending	3-point Bending
Mitragyna ciliata	466	9.208	12.757	80.2	107.8
Afromosia elata	480	11.849	13.429	131.2	171.2
Confusa grandiflora	462	12.740	14.026	89.2	109.2

Mitragyna ciliata was assigned to solid timber strength Class C18 based on its minimum characteristic bending strength of 80.2 N/mm^2, characteristic density of 466 kg/m^3 and minimum mean MOE parallel to grain of 9.2 kN/mm^2 from Table 3.28. The characteristic bending strength, density and mean Modulus of Elasticity parallel to grain for strength Class

C18 from Table 1 of the code EN 338: 2009 (E) are 18 N/mm^2, 320 kg/m^3 and 9 KN/mm^2 respectively. In the same vein, *Afromosia elata* and *Confusa grandiflora* timber species were both assigned to strength Class D_{30}, with characteristic bending strength, density and mean MOE parallel to grain from Table 1 of the code EN 338 : 2009(E) of 30N/mm^2, 530 kg/mm^2, 10 kN/m^3 respectively.

4.5.1: Results of other material properties

Table 4.31 presents the results for other material properties of timber computed from the empirical relationships given by EN 384 (2004). This was done while comparing the results obtained with those in Table 1 of BS EN 338:2003.

Table 4.31: Characteristic Values of Other Material Properties

Other Material Properties	Timber species		
	Mitragyna ciliata (Abura)	*Afromosia elata* (Afromosia)	*Confusa grandiflora* (Berlinia)
Tension Parallel $f_{t,0,k}$ (N/mm^2)	48.12	78.72	53.52
Tension Perpendicular $f_{t,90,k}$ (N/mm^2)	0.6	0.6	0.6
Compression Parallel $f_{,c,0,k}$ (N/mm^2)	35.96	57.27	47.22
Compression Perpendicular $f_{,c,90,k}$ ((N/mm^2)	3.30	3.36	3.2
Shear Strength $f_{v,k}$ (N/mm^2)	3.8	3.8	3.8

5% MOE Parallel $E_{0,05}$ (KN/mm^2)	5.3	6.5	6.7
Other Material Properties	*Mitragyna ciliata* (Abura)	*Afromosia elata* (Afromosia)	*Confusa grandiflora* (Berlinia)
Mean MOE Perpendicular $E_{90,mean}$ (KN/mm^2)	0.27	0.32	0.33
Mean Shear Modulus G_{mean} (KN/mm^2)	0.5	0.61	0.62
Mean Density ρ_{mean} (kg/m^3)	545	544	526

Table 4.32 presents the strength and strength related properties of the three (3) selected timbers extracted from table 1 of EN 338(2009).

Table 4.32: Properties of timber in strength Classes

SPECIES	***Mitragyna ciliata* (Abura)**	***Afromosia elata* (Afromosia)**	***Confusa grandiflora* (Berlinia)**
Strength classes	**C18**	**D30**	**D30**
Strength Properties (N/mm^2)			
Bending Strength, $f_{m,k}$	18	30	30
Tension Strength Parallel to grain, $f_{t,0,k}$	11	18	18
Tension Strength Perpendicular to grain, $f_{t,90,k}$	0.5	0.6	0.6
Compression Strength Parallel to grain, $f_{,c,0,k}$	18	23	23

Compression Strength Perpendicular to grain, $f_{,c,90,k}$	2.2	8.0	8.0
Shear Strength. $f_{v,k}$	2.0	3.0	3.0
SPECIES	***Mitragyna ciliata* (Abura)**	***Afromosia elata* (Afromosia)**	***Confusa grandiflora* (Berlinia)**
Strength classes	**C18**	**D30**	**D30**
Stiffness Properties (kN/mm^2)			
Mean Modulus of Elasticity Parallel to grain, $E_{0,mean}$	9	10	10
5% Fractile value of Modulus of Elasticity in bending, $E_{0,05}$	6.0	8	8.0
Mean Modulus of Elasticity Perpendicular to grain, $E_{90,mean}$	0.30	0.64	0.64
Mean Shear Modulus, G_{mean}	0.56	0.6	0.6
Density (kg/m^3)			
Characteristic Density, ρ_k	320	530	530
Mean density, ρ_{mean}	380	640	640

4.6: RESULTS OF TEST OF GOODNESS OF FIT

The properties of a physical process generally suggest the form of its required distribution. If a process is composed of the sum of many individual effects, the Normal or Lognormal distribution may be appropriate. On the other hand, if the extreme conditions of a physical process are of interest, an extreme value distribution such as Gumbel or Weibull may be a suitable model. Density, strength and stiffness properties of timber have been modelled with normal, lognormal and extreme value distributions (JCSS, 2006).

In this study, three distributions were used to check data fitness for grade determining properties using " Fit distribution " command in EasyFit (2010). However, the plots of the probability density functions are superimposed on histograms for density, bending strength and MOE and are presented in Figs. 4.1 to 4.27.

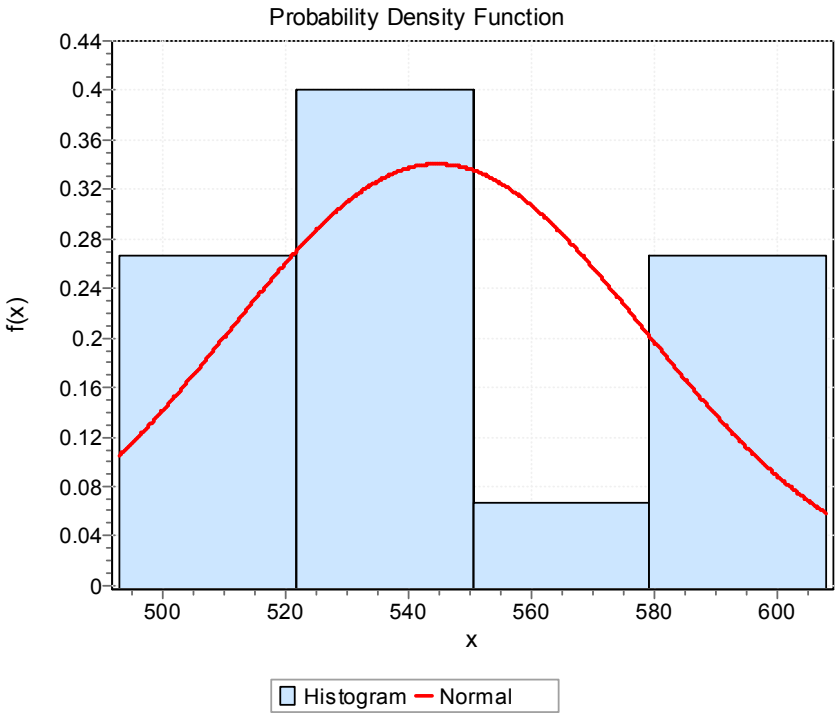


Fig. 4.1 Probability Density Function for *Mitragyna ciliata* considering Density as Normal (x is the density)

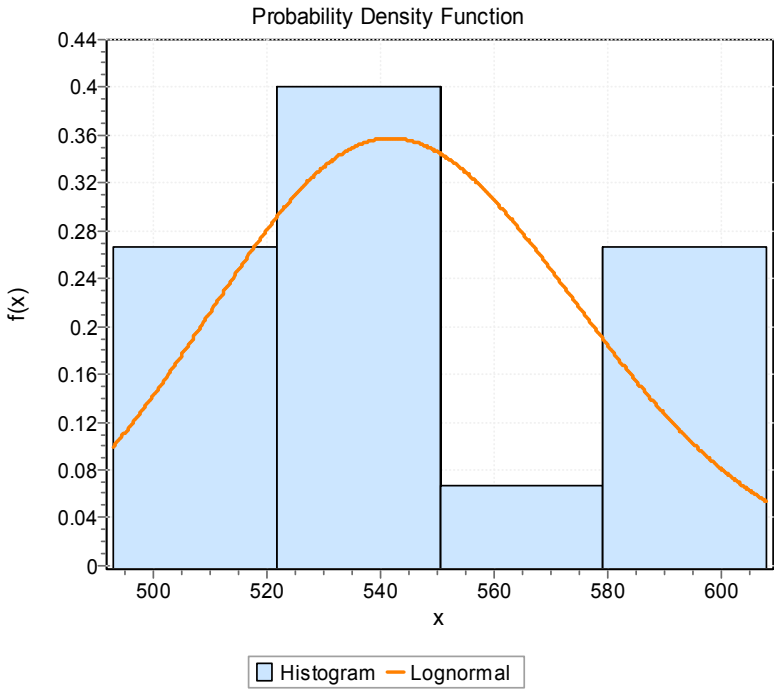


Fig. 4.2 Probability Density Function for *Mitragyna ciliata* considering Density as Lognormal (x is the density)

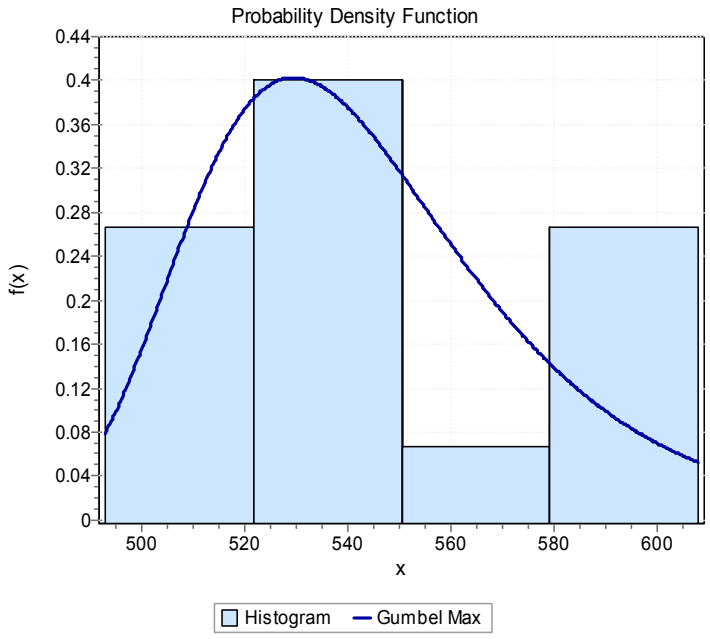


Fig. 4.3: Probability Density Function for *Mitragyna ciliata* considering density as Gumbel (x is the density)

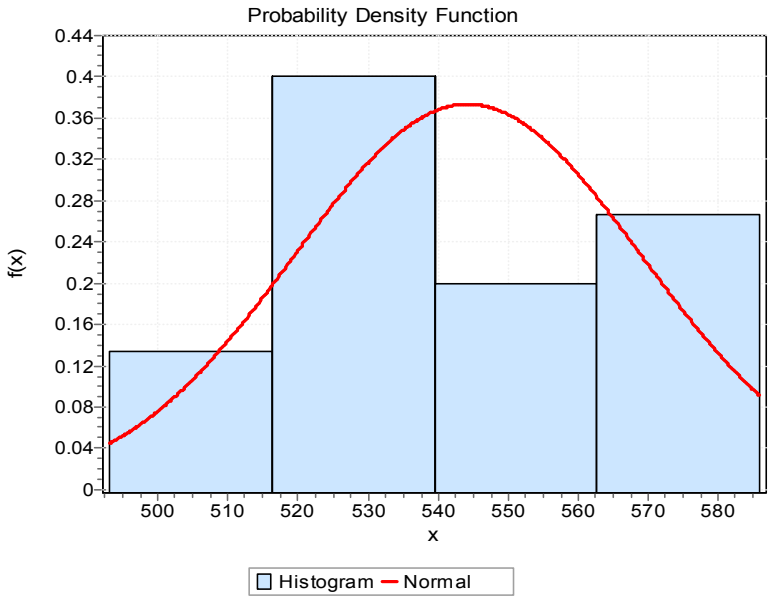


Fig. 4.4: Probability Density Function for *Afromosia elata* considering density as Normal

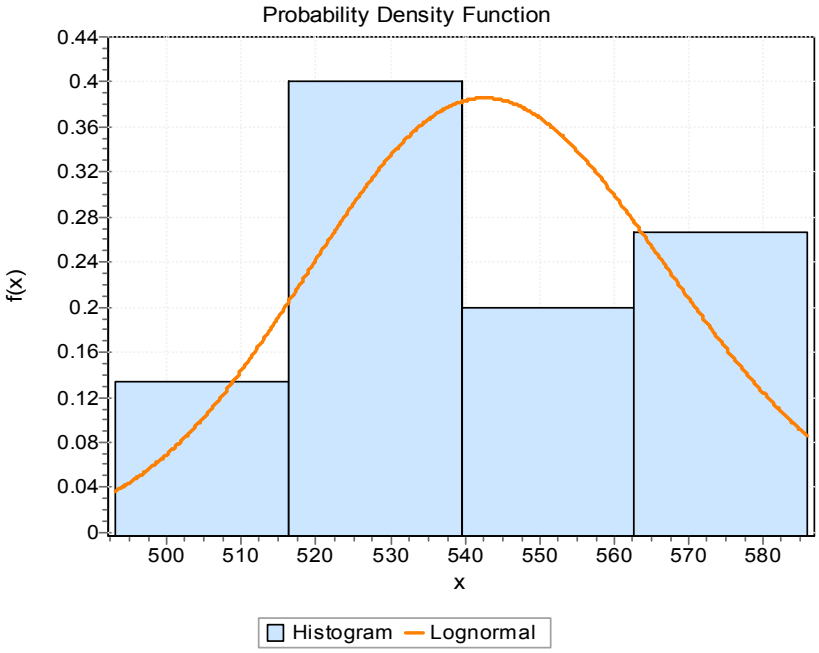


Fig. 4.5: Probability Density Function plot for *Afromosia elata* considering density as Lognormal (x is the density)

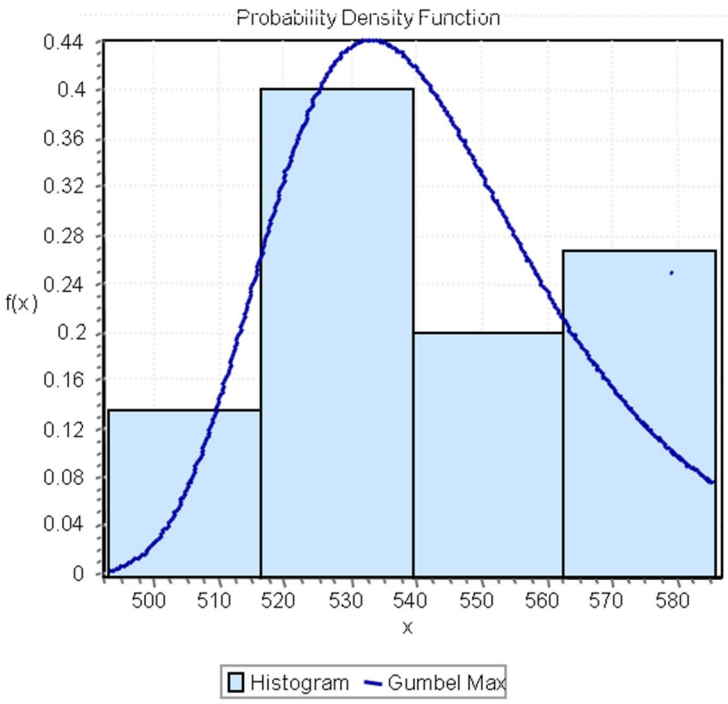


Fig. 4.6: Probability Density Function for *Afromosia elata* considering density as Gumbel (x is the density)

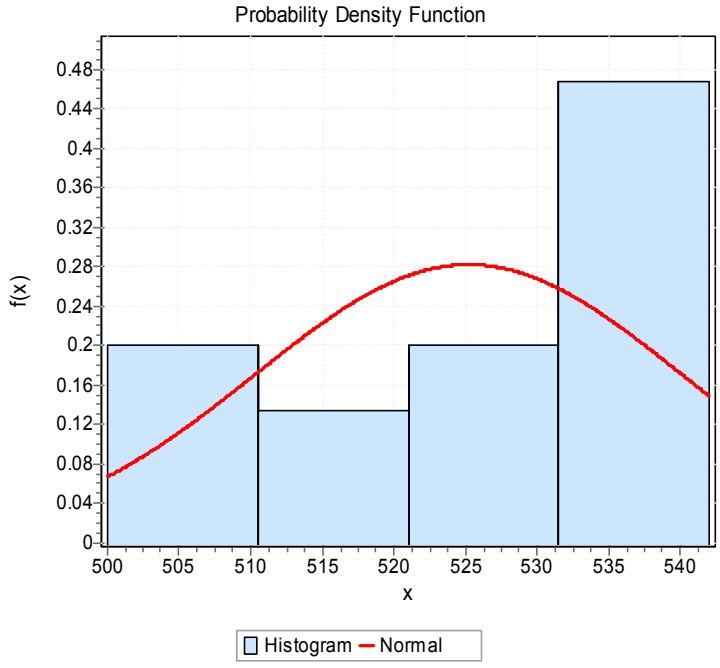


Fig. 4.7: Probability Density Function for *Confusa grandiflora* considering density as Normal (x is the density)

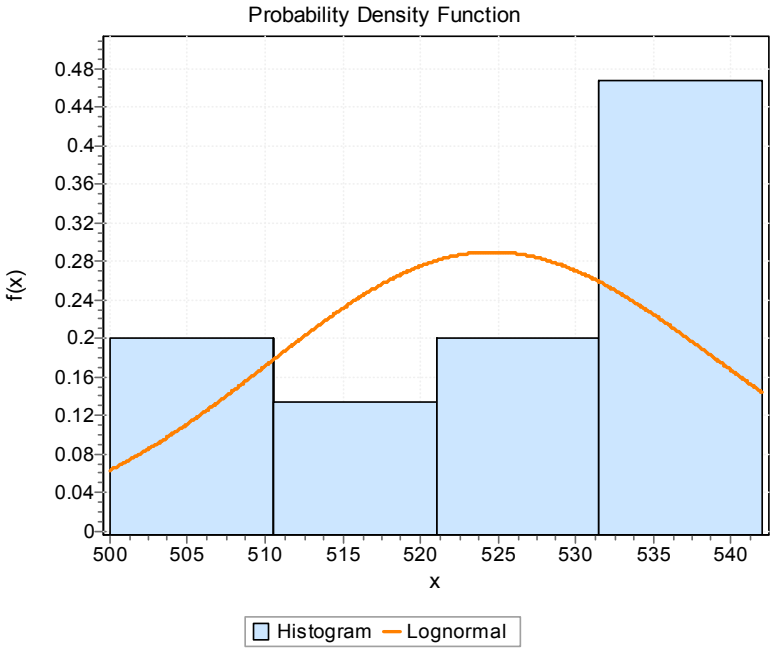


Fig. 4.8: Probability Density Function for *Confusa grandiflora* considering density as Lognormal (x is the density)

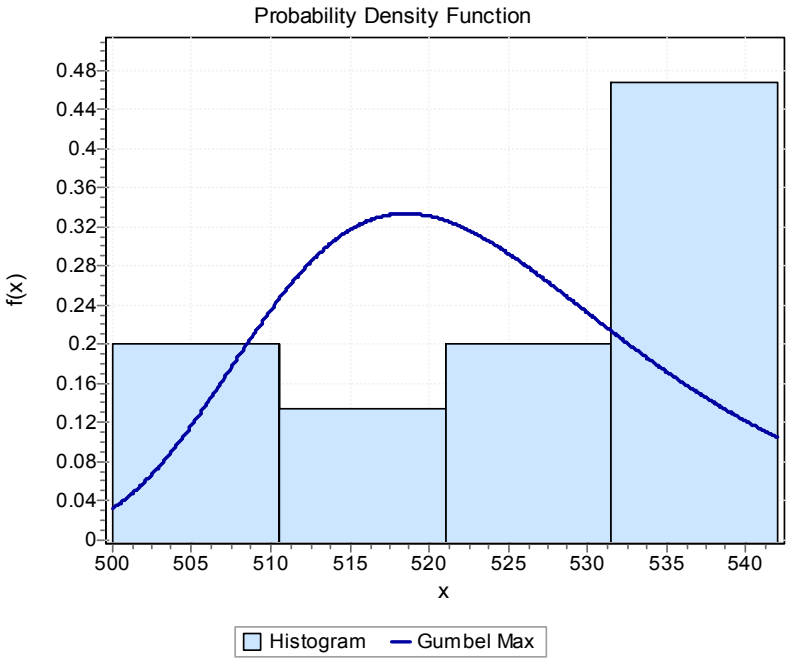


Fig. 4.9: Probability Density Function for *Confusa grandiflora* considering density as Gumbel (x is the density)

The plots shown in Figs, 4.1 – 4.9 probability density functions which constitute the possible values for density of the selected timber species.

In similar way, the random variables and the corresponding probabilities of the selected timber species for MOE and bending strength in Figs. (4.10) to (4.17) and (4.18) to (4.27) were also shown in the horizontal and vertical axes respectively.

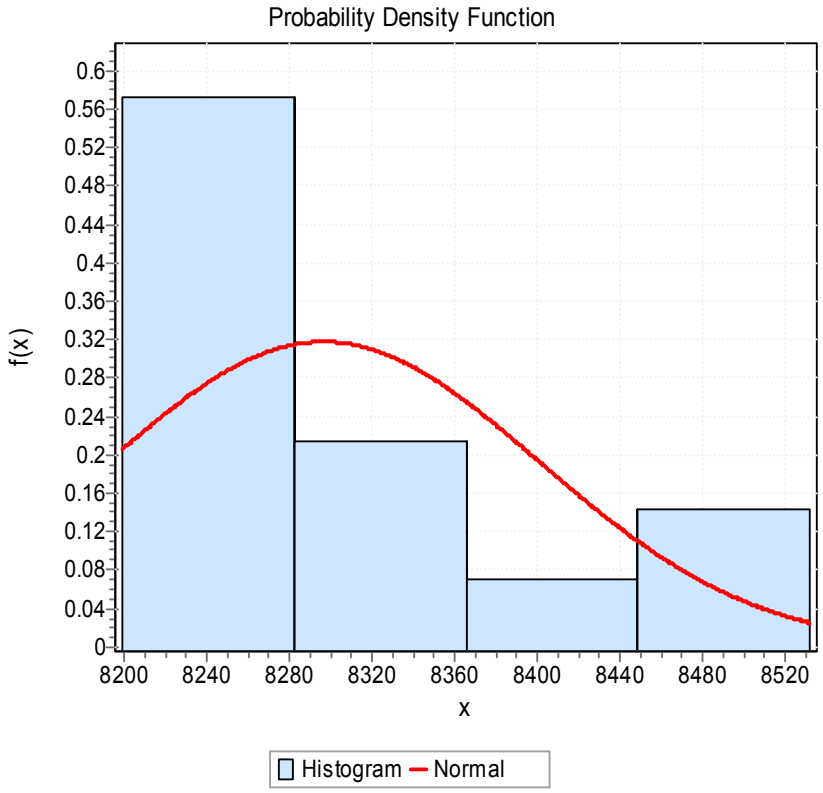


Fig. 4.10: Probability density function for *Mitragyna ciliata* considering MOE of Timber as Normal (x is for MOE)

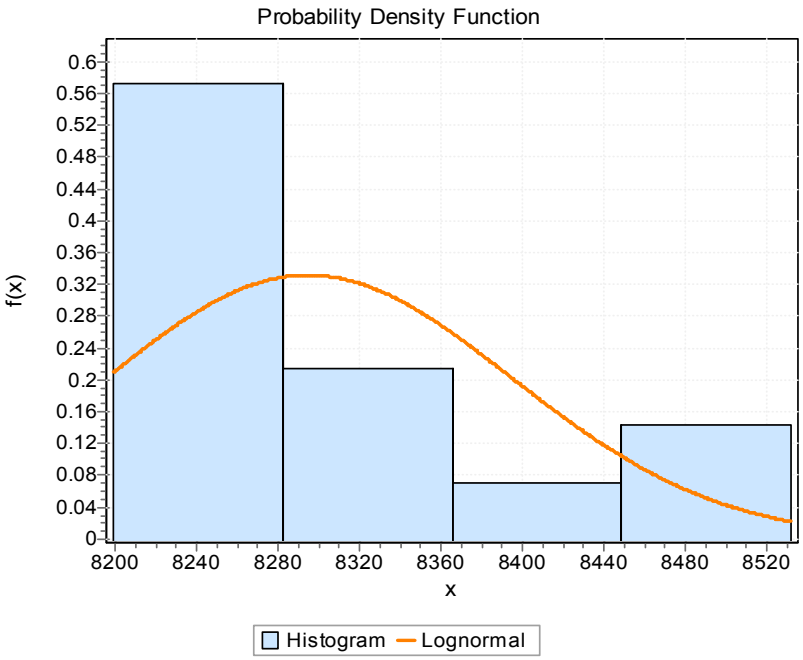


Fig. 4.11: Probability density function for *Mitragyna ciliata* considering MOE of Timber as Lognormal (x is for MOE)

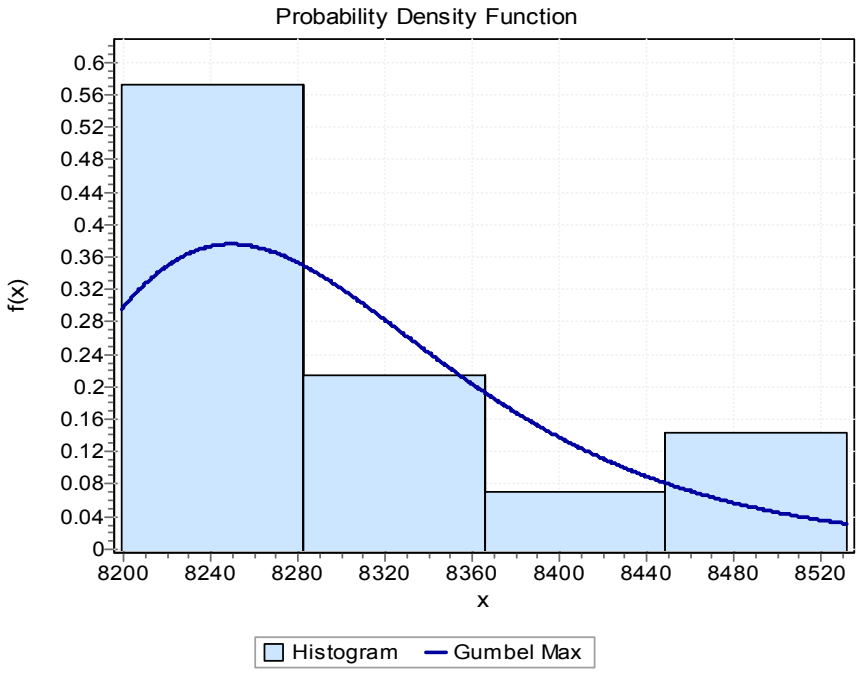


Fig. 4.12: Probability density function for *Mitragyna ciliata* considering MOE of Timber as Gumbel (x is for MOE)

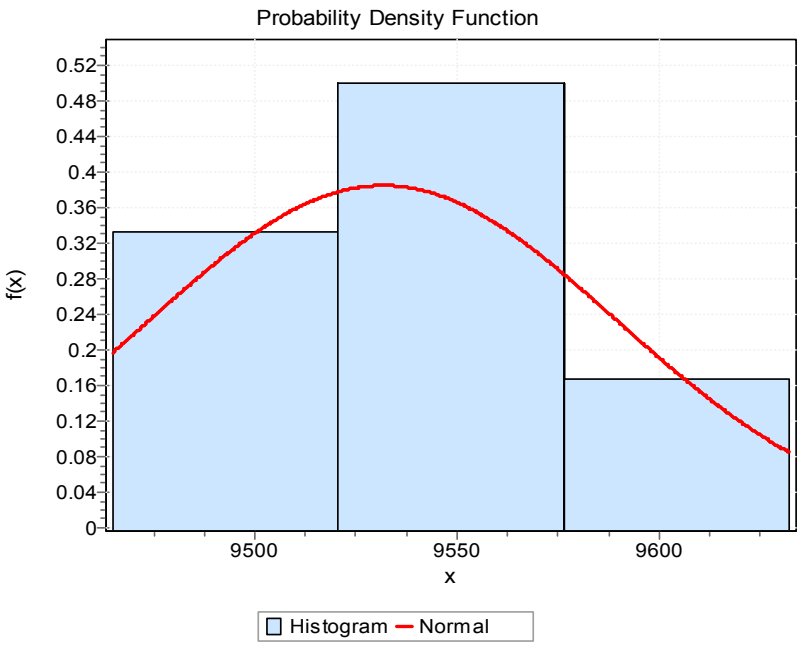


Fig. 4.13: Probability density function plot for *Afromosia elata* considering MOE of Timber as Normal

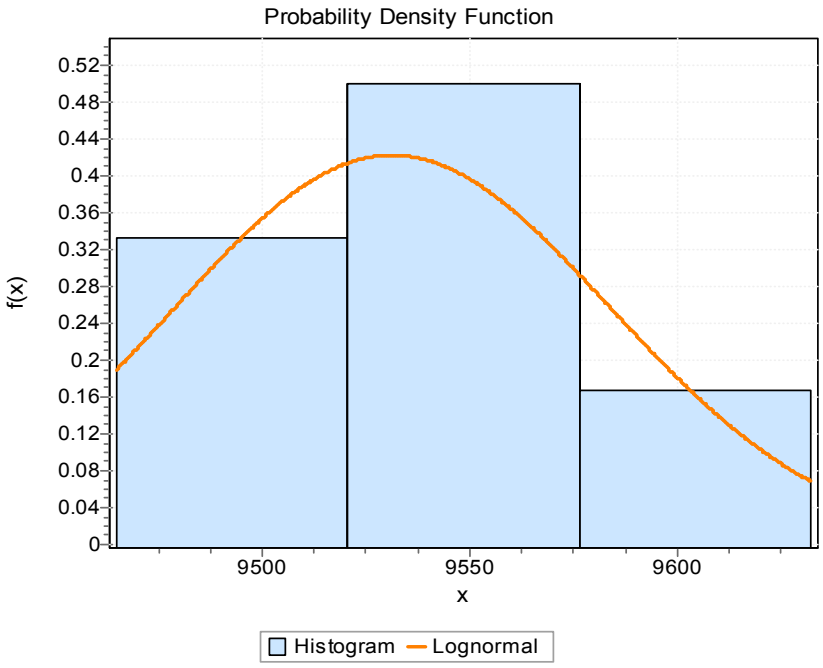


Fig. 4.14: Probability density function for *Afromosia elata* considering MOE of Timber as Lognormal (x is for MOE)

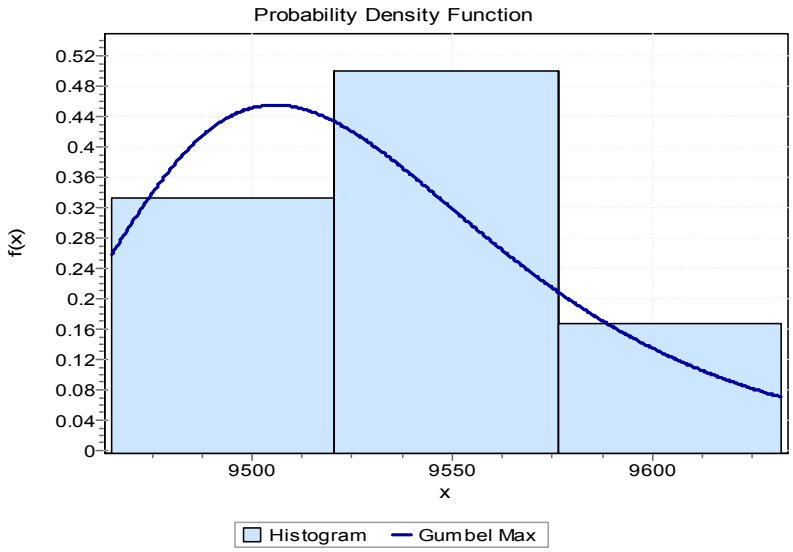


Fig. 4.15: Probability density function for *Afromosia elata* considering MOE of Timber as Gumbel (x is for MOE)

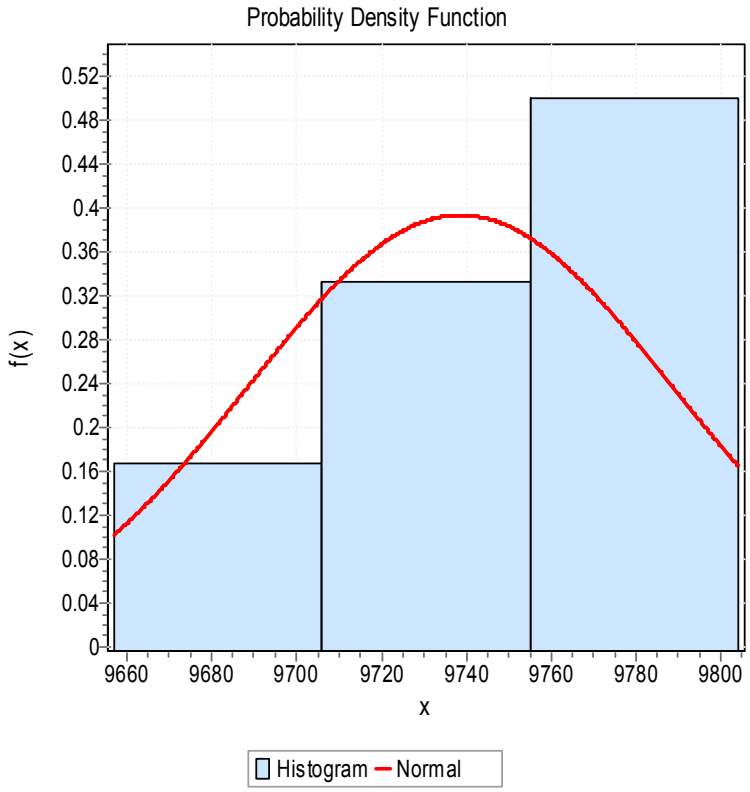


Fig. 4.16: Probability density function plot *Confusa grandiflora* considering MOE of Timber as Normal (x is for MOE)

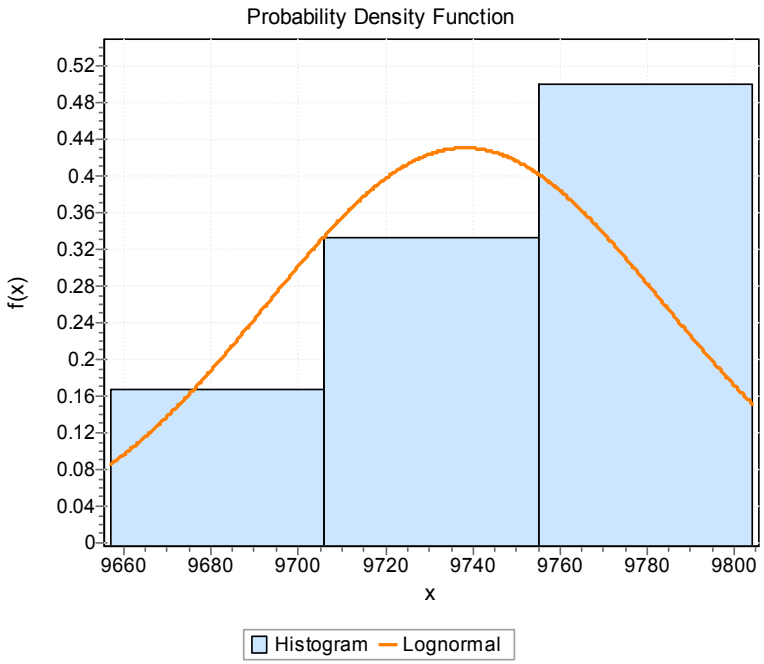


Fig. 4.17: Probability density function plot *Confusa grandiflora* considering MOE of Timber as Lognormal (x is for MOE)

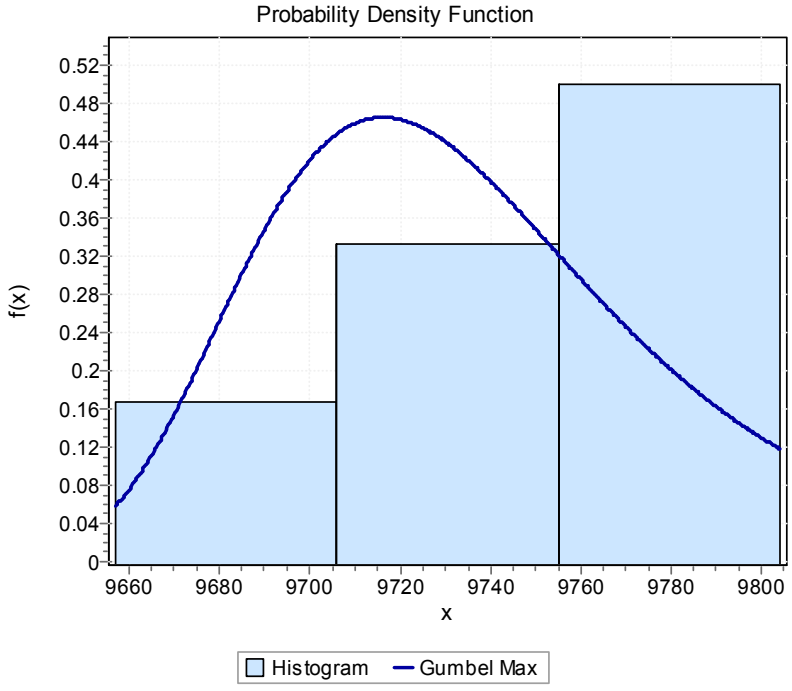


Fig. 4.18: Probability density function *Confusa grandiflora* considering MOE of Timber as Gumbel (x is for MOE)

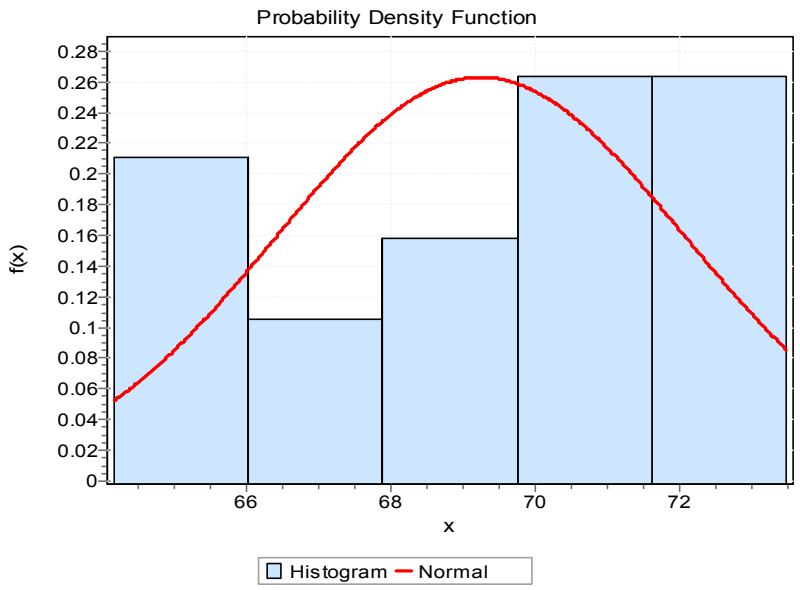


Fig. 4.19: Probability density function for *Mitragyna ciliata* considering bending strength of Timber as Normal (x is for bending strength)

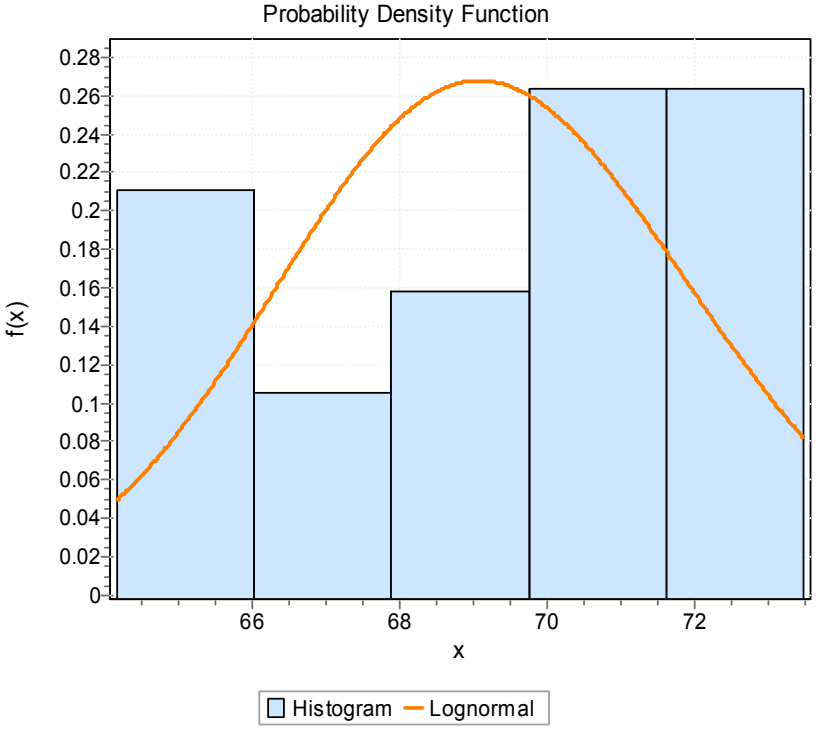


Fig. 4.20: Probability density function for *Mitragyna ciliata* considering bending strength of Timber as Lognormal (x is for bending strength)

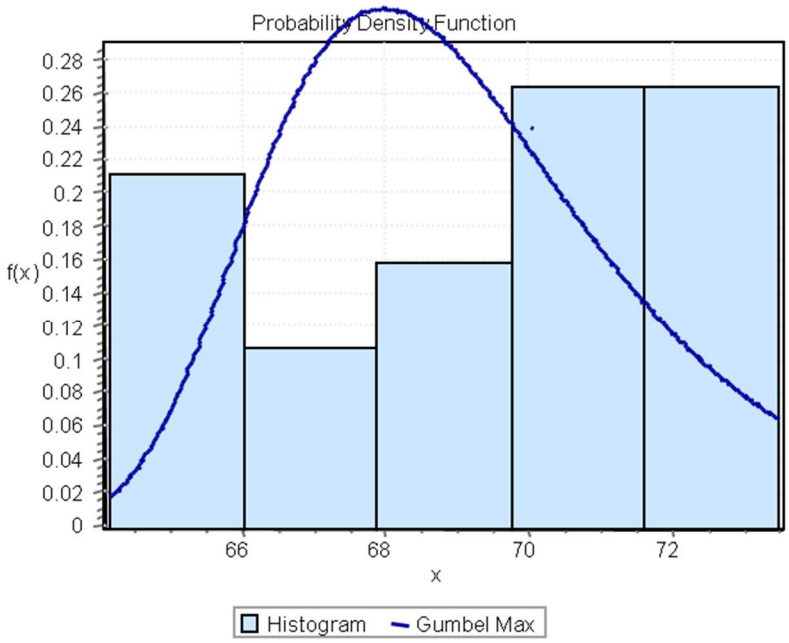


Fig. 4.21: Probability density function for *Mitragyna ciliata* considering bending strength of Timber as Gumbel (x is for bending strength)

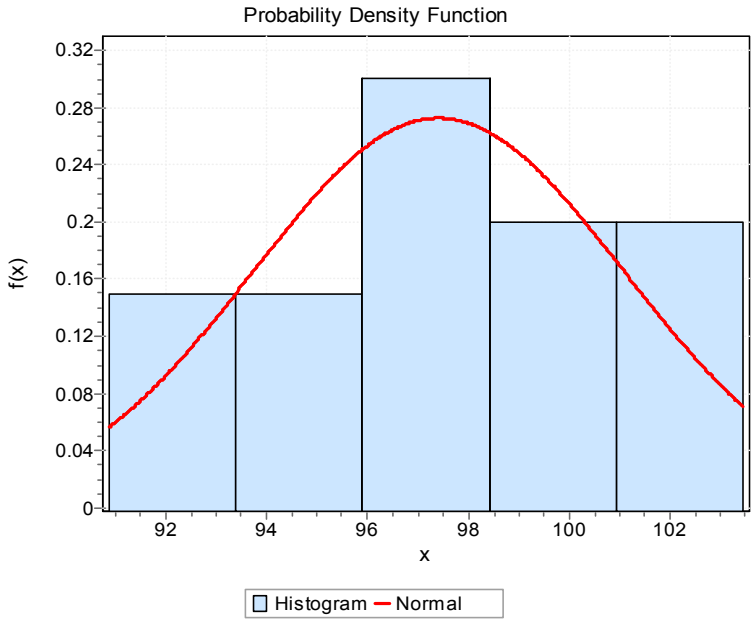


Fig. 4.22: Probability density function for *Afromosia elata* considering bending strength of Timber as Normal (x is for bending strength)

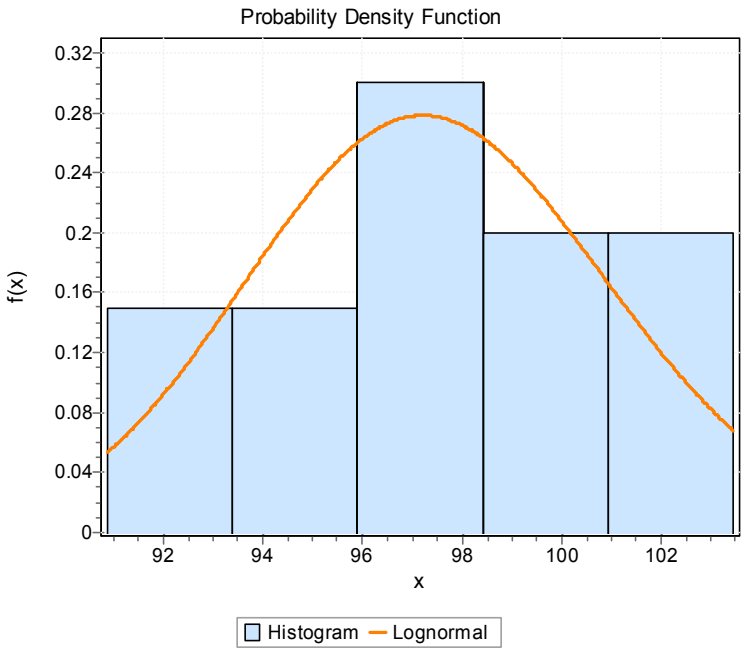


Fig. 4.23: Probability density function for *Afromosia elata* considering bending strength of Timber as Lognormal (x is for bending strength)

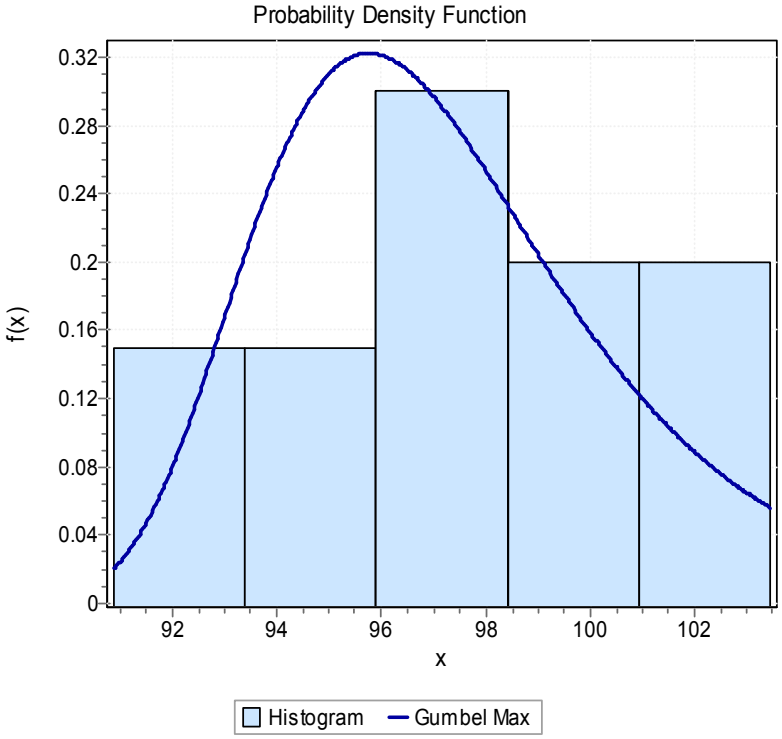


Fig. 4.24: Probability density function for *Afromosia elata* considering bending strength of Timber as Gumbel (x is for bending strength)

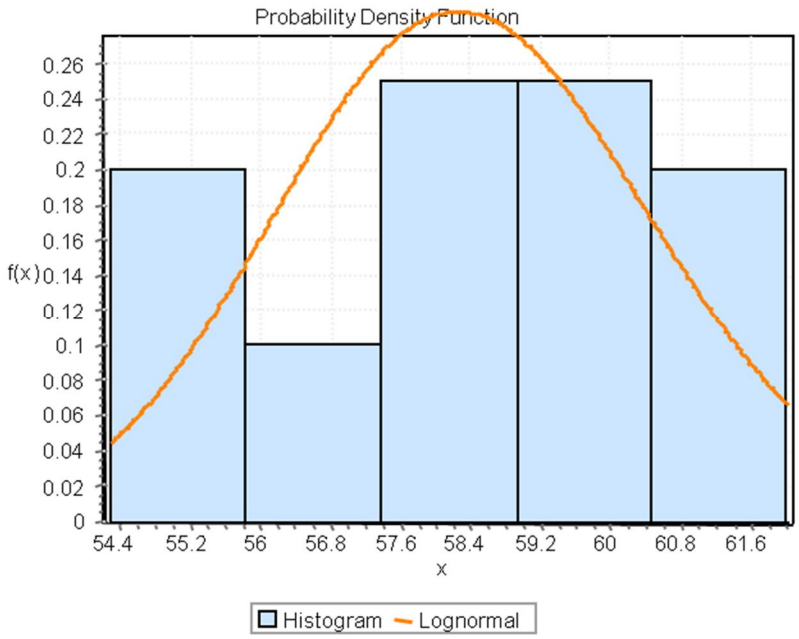


Fig. 4.25: Probability density function for *Confusa grandiflora* considering bending strength of Timber as Normal(x is for bending strength)

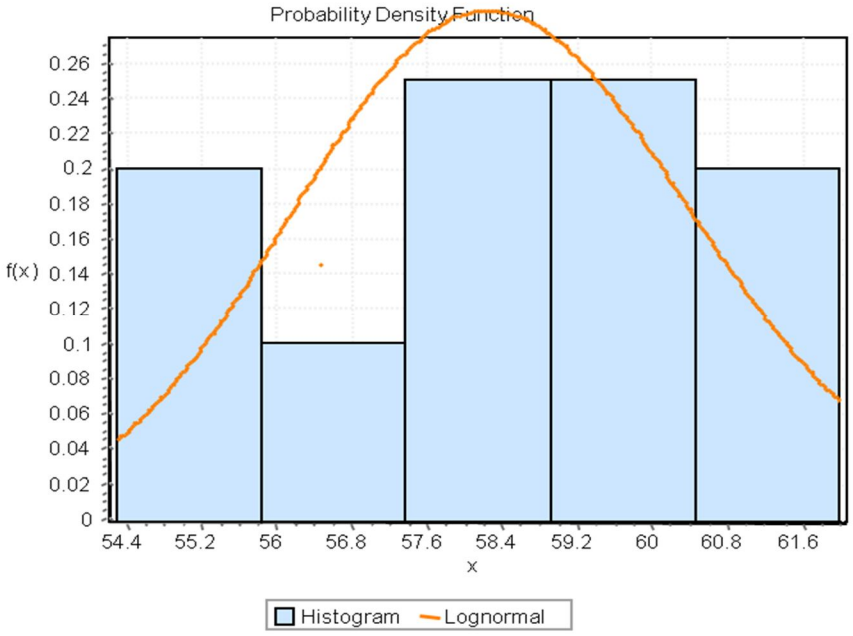


Fig. 4.26: Probability density function for *Confusa grandiflora* considering bending strength of Timber as Lognormal (x is for bending strength)

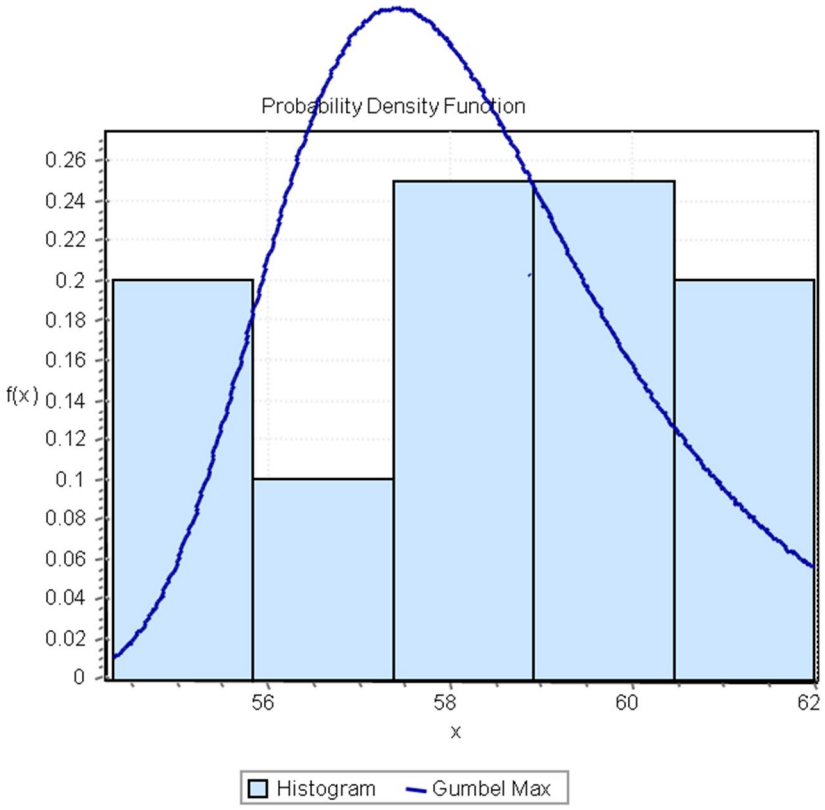


Fig. 4.27: Probability density function for *Confusa grandiflora* considering bending strength of Timber as Gumbel (x is for bending strength)

The results of K-S tests are presented in the form of interactive tables that help to decide the model that describes the data in the best way.

Table 4.33 presents the results of the Kolmogorov¯ Smirnov test results

Table 4.33: Kolmogorov Smirnov Test Results

Timber Property	Distribution type	ABURA (*Mitragyna ciliata*)			AFROMOSIA (*Afromosia elata*)			BERLINIA (*Berlinia/confusa grandiflora*)		
		D	$\alpha_{0.05}$	Rank	D	$\alpha_{0.05}$	Rank	D	$\alpha_{0.05}$	Rank
Density	Normal	0.222	0.338	3	0.177	0.338	1	0.185	0.338	1
	Lognormal	0.214	0.338	2	0.178	0.338	2	0.193	0.338	2
	Gumbel	0.156	0.338	1	0.214	0.338	3	0.245	0.338	3
MOE	Normal	0.182	0.349	2	0.249	0.519	3	0.195	0.519	2
	Lognormal	0.190	0.349	3	0.239	0.519	2	0.184	0.519	1
	Gumbel	0.169	0.349	1	0.203	0.519	1	0.247	0.519	3
Bending Strength	Normal	0.136	0.301	1	0.113	0.294	1	0.189	0.294	1
	Lognormal	0.144	0.301	2	0.114	0.294	2	0.196	0.294	2
	Gumbel	0.206	0.301	3	0.150	0.294	3	0.259	0.294	3

4.6.1: Probability Distribution Models for Grade Determining properties

The corresponding values of D from Table 4.33 at Kolmogorov - Smirnov test, α =0.05 values for density, MOE and bending strength were found to be lower than the reference values of α =0.05 for all the distributions as in the Table 4.33. This indicates that all the distributions can be used to model the three mechanical properties of the timber.

By ranking the three distribution models, the density for *Mitragyna ciliata* will be best modelled using Gumbel distribution, followed by lognormal and normal. The density for *Afromosia elata*, *Confusa grandiflora* follows thus; Normal, lognormal, Gumbel.

For MOE, *Mitragyna ciliata* can be best modelled using Gumbel, Normal then Lognormal. While Afromosia; Gumbel, Lognormal and then Normal. Berlinia can best be modelled using lognormal, Normal then Gumbel.

For Bending Strength, *Mitragyna ciliata*, *Afromosia elata* and *Confusa grandiflora* can best be modelled using Normal, Lognormal then Gumbel.

The least K-S statistics among all the three species for density corresponds to Gumbel distribution; this made it the best theoretical distribution model with the value of 0.156. Figs. 4.28 – 4.30 show the plots for the distributions.

While for MOE, the least K-S statistic for all the three timber specie corresponds also to Gumbel distribution with the value of 0.169, making it the best theoretical distribution model for MOE. Figs. 4.31 – 4.33 show the plots of the distributions.

Finally, the best theoretical model for bending strength for all the three timber specie goes to Normal distribution with K-S statistical value of 0.136. Figs. 4.34 – 4.36 show the plots of the distributions.

The cumulative probability distribution function $p(X \leq x) = \sum_{x_i \leq x} p(x_i)$ is obtained by adding the individual probabilities, where $p(x_i)$ is an individual probability function. The horizontal and vertical axes of the plot comprising of random variable and cumulative distribution function are shown in Figs. 4.28

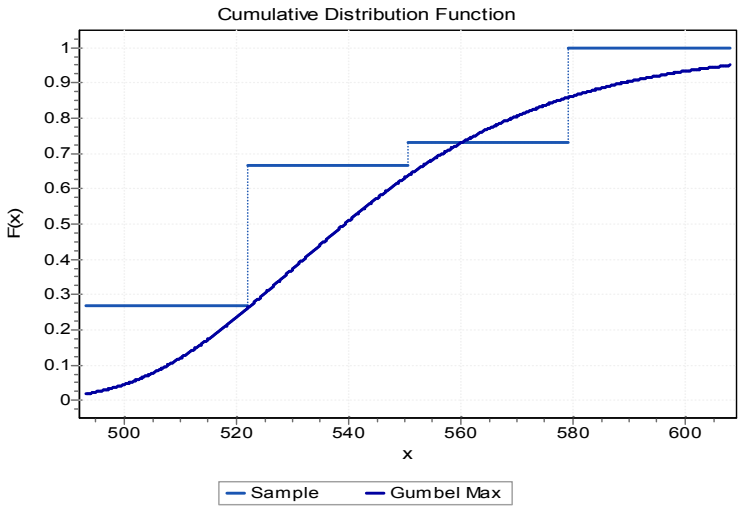


Fig. 4.28: Cumulative Distribution function for *Mitragyna ciliata* considering Density as Gumbel (Experimental) (x is for density)

Fig. 4.29: show the assigned probability distribution model for density of *Mitragyna ciliata* and Fig. 4.30 shows the assigned cumulative distribution model for the density of the selected species. The random variables and the cumulative probability distribution functions for density of timber are shown in the horizontal and vertical axes respectively.

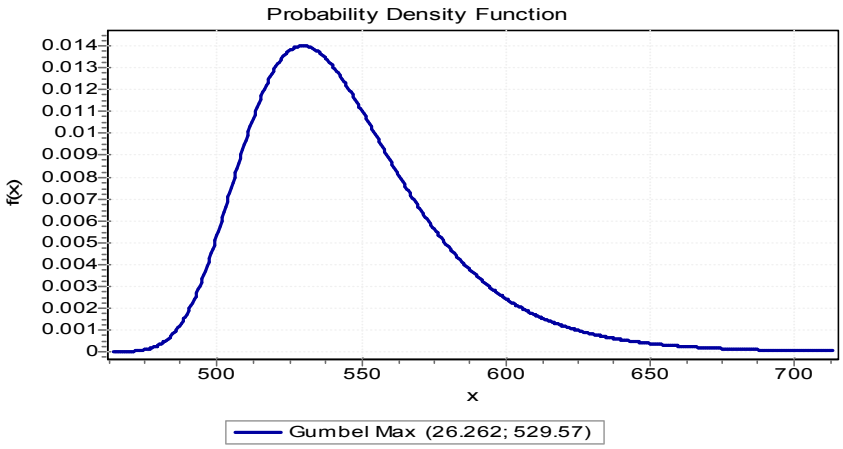


Fig. 4.29: Probability Distribution Model for *Mitragyna ciliata* considering Density as Gumbel (Assigned) (x is for density)

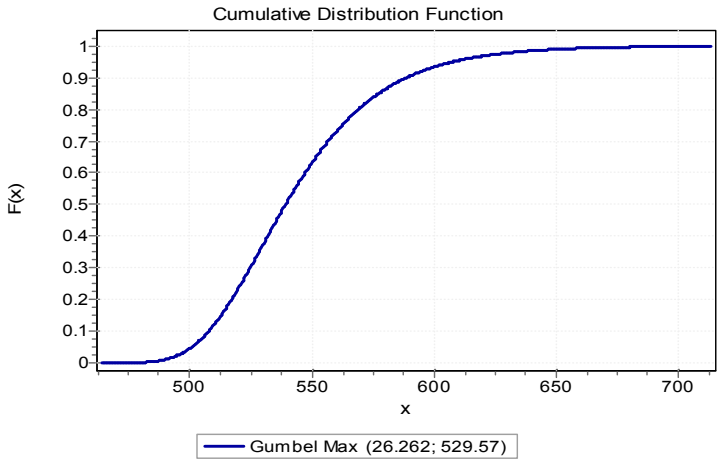


Fig. 4.30: Cumulative Distribution Model for *Mitragyna Ciliata* considering Density as Gumbel (Assigned) (x is for density)

The plots of MOE with Gumbel distribution as the best fit with respect to Cumulative distribution function, probability distribution model and cumulative distribution model following similar procedure with density are shown in figs. 4.31 – 4.33.

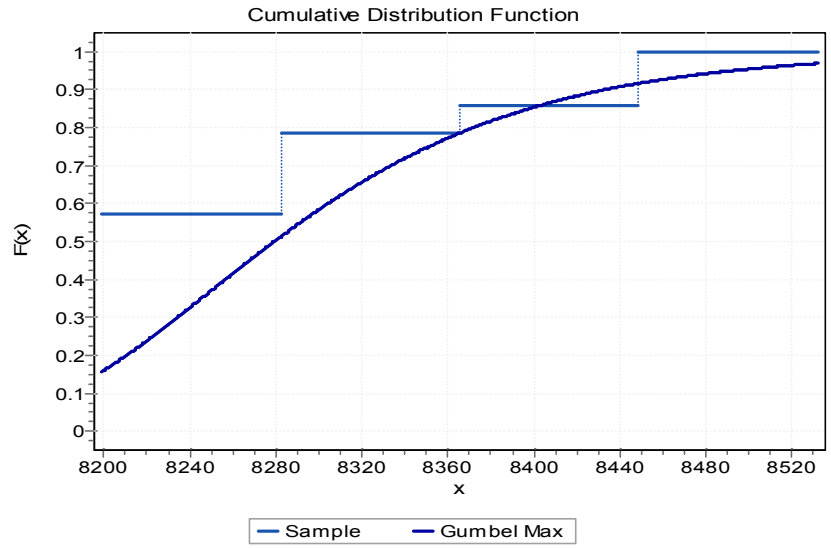


Fig. 4.31: Cumulative Distribution function for *Mitragyna ciliata* considering MOE as Gumbel (Experimental) (x is for MOE)

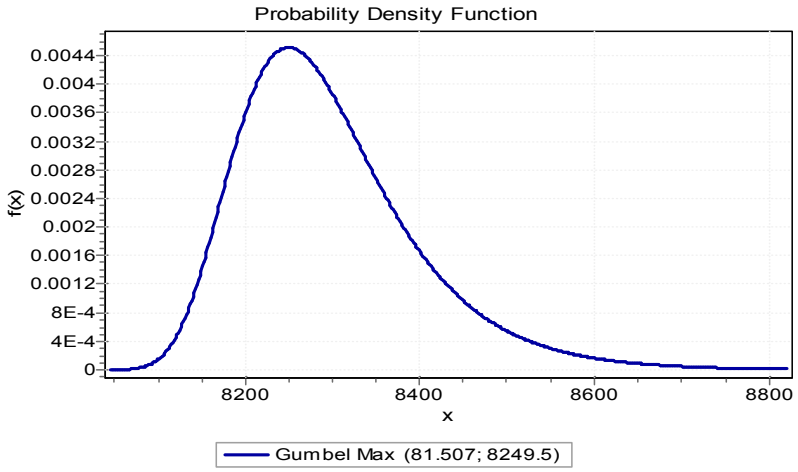


Fig. 4.32: probability Density function for *Mitragyna ciliata* considering MOE as Gumbel (Assigned) (x is for density)

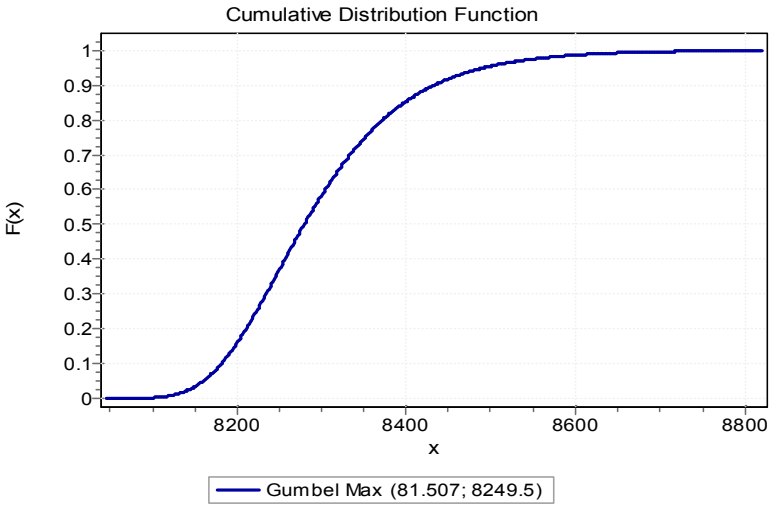


Fig. 4.33: Cumulative Distribution model for *Mitragyna ciliata* considering MOE as Gumbel (Assigned) (x is for density)

The plots of bending strength with normal distribution as the best fit with respect to Cumulative distribution function, probability distribution model and cumulative distribution model following similar procedure with density are shown in figs. 4.34 – 4.36

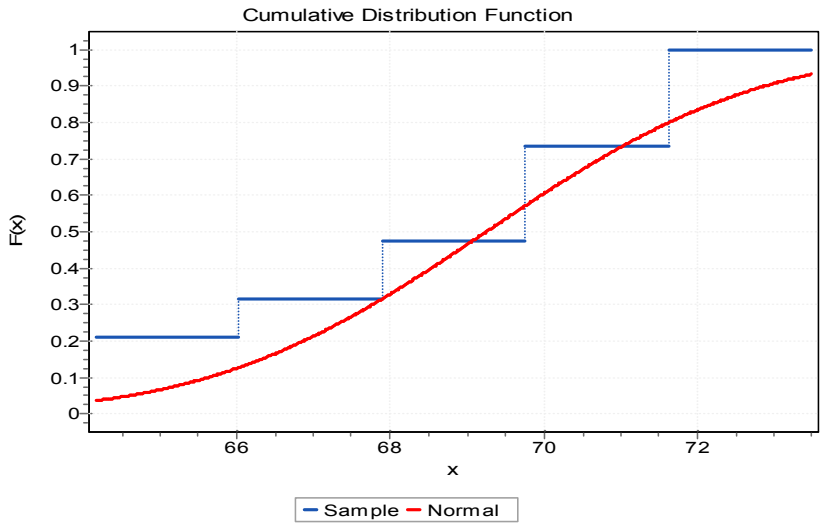


Fig. 4.34: Cumulative Distribution function for *Mitragyna ciliata* considering bending strength as Normal (Experimental) (x is for bending strength)

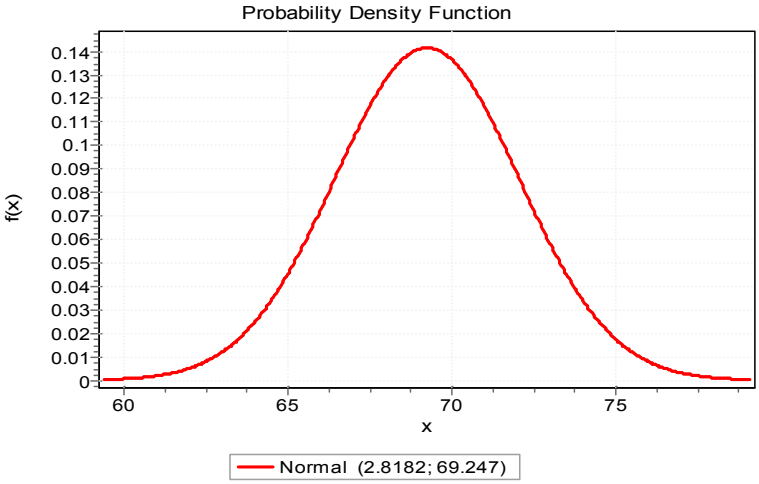


Fig. 4.35: Probability Distribution Model for *Mitragyna ciliata* considering bending strength as Normal (Assigned) (x is for bending strength)

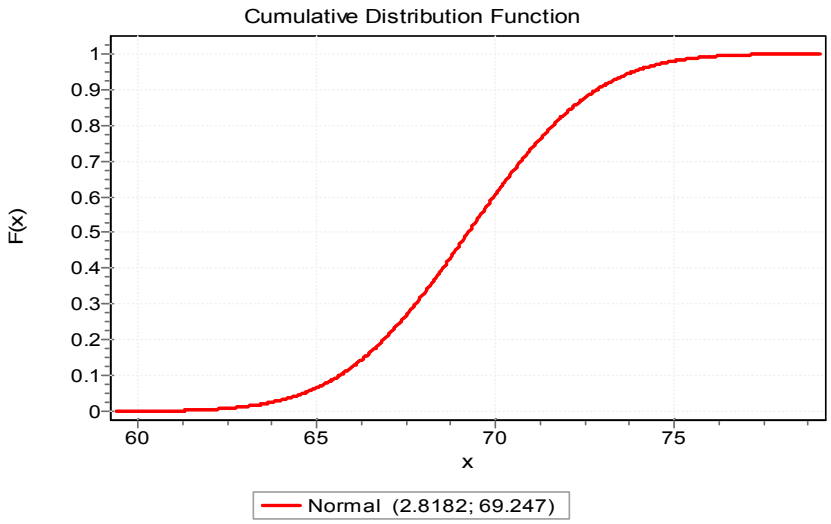


Fig. 4.36: Cumulative Distribution Model for *Mitragyna ciliata* considering bending strength as Normal (Assigned) (x is for bending strength)

4.7: STOCHASTIC MODELS OF THE BASIC VARIABLES FOR RELIABILITY ANALYSIS

Considering the three timber specie *Mitragyna ciliata* which has been classified as C_{18} based on EN338:2003(E) has the following design parameters:

The bending strength is

$$f_{m,o,d} = \frac{k \bmod f_{m,0,k}}{\gamma} = \frac{0.6*18}{1.3} = 8.31N / mm^2$$

The compressive strength is

$$f_{c,o,d} = \frac{k \bmod f_{c,0,k}}{\gamma} = \frac{0.6*18}{1.3} = 8.31N / mm^2 \text{ N/mm}^2$$

Also *Afromosia elata*, classified as D_{30}

$$f_{m,o,d} = \frac{k_{mod} f_{m,0,k}}{\gamma} = \frac{0.6*30}{1.3} = 13.85 \text{N/mm}^2$$

$$f_{c,o,d} = \frac{k_{mod} f_{c,0,k}}{\gamma} = \frac{0.6*22}{1.3} = 10.15 \text{N/mm}^2$$

And *Confusa grandiflora*, classified as D_{30}

$$f_{m,o,d} = \frac{k_{mod} f_{m,0,k}}{\gamma} = \frac{0.6*30}{1.3} = 13.85 N/mm^2$$

$$f_{c,o,d} = \frac{k_{mod} f_{c,0,k}}{\gamma} = \frac{0.6*23}{1.3} = 10.6 N/mm^2 \text{ N/mm}^2$$

4.7.1 : Stochastic Models of The Basic Variables,

From the descriptive statistic results presented in Table 4.14 the stochastic parameters for each of the failure modes are shown in Tables 4.34 to 4.47 for different species under consideration. It is to be noted that for all the Tables mentioned, γ is 1.3, k_{mod} is 0.6, and α is 0.8 for compression and bending failure criteria, and 0.2 for flexural and lateral torsional buckling criteria. The α varies from 0.2, 0.4, 0.6 and 0.8 for carrying capacity of dowel and shear stress failure. $k_{c,z}$ is 0.35,k_{crit} is 0.5, n_1 is 22no. n_2 is 17no. r_1 is 504mm, r_2 is 384mm, $F_{v,k}$ is 2, γ_m is 1.3. Values of covariance were obtained from (Afolayan , 2005)

Table 4.34: Stochastic parameters for *Mitragyna ciliata* Timber Specie (Compression failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fc,0,k*	Characteristic compression strength // to grain or	Lognormal	18.0 N/mm^2	0.03955	0.70 N/mm^2
2.0	X_2	Q_k	Imposed load	Gumbel	18.75 kN	0.4	7.5 kN
3.0	X_3	*L*	Length of portal frame	normal	10000 mm	0.06	24 mm
4.0	X_4	*B*	Width of column	Normal	400mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200mm	0.06	72mm

Table 4.35: Stochastic parameters for *Afromosia elata* Timber Specie (compression failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fc,0,k*	Characteristic compression strength // to grain or	Lognormal	22 N/mm^2	0.03844	0.86 N/mm^2
2.0	X_2	Q_k	Imposed load	Gumbel	18.75 kN	0.4	7.5 kN
3.0	X_3	*L*	Length of portal frame	normal	10000 mm	0.06	24mm
4.0	X_4	*B*	Width of column	Normal	400 mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200 mm	0.06	72 mm

Table 4.36: Stochastic parameters for *Confusa grandiflora* Timber Specie (compression failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fc,0,k*	Characteristic compression strength // to grain or	Lognormal	23 N/mm^2	0.03605	0.83N/mm^2
2.0	X_2	Q_k	Imposed load	Gumbel	18.75 kN	0.4	7.5kN
3.0	X_3	*L*	Length of portal frame	normal	10000 mm	0.06	24mm
4.0	X_4	*B*	Width of column	Normal	400mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200mm	0.06	72mm

Table 4.37: Stochastic parameters for *Mitragyna ciliata* Timber Specie (bending failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fm,k*	Characteristic bending strength // to grain	Lognormal	18N/mm^2	0.039	0.70N/mm^2
2.0	X_2	*L*	Length of portal frame	Normal	10000mm	0.6	600mm
3.0	X_3	Q_k	Imposed load	Gumbel	18.75kN	0.4	7.5kN
4.0	X_4	*h1*	Height of column	Normal	3000mm	0.06	180mm
5.0	X_5	*B*	width of column	Normal	400mm	0.06	24mm
6.0	*X6*	*H*	Depth of column	Normal	1200mm	0.06	7.2mm

Table 4.38: Stochastic parameters for *Afromosia elata* Timber Specie (bending failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fm,k*	Characteristic bending strength // to grain	Lognormal	22 N/mm^2	0.3844	0.70 N/mm^2
2.0	X_2	*L*	Length of portal frame	Normal	10000 mm	0.6	600 mm
3.0	X_3	Q_k	Imposed load	Gumbel	18.75 kN	0.4	7.5kN
4.0	X_4	*h1*	Height of column	Normal	3000 mm	0.06	180 mm
5.0	X_5	*B*	width of column	Normal	400 mm	0.06	24 mm
6.0	*X6*	*H*	Depth of column	Normal	1200	0.06	7.2

Table 4.39: Stochastic parameters for *Confusa grandiflora* Timber Specie (bending failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*fm,k*	Characteristic bending strength // to grain	Lognormal	23 N/mm^2	0.03605	0.83N/ mm^2
2.0	X_2	*L*	Length of portal frame	Normal	10000 mm	0.6	600mm
3.0	X_3	Q_k	Imposed load	Gumbel	18.75 kN	0.4	7.5kN
4.0	X_4	*h1*	Height of column	Normal	3000mm	0.06	180mm
5.0	X_5	*B*	width of column	Normal	400mm	0.06	24mm
6.0	*X6*	*H*	Depth of column	Normal	1200mm	0.06	7.2mm

Table 4.40: Stochastic Table for *Mitragyna ciliata* Timber Specie (flexural buckling failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*L*	Length of portal frame	Normal	10000mm	0.06	600mm
2.0	X_2	*Qk*	Imposed load	Gumbel	18.75 kN	0.4	7.5kN
3.0	X_3	*h1*	Height of column	normal	3000mm	0.06	180mm
4.0	X_4	*B*	Width of column	Normal	350mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200mm	0.06	72mm
6.0	X_6	*fm,d*	bending strength // to grain	lognormal	8.31N/mm^2	0.039	0.32 N/mm^2
7.0	X_7	*fc,0,d*	Compressive strength // to grain	Lognormal	12.96 N/mm^2	0.039	0.51N/mm^2

Table 4.41: Stochastic parameters for *Afromosia elata* Timber Specie (flexural buckling mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*L*	Length of portal frame	Normal	10000mm	0.06	600mm
2.0	X_2	*Qk*	Imposed load	Gumbel	18.75kN	0.4	7.5kN
3.0	X_3	*h1*	Height of column	normal	3000mm	0.06	180mm
4.0	X_4	*B*	Width of column	Normal	350mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200mm	0.06	72mm
6.0	X_6	*fm,d*	bending strength // to grain	lognormal	12.46 N/mm^2	0.026	0.32N/mm^2
7.0	X_7	*fc,0,d*	Compressive strength // to grain	Lognormal	10.15N/mm^2	0.05	0.51 N/mm^2

Table 4.42: Stochastic parameters for *Confusa grandiflora* Timber Specie (flexural buckling mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	*L*	Length of portal frame	Normal	10000mm	0.06	600mm
2.0	X_2	*Qk*	Imposed load	Gumbel	18.75kN	0.4	7.5kN
3.0	X_3	*h1*	Height of column	normal	3000mm	0.06	180 mm
4.0	X_4	*B*	Width of column	Normal	350mm	0.06	21mm
5.0	X_5	*H*	Depth of column	Normal	1200mm	0.06	72 mm
6.0	X_6	*fm,d*	bending strength // to grain	lognormal	13.85N/mm^2	0.023	0.32 N/mm^2
7.0	X_7	*fc,0,d*	Compressive strength // to grain	Lognormal	10.6 N/mm^2	0.048	0.51 N/mm^2

Table 4.43: Stochastic parameters for *Mitragyna ciliata* Timber Specie (lateral torsional buckling failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	Q_K	Imposed load	Gumbel	18.75 kN	0.4	7.5 kN
2.0	X_2	*B*	Width of column	Normal	350 mm	0.06	21mm
3.0	X_3	*H*	Depth of column	Normal	1200 mm	0.06	72 mm
4.0	X_4	*L*	Length of portal frame	Normal	10000 mm	0.06	600 mm
5.0	X_5	*fm,d*	Bending strength // to grain	lognormal	8.31 N/mm^2	0.039	0.32 N/mm^2
6.0	X_6	*h1*	Height of column	Normal	3000 mm	0.06	180mm
7.0	X_7	$f_{c,0,d}$	Compressive strength // to grain	lognormal	12.96 N/mm^2	0.04	0.51 N/mm^2

Table 4.44: Stochastic parameters for *Afromosia elata* Timber Specie (lateral torsional buckling failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	Q_K	Imposed load	Gumbel	18.75 kN	0.4	7.5kN
2.0	X_2	B	Width of column	Normal	350 mm	0.06	21mm
3.0	X_3	H	Depth of column	Normal	1200 mm	0.06	72mm
4.0	X_4	L	Length of portal frame	Normal	10000mm	0.06	600mm
5.0	X_5	fm,d	Bending strength // to grain	lognormal	12.46 N/mm^2	0.15	1.87N/mm^2
6.0	X_6	$h1$	Height of column	Normal	3000 mm	0.06	180mm
7.0	X_7	$f_{c,0,d}$	Compressive strength // to grain	lognormal	10.15 N/mm^2	0.15	1.52N/mm^2

Table 4.45: Stochastic parameters for *Confusa grandiflora* Timber Species (lateral torsional buckling failure mode)

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	Q_K	Imposed load	Gumbel	18.75kN	0.4	7.5kN
2.0	X_2	B	Width of column	Normal	350mm	0.06	21mm
3.0	X_3	H	Depth of column	Normal	1200mm	0.06	72mm
4.0	X_4	L	Length of portal frame	Normal	10000mm	0.06	600mm
5.0	X_5	fm,d	Bending strength // to grain	lognormal	13.85N/mm^2	0.15	2.08 N/mm^2
6.0	X_6	$h1$	Height of column	Normal	3000mm	0.06	180mm
7.0	X_7	$f_{c,0,d}$	Compressive strength // to grain	lognormal	10.6 N/mm^2	0.15	1.5N/mm^2

Table 4.46: Stochastic parameters of Basic variables for Carrying Capacity of Dowel for the Three Timber Specie

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	X_1	L	Length of portal frame	Normal	10000mm	0.6	600mm
2.0	X_2	Q_k	Imposed Load	Normal	18.75kN	0.4	7.5kN
3.0	X_3	h_1	Height of column	Normal	3000mm	0.06	180

Table 4.47: Stochastic parameters of the Basic variables For Shear stress failure for the three Timber species

S/NO.	VALUE	VARIABLE	MEANING	DISTRIBUTION TYPE	EX	COVARIANCE	SX
1.0	*X1*	V_M	Force at joint	lognormal	161.36 kN	0.15	24.20 kN
1.0	*X1*	*Qk*	Imposed load	Gumbel	18.75kN	0.4	7.5kN
2.0	X_2	*b*	Width of column	Normal	400 mm	0.06	24mm
2.0	X_3	*h*	Length of portal frame	Normal	1200 mm	0.06	72 mm

4.8: RESULTS OF RELIABILITY ANALYSIS

The results of reliability analysis of the solid slender timber columns were obtained for the three timber specie using the program developed. The results are as presented in Tables F1-F20 of Appendix F and are plotted in Figs. 4.37 to 4.56.

4.8.1 Compression Failure Criterion

Figures 4.37 to 4.39 present safety levels for *Mitragyna ciliata*, *Afromosia elata* and *Confusa grandiflora* timber specie under the influence of compression failure criterion.

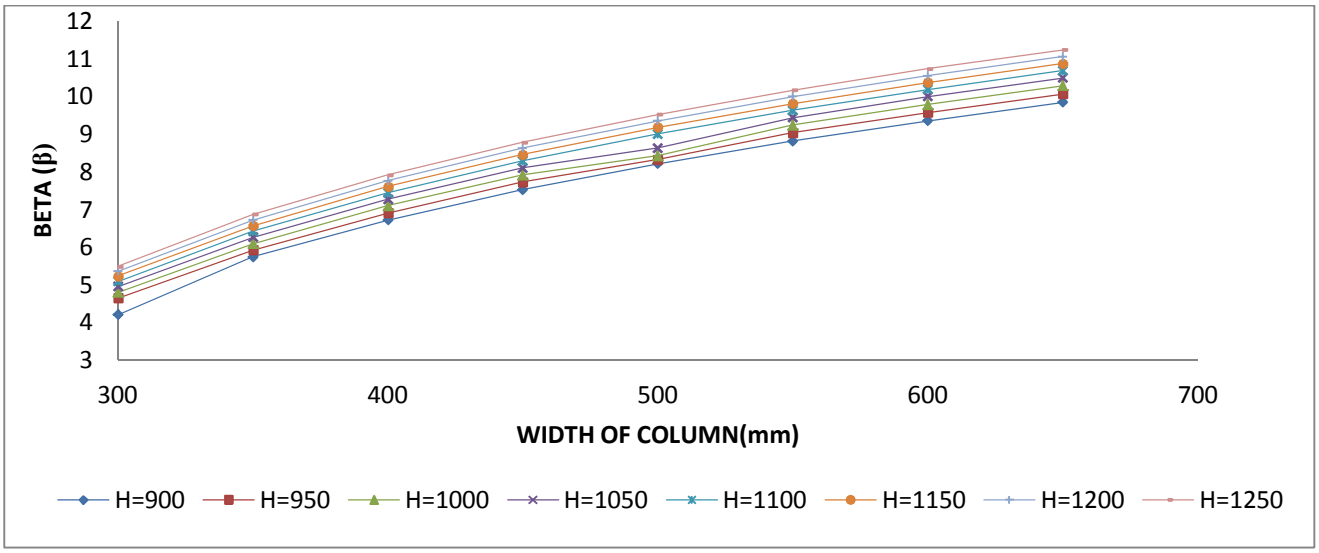


Fig.4.37: Safety Index Versus Width of Column for *Mitragyna ciliata* Timber-compression (α=0.8)

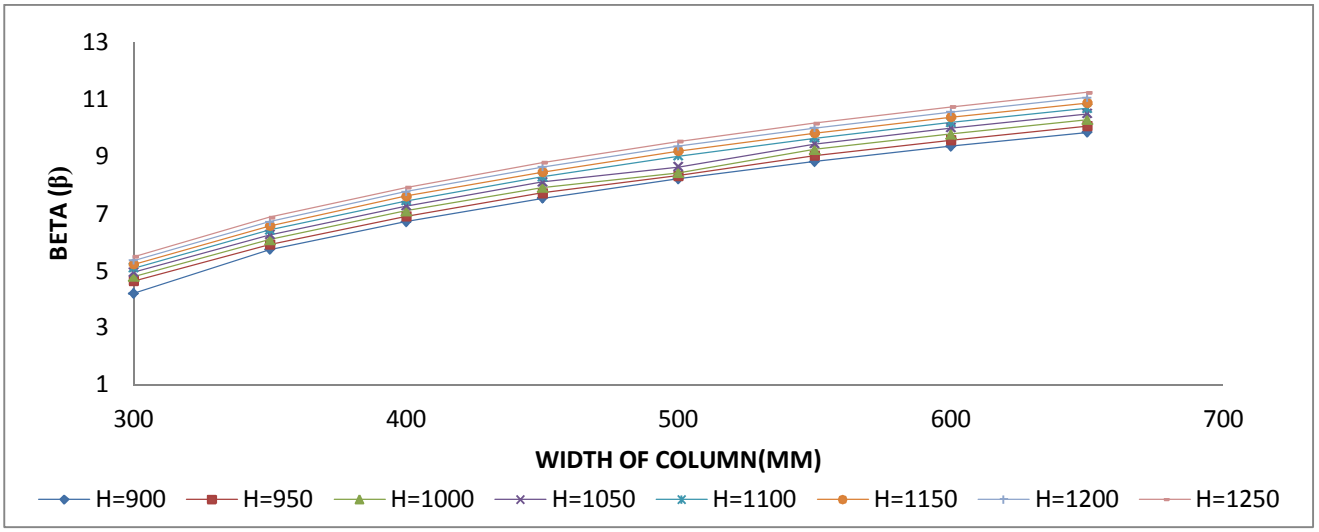


Fig.4.38: Safety index versus width of column for *Afromosia elata* compression (α=0.8)

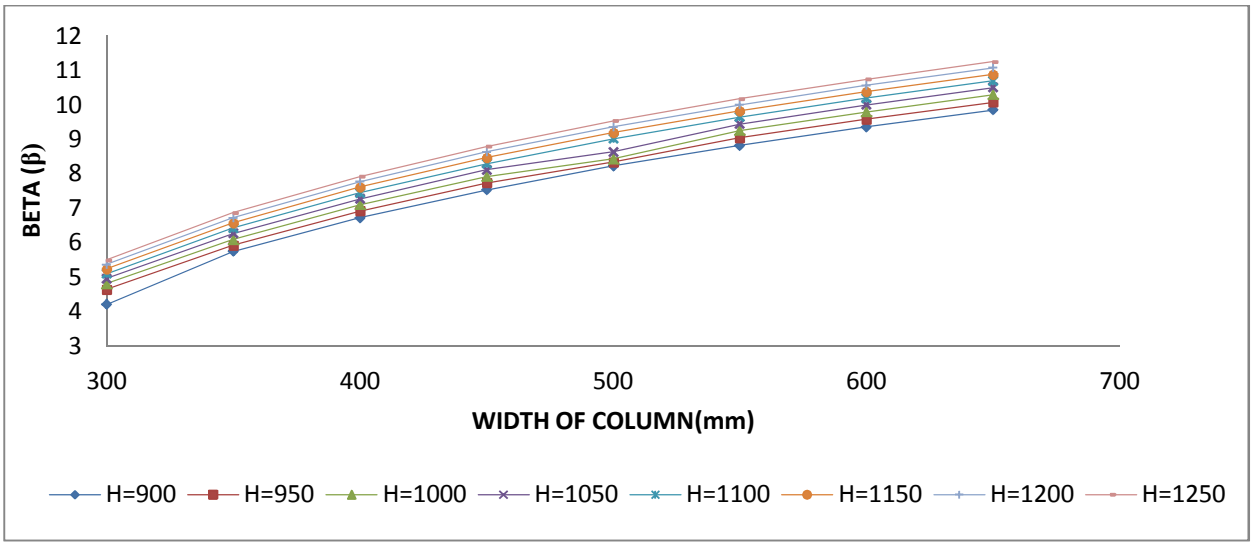


Fig.4.39: Safety index versus width of column for *Confusa grandiflora*-compression (α=0.8)

From Figs (4.37) to (4.39) of the safety index values against the width of column for the three timber specie for failure mode I (compression criterion) obtained indicated that as the width and depth of section were independently increased, the safety index also increases. The increase in the safety index could be attributed to increase in axial rigidity (AE) of the column due to increase in width and depth of the column.

4.8.2 Bending Failure Criterion

Figures 4.40 to 4.42 present the graph safety levels for the three timber species under the influence of bending failure criterion.

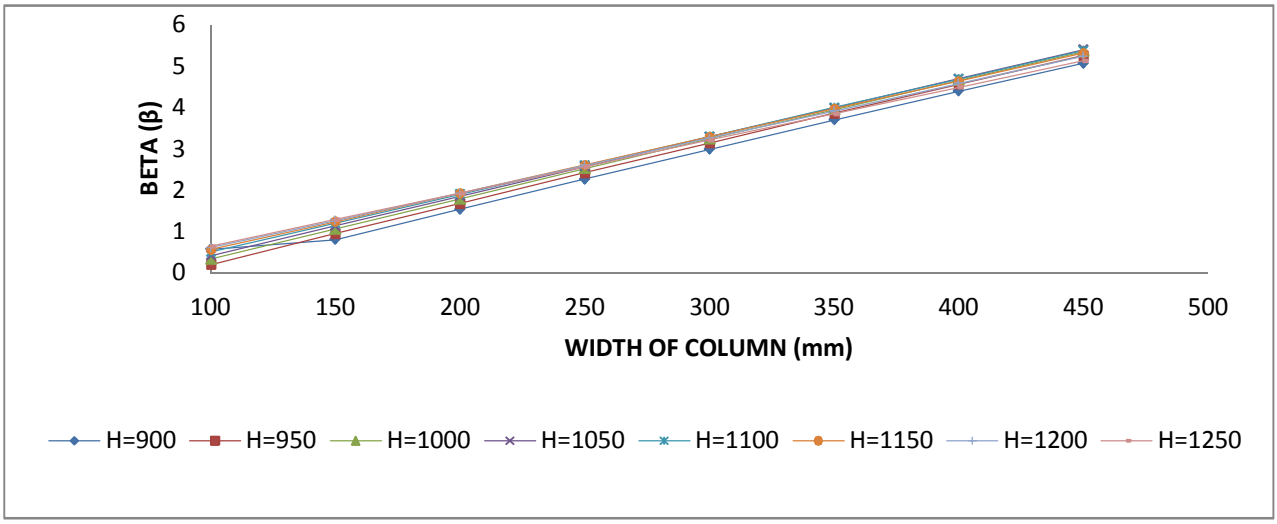


Fig.4.40: Safety Index versus Width of Column for *Mitragyna ciliata* Timber-Bending (α=0.8)

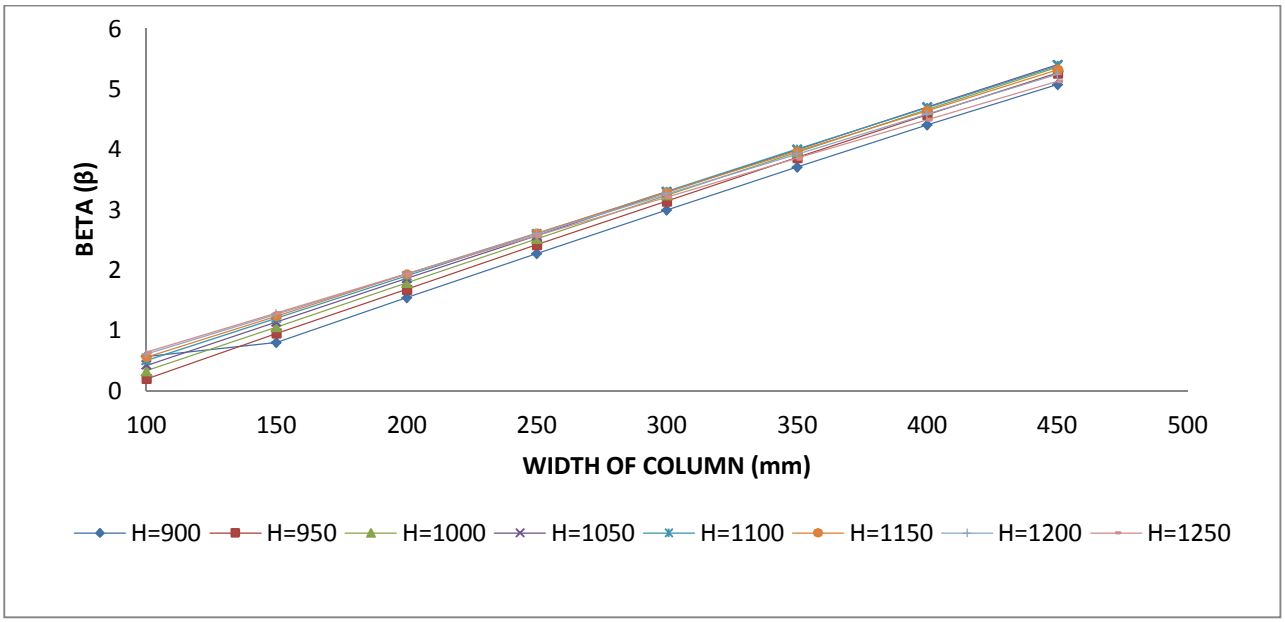


Fig.4.41: Safety index versus width of column *Afromosia elata*- Bending (α=0.8)

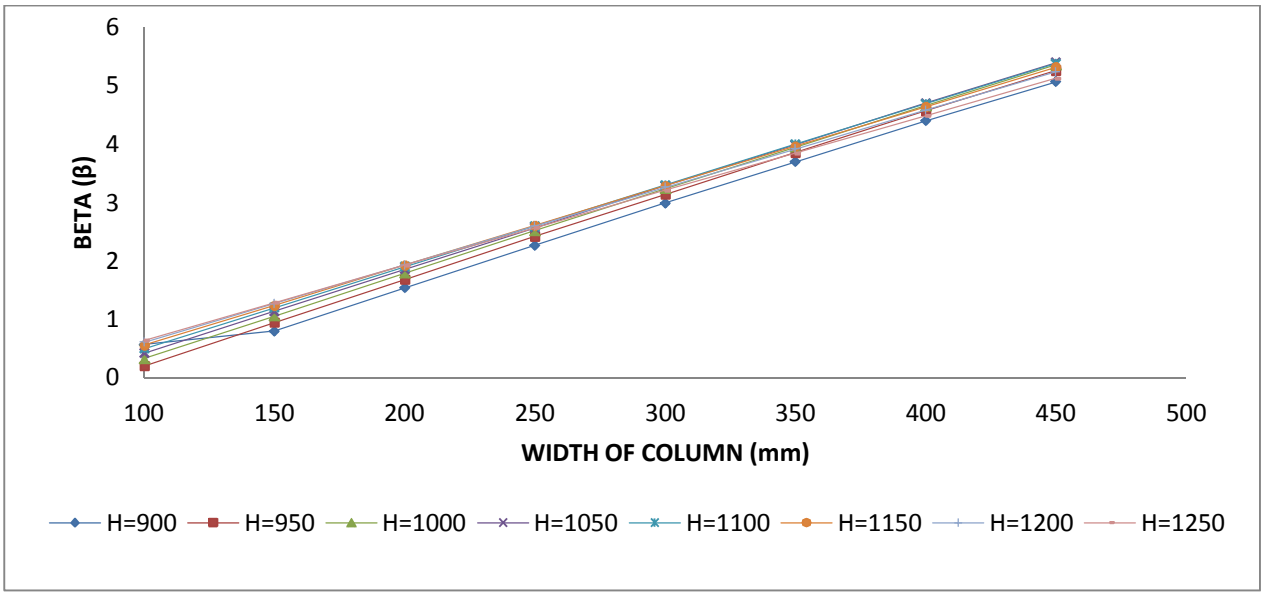


Fig. 4.42: Safety index versus width of column, *Confusa grandiflora*-Bending (α=0.8)

In Figures (4.40) to (4.42) there is consistent increase of safety index (β) as the width of column was increased from 100 -500mm, for a column subjected to bending parallel to grain at ultimate limit state of loading. This could be attributed to increase in flexural rigidity (EI) values which increase the rigidity of the column section

4.8.3 Flexural buckling Failure Criterion

Figures (4.43) to (4.45) present the output for failure mode III (Flexural Buckling) for the three timber species.

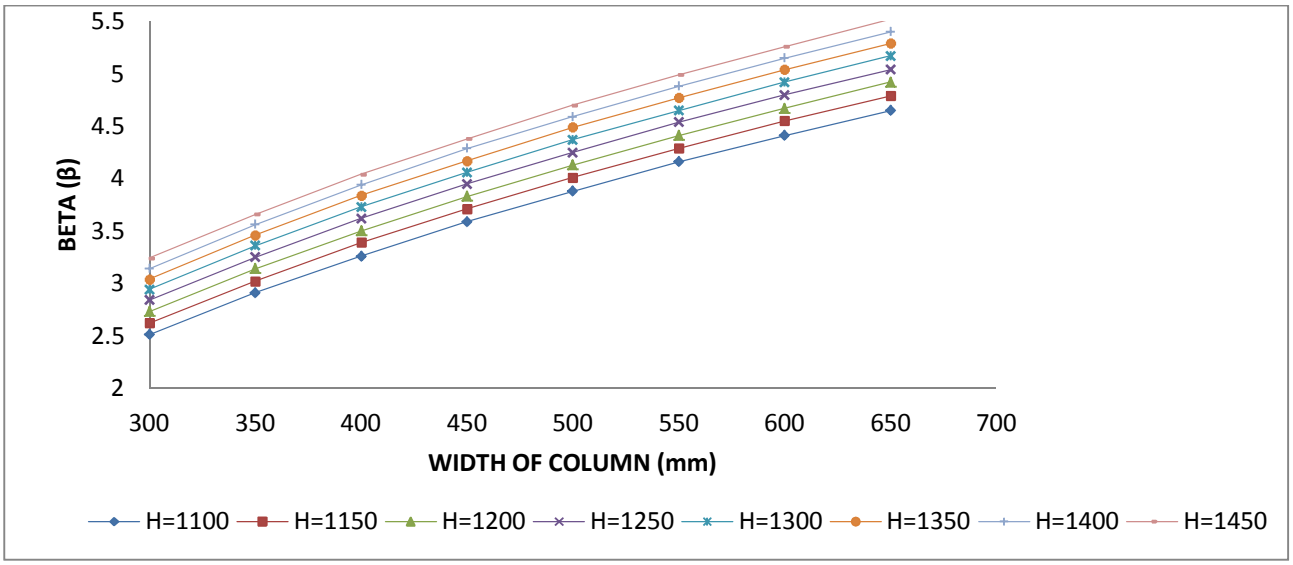


Fig.4.43: Safety index versus width of column for *Mitragyna ciliata* Timber F-buckle (α=0.2)

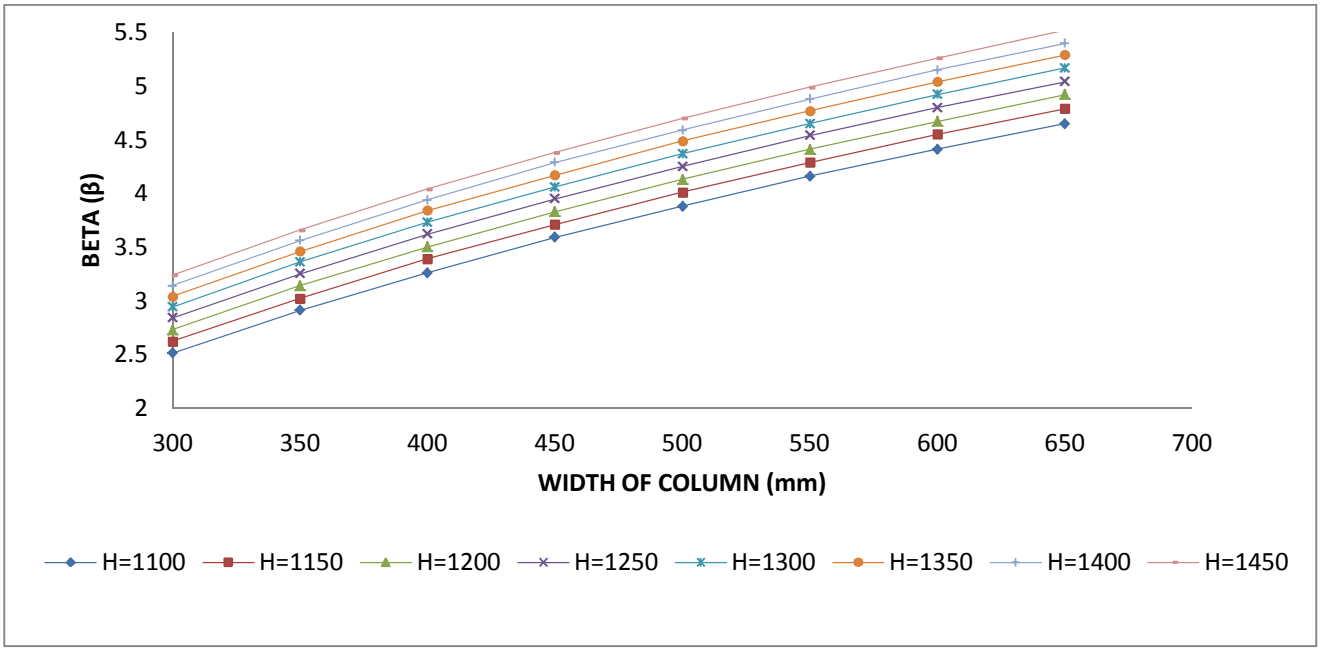


Fig.4.44: Safety index versus width of column for *Afromosia elata* f-Buckle (α=0.2)

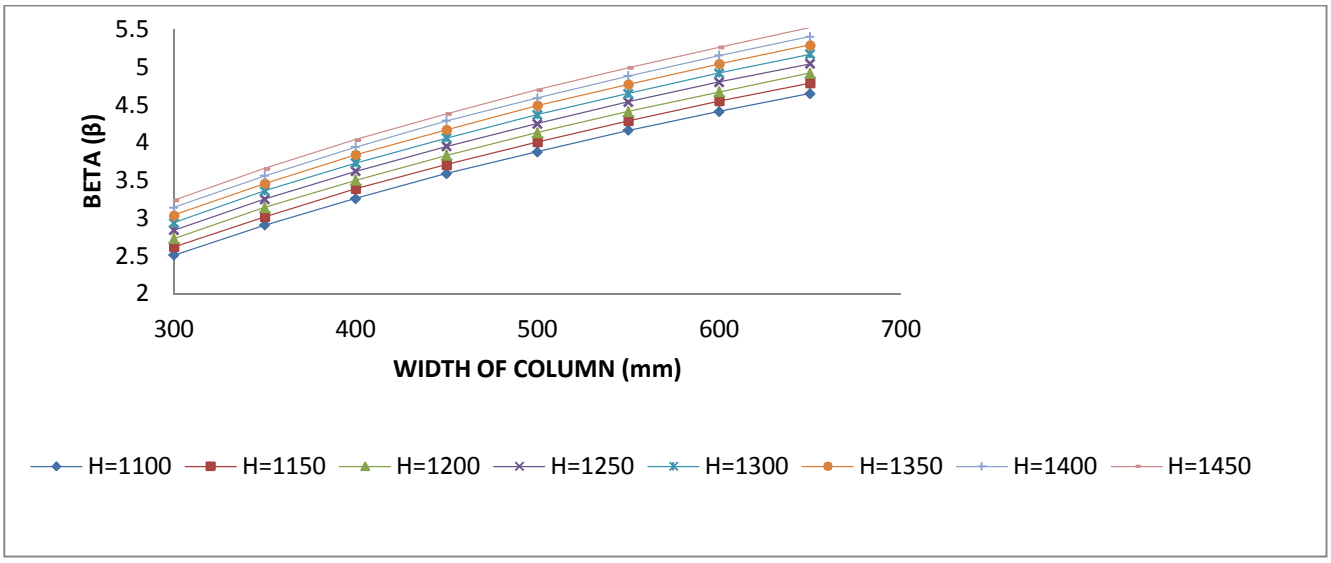


Fig.4.45: Safety index versus width of column for *Confusa grandiflora*-F-buckle (α=0.2)

The plots for Failure mode III (flexural buckling failure), have the same trend with failure mode I (compression criterion) and II (bending criterion). In Figs 4.43 – 4.45 there is increase in safety index (β) as the width of column increase from 300 to 650mm. This could be attributed also to increase in flexural rigidity (EI).

4.8.4 Lateral Torsional Buckling Failure Criterion

Figures (4.46) to (4.48) present safety indices for the three timber species under the influence of lateral torsional buckling.

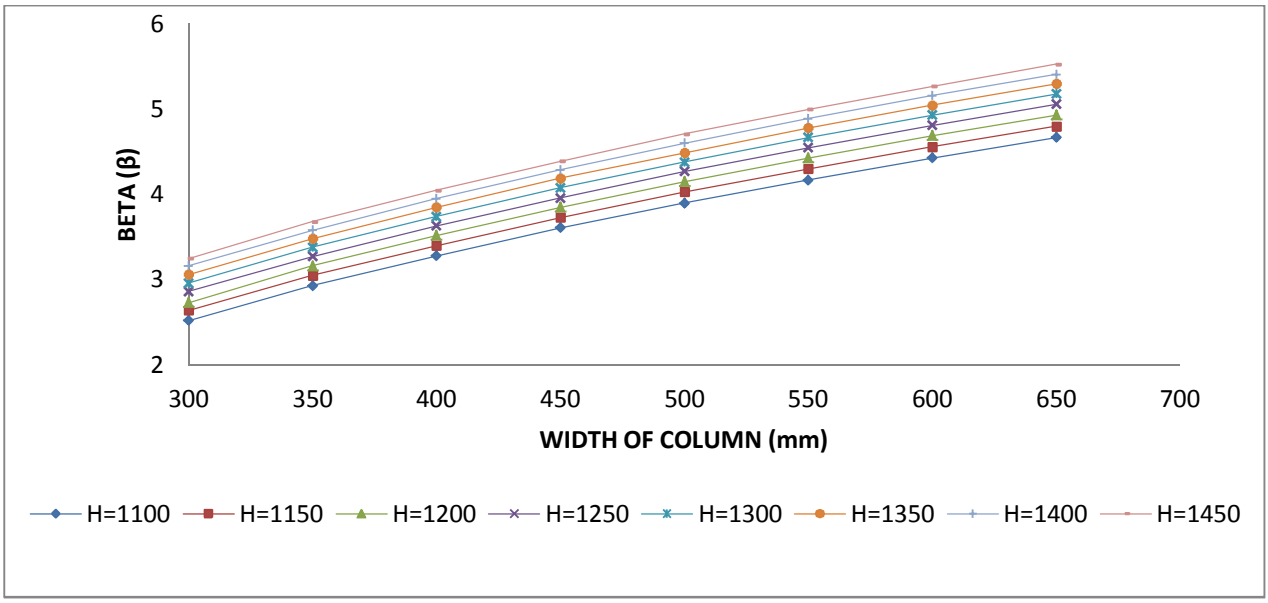


Fig.4.46: Safety index versus width of column for *Mitragyna ciliata* Timber LT-Buckle (α=0.2)

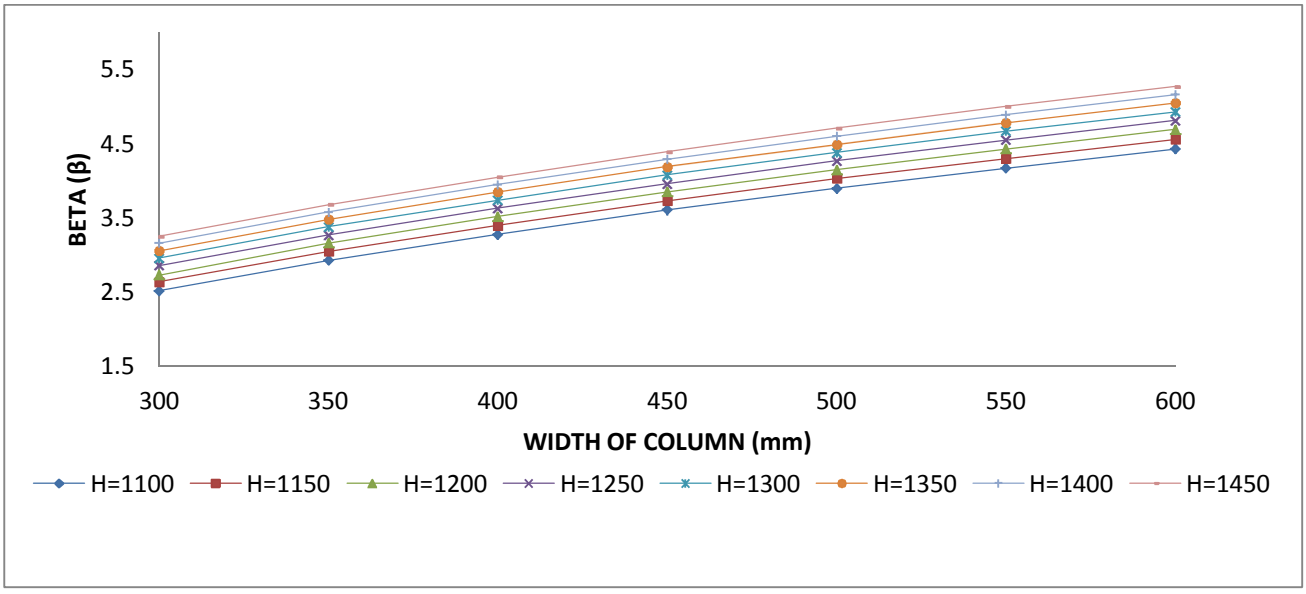


Fig.4.47: Safety index versus width of column for *Afromosia elata* LT-Buckling (α=0.2)

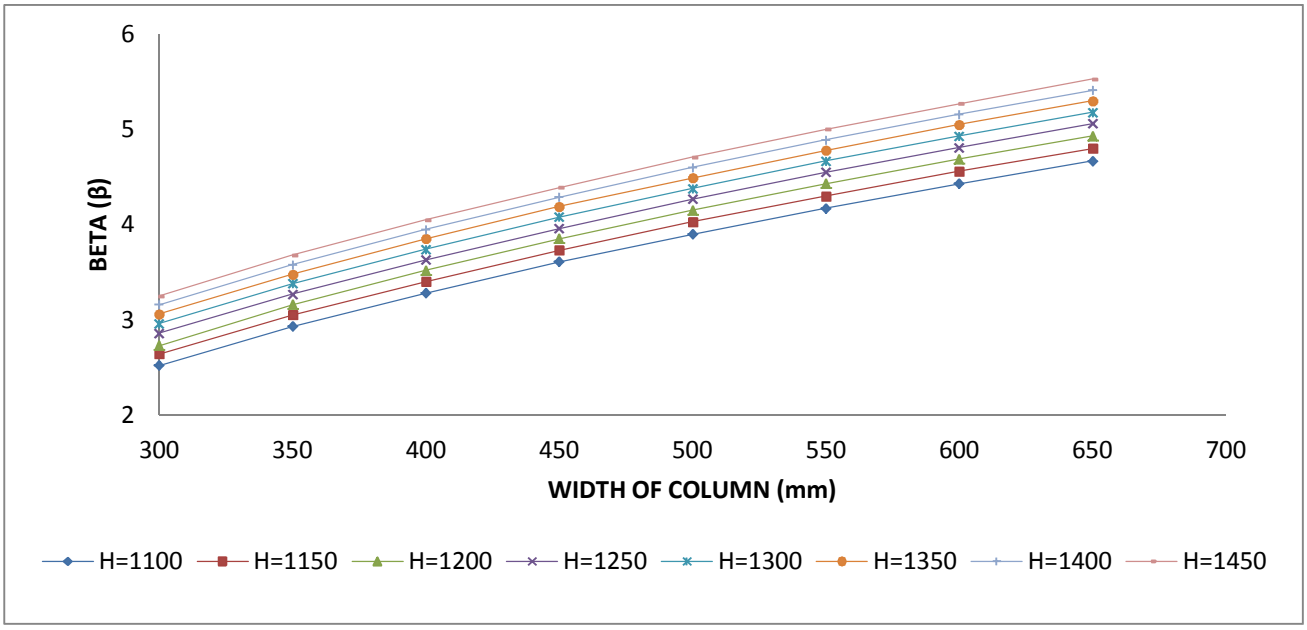


Fig.4.48: Safety Index versus Width of Column at Constant Depth (*Confusa grandiflora*-- Lt-Buckling) (α = 0.2)

Figures 4.46 to 4.48 which are for failure mode IV show a linear relationship between the obtained generated safety indices and width of .This could be attributed also to the increase in axial rigidities due to increase in the section width and depth.

4.8.5. Carrying Capacity Criterion

Figures 4.49 to 4.52 presents the graph of portal frame under Dowel carrying capacity failure at varying imposed load and varying length of the portal frame. A decrease in safety index can be noted as the span increases from 6500 – 10,000mm. This is because, increasing the span and imposed load implies an increase in moment and shear force at the joint there by decreasing the safety index. The timber species could be considered reliable up to a length of 10000mm with reliability index of 1.09, width of 400mm and depth of 1200mm of the column section.

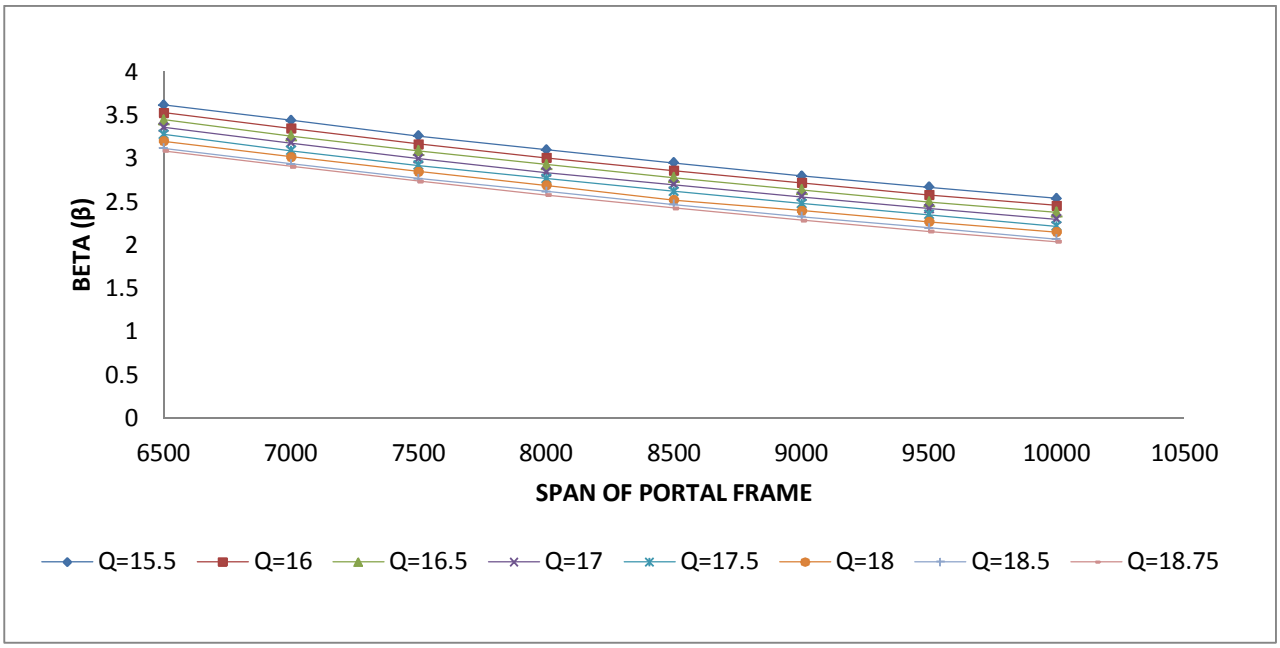


Fig 4.49: Safety Index versus span of Portal Frame, (α=0.2)

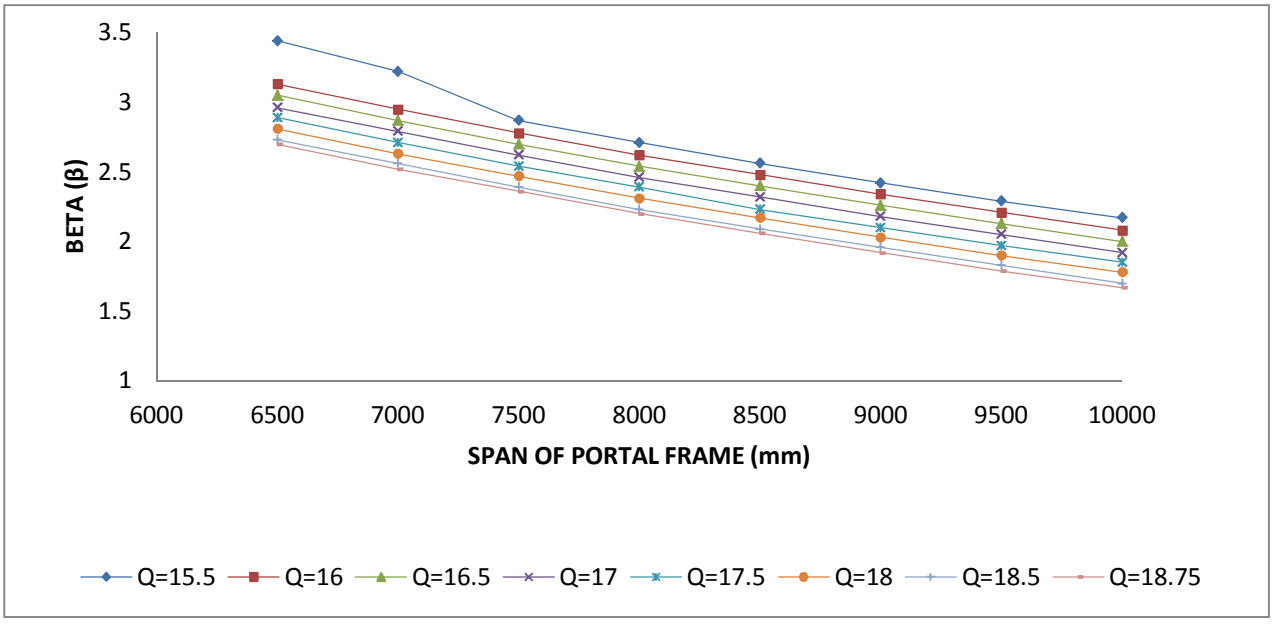


Fig 4.50: Safety index versus length of portal frame (α=0.4)

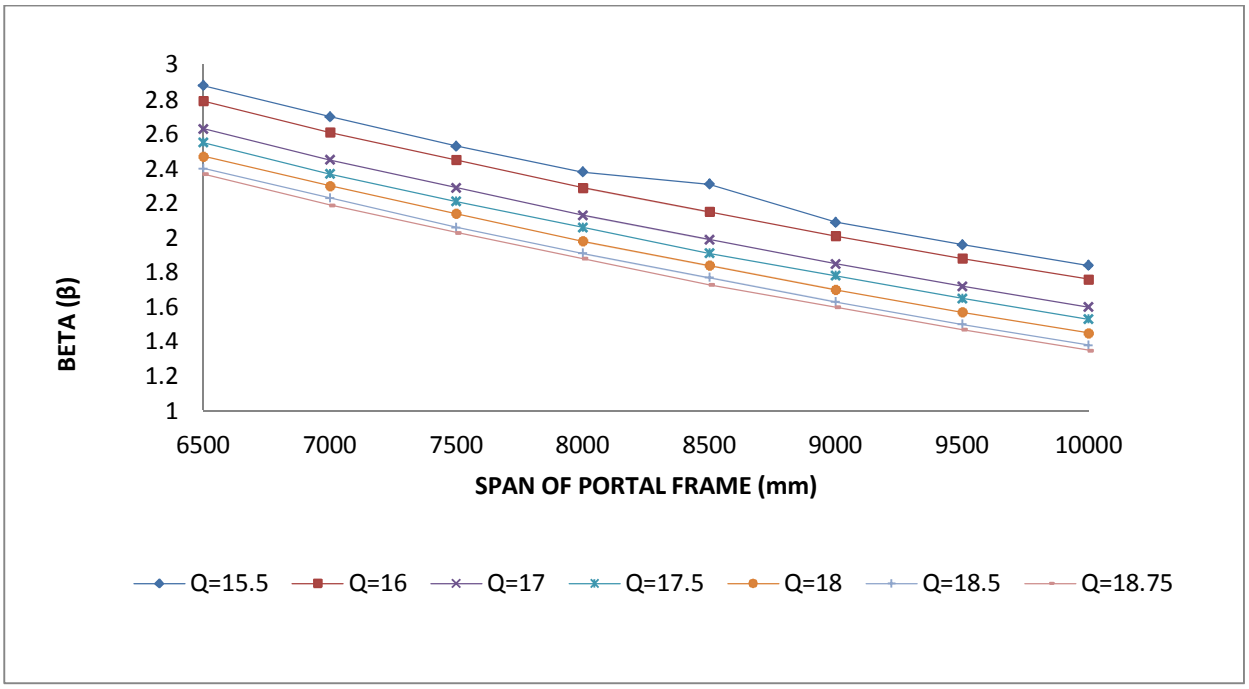


Fig 4.51: Safety index versus span of portal frame (α=0.6)

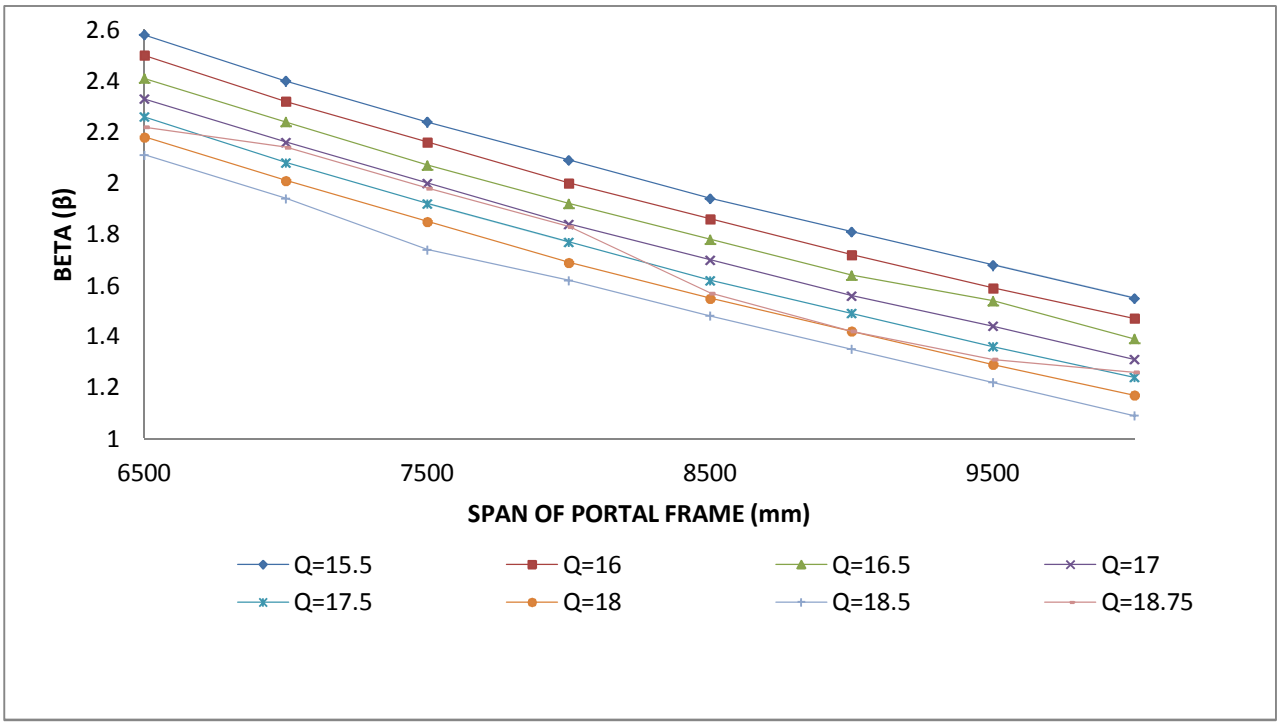


Fig 4.52: Safety index versus span of portal frame (α=0.8)

4.8.5 Shear stress Criterion

Figures (4.53) to (4.56) indicate a gentle decrease in safety index in the shear stress criterion. This could be attributed to the increase in applied force at the joint. The safety index would automatically decrease with increase in moment and shear force that will be generated at the joint.

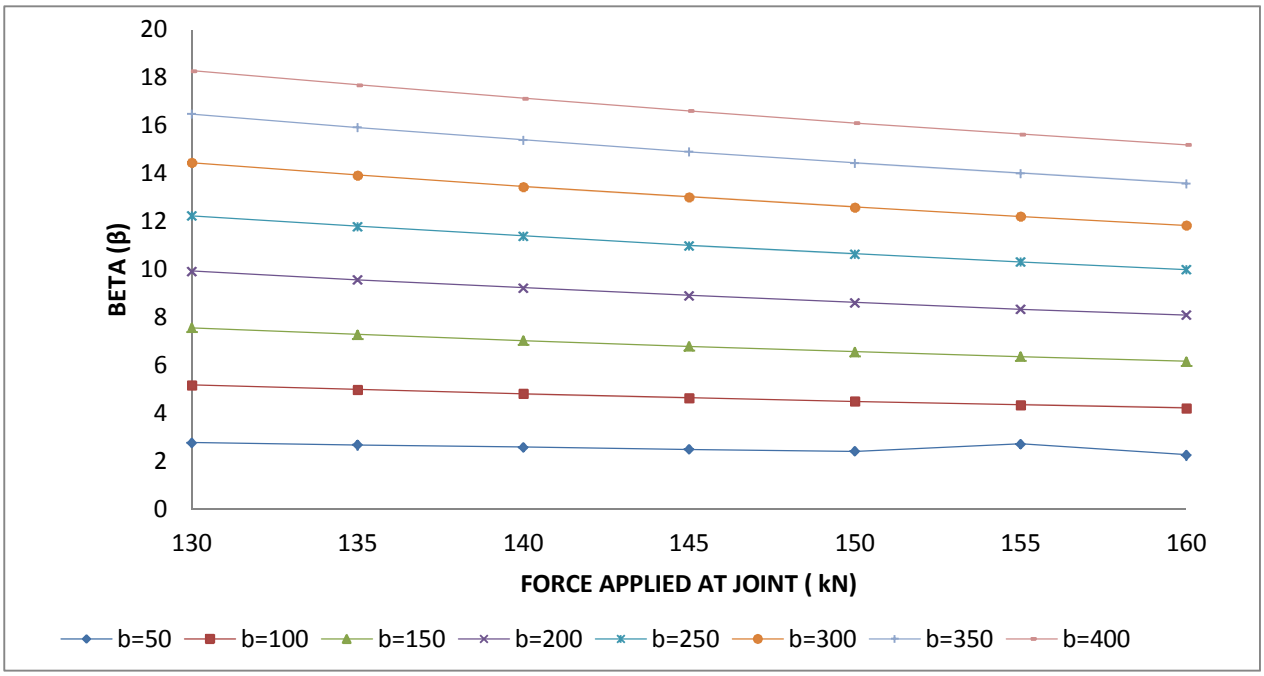


Fig.4.53: Safety index versus V_m (applied force at joint) (α=0.2)

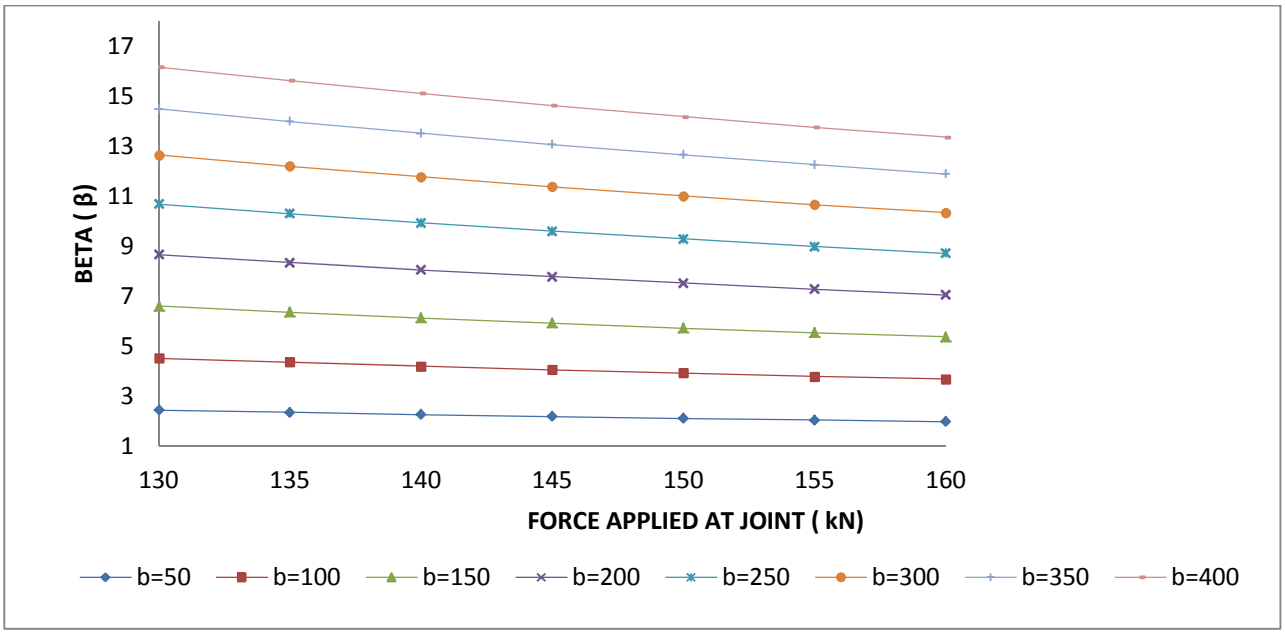


Fig.4.54: Safety index versus V_m (applied force at joint) (α=0.4)

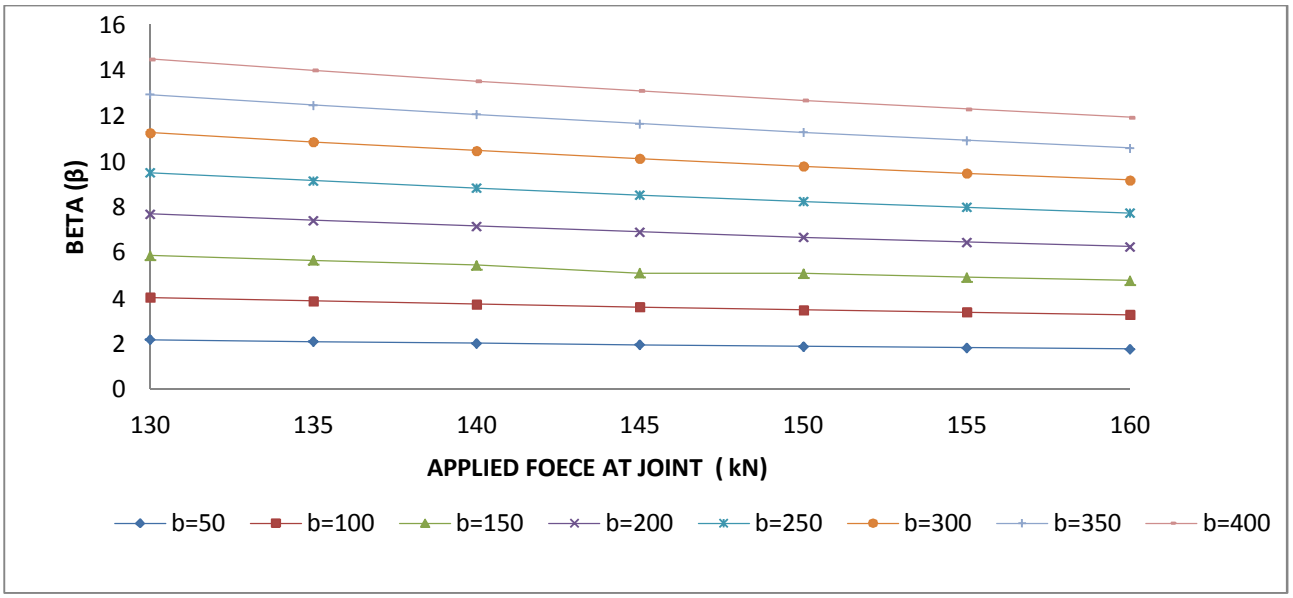


Fig.4.55: Safety index versus V_m at constant width (α=0.6)

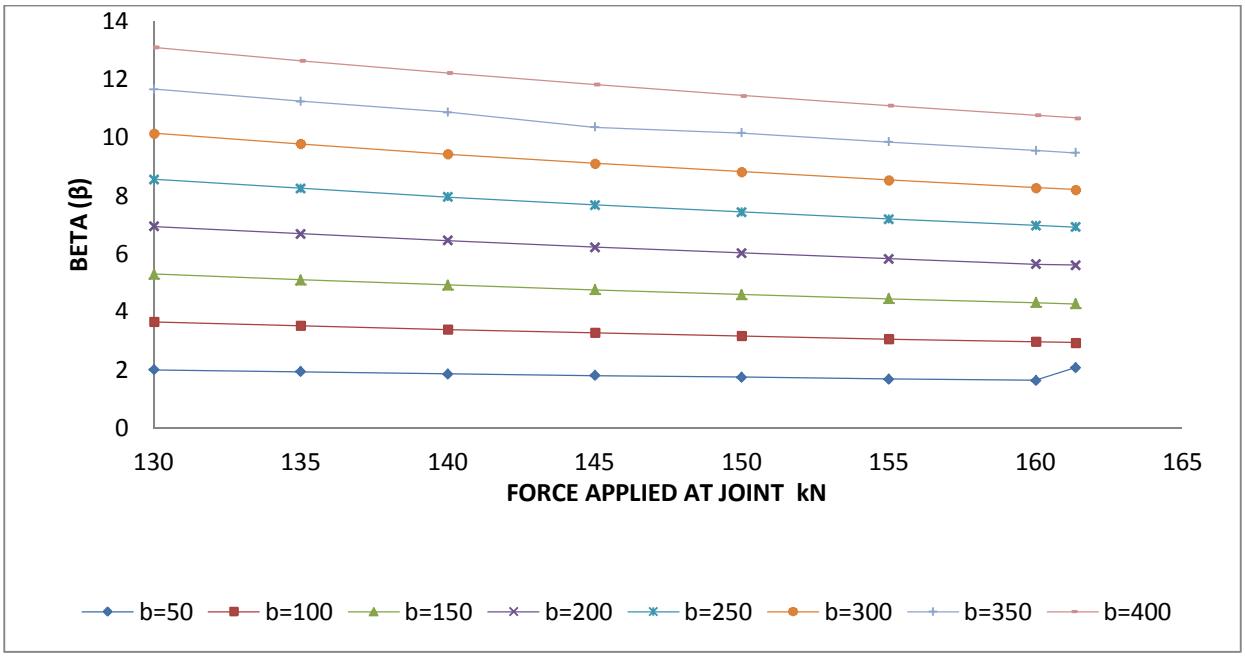


Fig.4.56: Safety index versus V_m (applied force at joint) (α=0.8)

From the safety indices obtained as presented in Fig.3.37 to 4.56, the following can be deduced:

The safety indices obtained using the deterministic design values for depth = 1200mm and width = 400mm of the section, for compression criterion considering Abura is 7.66 (Table F1 Appendix F), for Afromosia is 10.12 (Table F5 Appendix F) and for Berlinia is 10.56(Table F9 Appendix F). The values of safety indices obtained are higher than the code values of 3.8 (BS EN 1990:2002).

Also safety values obtained considering bending criterion using the deterministic parameters as in compression for *Mitragyana cilita* Timber species is 3.79(Table F2 Appendix F), *Afromosia elata* is 4.45 (Table F6 Appendix F), *Confusa grandiflora* is 4.58 (Table F10), Appendix F). The values obtained also indicate that they are higher than the code value of 3.8 (BS EN 1990:2002).

Values of safety index obtained considering flexural buckling using the deterministic values for Abura is 2.88 (Table F3, Appendix F), Afromosia is 3.38 (Table F7, Appendix F), Berlinia (Table F11, Appendix F). The values obtained are also lower than the code value of 3.8 (BS EN 1990:2002).

The values for lateral torsional buckling obtained using the same condition (deterministic design values for depth = 1200 and width = 400 of the section) as in the previous failure modes are; for *Mitragyana ciliata)* is 2.87 (Table F4, Appendix F), *Afromosia elata* is 3.4 (Table F8, Appendix F) and *Confusa grandiflora* is 3.52 (Table F12, Appendix F). Values obtained are also lower than the code value of 3.8 (BS EN 1990:2002). Generally, it can be observed that the higher the grade of timber, the higher the safety index.

Considering failure mode 5, carrying capacity of dowel, the values of safety index obtained using the deterministic design parameters of length = 10000mm and imposed load of 18.75kN at load ratio of 0.2 is 2.04 (Table F13, Appendix F), for load ratio of 0.4 the value is 1.67 (Table F14, Appendix, F) and for load ratio of 0.6 the value is 1.35 (Table F15, Appendix F) and finally for load ratio of 0.8 the safety index is 1.09 (Table F16, Appendix F16).

For shear capacity failure mode 6, values of safety index obtained considering deterministic values of width = 400mm and force applied at joint Vm =161.36KN at load ratio =0.2 is 15.08, at load ratio =0.4 is 13.26 at load ratio=0.6 is 11.84 and at load ratio=0.8 the value is 10.68. These shows that as the load ratio increases the safety index increases. Detailed results can be seen in Tables shown in Appendix F.

Conclusively, considering the first four failure modes, the critical mode of failure is at lateral torsional buckling, with the lowest safety index of 2.87 for Abura,3.4 for Afromosia and 3.52 for Berlinia (at load ratio=0.8 for compression and bending and load ratio=0.2 for Flexural buckling and lateral torsional buckling).

However, considering connections, the carrying capacity of dowel is the one having a critical value of Safety index of 1.09 at load ratio =0.8 which is lower than the code value of safety index of 3.8 (BS EN 1990:2002). Appendix F presents the tables for the generated safety indices for all the six failure modes.

4.9: SAFETY INDICES OBTAINED FOR THE DETERMINISTIC DESIGN USING THE TIMBER SPECIES

Considering the deterministic values used in the design of portal frame .The safety indices obtained for the three timber species considering failure mode I (compression failure) at load ratio of 1.42 and depth and width of section =1200mm and 400mm respectively are : 7.40, 8.96 and 9.39. While for failure mode II (bending failure), the safety indices obtained at 1.42 load ratio and depth and width of section of 1200mm and 400mm respectively are : 3.17, 4.02. 4.20. For Flexural buckling (failure mode III), at 1.42 load ratio and depth and width of section of 1200mm and 400mm respectively the safety indices are; 1.07,1.56, 1.67. For lateral torsional buckling (failure mode IV), the safety indices at the same condition with previous failure modes are; 1.18, 1.69, 1.80. As for connections, the Carrying capacity of dowel (failure mode V) under the same condition with the above modes of failure, the safety index is 0.33 and the value of safety index for shear stress failure (failure mode VI) is 8.21.

From the results, the critical mode of failure is flexural buckling with safety index of 1.07,for Abura under compression, which is relatively lower than the code value of safety index of 3.8 (BS EN 1990:2002).

Table 4.48: Safety index for the six failure modes using the design load ratio of α= 1.42, depth of 1200mm and width of 400mm

	Safety-Values at Load Ratio = 1.42					
Specie	Compression Criterion	Bending Criterion	Flexural Buckling Criterion	Lateral Torsional Buckling Criterion	Carrying Capacity Of Dowel Criterion	Shear Stress Failure Criterion
Mitragyna ciliata	7.4	3.17	1.07	1.18	0.33	8.21
Afromosia elata	8.96	4.02	1.56	1.69	0.33	0.33
Confusa grandiflora	9.39	4.20	1.67	1.8	0.33	0.33

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSION

Mitragyna ciliata, was classified as C18 while *Afromosia elata* and *Confusa grandiflora* Nigerian timber species were classified both as D30 based on EN 338 (2009).

The goodness of fit test carried on the three timber species using EasyFit (2010) has shown that the bending strength property was best modelled with normal distribution, while density and modulus of elasticity were best modelled using Gumbel distribution.

The reliability analysis of the solid slender Nigerian timber columns using Eurocode 5 (EC5) considering the three timber species revealed the following:

1. The four failure modes (i.e. compression, bending, flexural buckling and lateral torsional buckling) have shown that there was a linear relationship between the safety indices obtained and the width of the section. This could be attributed to the increase in axial and flexural rigidities due to increase in the width and depth of the section. The safety indices increase as the width and depth of section were independently increased.
2. The higher the timber strength class, the higher the value of safety index. For instance, under compression criterion, *Mitragyna ciliata* with strength Class C18 has safety index of 7.66 at load ratio of 0.8, while *Afromosia elata* and *Confusa grandiflora* with Class D30 has safety index of 10.12 and 10.56 respectively at load ratio of 0.8.
3. Values of Safety index for compression criterion are higher than the code value of 3.8 (BS EN 1990:2002) (Abura is 7.66, Afromosia is 10.12 and Berlinia is10.56) at load of 0.8.

4. Increase in load ratio resulted in decrease in the safety index. The load ratio of 1.42 used in deterministic design of the column in the portal frame gave the following safety indices; 7.40 for Abura, 8.96 for Afromosia and 9.39 for Berlinia, under compression criterion.
5. The critical mode of failure for the first 4 modes of failure is lateral torsional buckling, with the lowest safety index of 2.87 for Abura, 3.4 for Afromosia and 3.52 for Berlinia (load ratio α of 0.8 for compression and bending and 0.2 for Flexural Buckling criteria and Lateral Torsional Buckling)
6. As for Dowel connection, using load ratios of 0.2, 0.4, 0.6, and 0.8 the safety indices were found to decrease when the length of portal frame was increased with increase in imposed load. Also, increase in load ratio resulted in the decrease in the safety indices. (at load ratio 0.2, safety index is 2.04, at load ratio 0.4, safety index is 1.67, at load ratio 0.6 safety index is 1.35, at load ration 0.8, safety index is1.09)
7. Values of safety indices for carrying capacity of dowel are below the code value of safety index of 3.8(BS EN 1990:2002).

10 As for shear capacity criterion, using load ratio of 0.2, 0.4, 0.6, and 0.8, the safety indices obtained indicate that as the load ratio increases, the safety indices decrease (at load ratio 0.2, safety index is 15.08, at load ratio 0.4, safety index is 13.06, at load ratio 0.6, safety index is 11.84, at load ratio 0.8, safety index is 10.68). Values of safety indices obtained are above the code safety index 3.8 (BS EN 1990:2002)

11 Critical mode of failure for Dowel connection was found to be the carrying capacity of the dowel, having a critical value of safety index of 1.09 at load ratio of 0.8 which is lower than code value of 3.8 (BS EN 1990:2002).

5.2 RECOMMENDATIONS

Based on the findings of this research work, the following recommendations can be made:

i. The three (3) classified Nigerian timber specis are reliable structural material that can be used based on the recommendation of Eurocode 5 (EC5 (2004)) at load ratio of 0.8 for light weight portal frame and 0.2 for heavy weight portal frame.

ii. NCP2 (1973) is recommended for revision in order to adopt the recent timber design methodology for all Nigerian Timbers.

iii. Assigned stochastic distribution models are also recommended for use in timber structural reliability analysis.

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APPENDICES

APPENDIX A

Tables of Densities at Various Moisture Content

Table A1: Wet Density for Abura Timber at 21.38 % MC

Test Slice	Width (mm)	Depth (mm)	Length (mm)	Area (mm^2)	Total Volume V (mm^3)	m_1 (g)	Slice Density (kg/m^3)
ABR01	41	72	49	2952	144648	88	608
ABR02	40	75	50	3000	150000	74	493
ABR03	41	72	51	2952	150552	89	591
ABR04	40	75	52	3000	156000	80	513
ABR05	40	75	52	3000	156000	84	538
ABR06	41	72	50	2952	147600	86	583
ABR07	40	75	50	3000	150000	76	507
ABR08	41	73	49	2993	146657	77	525
ABR09	40	73	52	2920	151840	89	586
ABR10	41	72	51	2952	150552	84	558
ABR11	40	75	50	3000	150000	81	540
ABR12	40	75	50	3000	150000	78	520
ABR13	40	75	49	3000	147000	79	537
ABR14	41	72	52	2952	153504	82	534
ABR15	41	72	51	2952	150552	81	538

Table A2: Wet Density for Afromosia Timber at 24.71 % MC

Test Slice	Width (mm)	Depth (mm)	Length (mm)	Area (mm^2)	Total Volume V (mm^3)	m_1 (g)	Slice Density (kg/m^3)
AFR01	41	72	49	2952	144648	82	567
AFR02	40	75	50	3000	150000	74	493
AFR03	41	72	51	2952	150552	83	551
AFR04	40	75	52	3000	156000	80	513
AFR05	40	75	52	3000	156000	84	538
AFR06	41	72	50	2952	147600	86	583
AFR07	40	75	50	3000	150000	80	533
AFR08	41	73	49	2993	146657	78	532
AFR09	40	73	52	2920	151840	89	586
AFR10	41	72	51	2952	150552	84	558
AFR11	40	75	50	3000	150000	81	540
AFR12	40	75	50	3000	150000	80	533
AFR13	40	75	49	3000	147000	83	565
AFR14	41	72	52	2952	153504	82	534
AFR15	41	72	51	2952	150552	81	538

Table A3: Wet Density for Berlinia Timber at 27.27 % MC

Test Slice	Width (mm)	Depth (mm)	Length (mm)	Area (mm^2)	Total Volume V (mm^3)	m_1 (g)	Slice Density (kg/m^3)
BERNI01	41	72	49	2952	144648	77	532
BERNI02	40	75	50	3000	150000	75	500
BERNI03	41	72	51	2952	150552	80	531
BERNI04	40	75	52	3000	156000	80	513
BERNI05	40	75	52	3000	156000	78	500
BERNI06	41	72	50	2952	147600	80	542
BERNI07	40	75	50	3000	150000	81	540
BERNI08	41	73	49	2993	146657	79	539
BERNI09	40	73	52	2920	151840	80	527
BERNI10	41	72	51	2952	150552	76	505
BERNI11	40	75	50	3000	150000	80	533
BERNI12	40	75	50	3000	150000	81	540
BERNI13	40	75	49	3000	147000	79	537
BERNI14	41	72	52	2952	153504	80	521
BERNI15	41	72	51	2952	150552	78	518

APPENDIX B
Tables of Four and Three Point Bending Tests with Respective Bending Strength Values

Table B1: Reading of Four Point Bending Test for Abura Timber

S/NO	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load
	2000N	4000N	6000N	2000N	4000N	6000N	F_{max} (KN)
1	900	1274	1422	9.00	12.74	14.22	17.90
2	986	1209	1476	9.86	12.09	14.76	17.50
3	945	1252	1399	9.45	12.52	13.99	17.60
4	995	1172	1411	9.95	11.72	14.11	17.30
5	934	1236	1456	9.34	12.36	14.56	17.60
6	945	1282	1487	9.45	12.82	14.87	17.80
7	934	1181	1422	9.34	11.81	14.22	17.30
8	965	1297	1402	9.65	12.97	14.02	17.90
9	956	1206	1452	9.56	12.06	14.52	17.40
10	927	1171	1478	9.27	11.71	14.78	17.30
11	965	1190	1422	9.65	11.90	14.22	17.40
12	978	1250	1412	9.78	12.50	14.12	17.60
13	945	1216	1478	9.45	12.16	14.78	17.50
14	967	1160	1423	9.67	11.60	14.23	17.20
15	961	1148	1433	9.61	11.48	14.33	17.20
16	967	1211	1411	9.67	12.11	14.11	17.50
17	958	1286	1432	9.58	12.86	14.32	17.80
18	955	1252	1466	9.55	12.52	14.66	17.60
19	987	1234	1498	9.87	12.34	14.98	17.40
20	956	1265	1422	9.56	12.65	14.22	17.20

Table B2: Reading of Four Point Bending Test for Afromosia Timber

S/No.	Specimen Number	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load F_{max} (KN)
		2000N	4000N	6000N	2000N	4000N	6000N	
1	AFR01	967	1156	1301	9.67	11.56	13.01	23.40
2	AFR02	978	1143	1324	9.78	11.43	13.24	25.30
3	AFR03	956	1156	1356	9.56	11.56	13.56	24.40
4	AFR04	985	1109	1327	9.85	11.09	13.27	24.50
5	AFR05	987	1154	1378	9.87	11.54	13.78	25.40
6	AFR06	978	1189	1324	9.78	11.89	13.24	24.40
7	AFR07	934	1187	1376	9.34	11.87	13.76	24.60
8	AFR08	976	1143	1345	9.76	11.43	13.45	24.60
9	AFR09	945	1187	1324	9.45	11.87	13.24	24.60
10	AFR10	934	1165	1356	9.34	11.65	13.56	24.70
11	AFR11	976	1156	1366	9.76	11.56	13.66	24.40
12	AFR12	970	1187	1332	9.70	11.87	13.32	24.50
13	AFR13	934	1137	1305	9.34	11.37	13.05	24.70
14	AFR14	965	1186	1376	9.65	11.86	13.76	24.60
15	AFR15	974	1132	1356	9.74	11.32	13.56	24.10
16	AFR16	934	1143	1390	9.34	11.43	13.90	25.00
17	AFR17	945	1156	1324	9.45	11.56	13.24	24.70
18	AFR18	967	1165	1354	9.67	11.65	13.54	24.80
19	AFR19	976	1142	1323	9.76	11.42	13.23	24.50
20	AFR20	954	1132	1365	9.54	11.32	13.65	25.10

Table B3: Reading of Four Point Bending Test for Berlinia Timber

S/No.	Specimen Number	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load
		2000N	4000N	6000N	2000N	4000N	6000N	F_{max} (KN)
1	BERNI01	991	1174	1380	9.91	11.74	13.80	14.9
2	BERNI02	983	1160	1355	9.83	11.60	13.55	14.7
3	BERNI03	987	1167	1367	9.87	11.67	13.67	14.8
4	BERNI04	980	1154	1344	9.80	11.54	13.44	14.6
5	BERNI05	986	1164	1362	9.86	11.64	13.62	14.8
6	BERNI06	990	1171	1375	9.90	11.71	13.75	14.9
7	BERNI07	981	1156	1347	9.81	11.56	13.47	14.7
8	BERNI08	991	1174	1379	9.91	11.74	13.79	14.9
9	BERNI09	983	1160	1354	9.83	11.60	13.54	14.7
10	BERNI10	980	1154	1344	9.80	11.54	13.44	14.6
11	BERNI11	981	1157	1350	9.81	11.57	13.50	14.7
12	BERNI12	987	1166	1366	9.87	11.66	13.66	14.8
13	BERNI13	984	1161	1357	9.84	11.61	13.57	14.7
14	BERNI14	979	1152	1341	9.79	11.52	13.41	14.6
15	BERNI15	978	1151	1338	9.78	11.51	13.38	14.6
16	BERN16	983	1160	1355	9.83	11.60	13.55	14.7
17	BERNI17	990	1172	1376	9.90	11.72	13.76	14.9
18	BERNI18	987	1167	1367	9.87	11.67	13.67	14.8
19	BERNI19	982	1157	1350	9.82	11.57	13.50	14.7
20	BERNI20	978	1151	1339	9.78	11.51	13.39	14.6

Table B4: Reading of Three Point Bending Test for Abura Timber

S/No.	Specimen Number	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load
		2 000N	4 000N	6 000N	2 000N	4 000N	6 000N	F_{max} (KN)
1	ABR01	980	1108	1401	9.80	11.08	14.01	15.80
2	ABR02	978	1143	1431	9.78	11.43	14.31	15.50
3	ABR03	956	1167	1423	9.56	11.67	14.23	15.60
4	ABR04	985	1109	1462	9.85	11.09	14.62	15.30
5	ABR05	934	1154	1476	9.34	11.54	14.76	15.60
6	ABR06	945	1126	1423	9.45	11.26	14.23	15.70
7	ABR07	954	1187	1432	9.54	11.87	14.32	15.30
8	ABR08	976	1167	1465	9.76	11.67	14.65	15.80
9	ABR09	934	1187	1416	9.34	11.87	14.16	15.40
10	ABR10	967	1156	1453	9.67	11.56	14.53	15.60
11	ABR11	976	1156	1432	9.76	11.56	14.32	15.40
12	ABR12	976	1187	1421	9.76	11.87	14.21	15.60
13	ABR13	987	1134	1421	9.87	11.34	14.21	15.50
14	ABR14	934	1172	1439	9.34	11.72	14.39	15.20
15	ABR15	956	1132	1459	9.56	11.32	14.59	15.20
16	ABR16	965	1186	1462	9.65	11.86	14.62	15.50
17	ABR17	945	1156	1432	9.45	11.56	14.32	15.80
18	ABR18	978	1176	1252	9.78	11.76	12.52	15.60
19	ABR19	976	1105	1192	9.76	11.05	11.92	15.40
20	ABR20	451	798	1153	4.51	7.98	11.53	15.20

Table B5: Reading of Three Point Bending Test for Afromosia Timber

S/No.	Specimen Number	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load F_{max} (KN)
		2 000N	4 000N	6 000N	2 000N	4 000N	6 000N	
1	AFR01	967	1145	1265	9.67	11.45	12.65	21.30
2	AFR02	967	1132	1230	9.67	11.32	12.30	21.30
3	AFR03	956	1156	1230	9.56	11.56	12.30	21.90
4	AFR04	976	1127	1265	9.76	11.27	12.65	21.40
5	AFR05	987	1143	1278	9.87	11.43	12.78	21.80
6	AFR06	945	1165	1282	9.45	11.65	12.82	21.90
7	AFR07	974	1135	1237	9.74	11.35	12.37	21.90
8	AFR08	976	1164	1232	9.76	11.64	12.32	21.90
9	AFR09	955	1139	1256	9.55	11.39	12.56	21.60
10	AFR10	975	1187	1275	9.75	11.87	12.75	21.40
11	AFR11	976	1167	1265	9.76	11.67	12.65	21.60
12	AFR12	975	1158	1245	9.75	11.58	12.45	21.70
13	AFR13	975	1176	1228	9.75	11.76	12.28	21.70
14	AFR14	943	1165	1265	9.43	11.65	12.65	22.20
15	AFR15	966	1132	1276	9.66	11.32	12.76	21.90
16	AFR16	953	1167	1245	9.53	11.67	12.45	21.70
17	AFR17	944	1185	1252	9.44	11.85	12.52	21.40
18	AFR18	976	1165	1265	9.76	11.65	12.65	22.00
19	AFR19	988	1187	1267	9.88	11.87	12.67	22.40
20	AFR20	945	1175	1258	9.45	11.75	12.58	21.80

Table B6: Reading of Three Point Bending Test for Berlinia Timber

S/No.	Specimen Number	Dial Gauge reading caused by Loads (mm)			Deformation caused by Loads (mm)			Maximum Load
		2 000N	4 000N	6 000N	2 000N	4 000N	6 000N	F_{max} (KN)
1	BERLI01	796	987	1273	7.96	9.87	12.73	12.4
2	BERLI02	791	982	1265	7.91	9.82	12.65	11.8
3	BERLI03	793	984	1269	7.93	9.84	12.69	12.1
4	BERLI04	789	980	1262	7.89	9.80	12.62	11.6
5	BERLI05	792	983	1268	7.92	9.83	12.68	12.0
6	BERLI06	795	986	1272	7.95	9.86	12.72	12.3
7	BERLI07	789	980	1263	7.89	9.80	12.63	11.7
8	BERLI08	796	987	1273	7.96	9.87	12.73	12.4
9	BERLI09	791	982	1265	7.91	9.82	12.65	11.8
10	BERLI10	789	980	1262	7.89	9.80	12.62	11.6
11	BERLI11	790	981	1263	7.90	9.81	12.63	11.7
12	BERLI12	793	984	1269	7.93	9.84	12.69	12.1
13	BERLI13	791	982	1266	7.91	9.82	12.66	11.9
14	BERLI14	788	979	1261	7.88	9.79	12.61	11.5
15	BERLI15	787	978	1260	7.87	9.78	12.60	11.5
16	BERLI16	791	982	1265	7.91	9.82	12.65	11.9
17	BERLI17	795	986	1272	7.95	9.86	12.72	12.3
18	BERLI18	793	984	1269	7.93	9.84	12.69	12.1
19	BERLI19	790	981	1264	7.90	9.81	12.64	11.7
20	BERLI20	788	979	1260	7.88	9.79	12.60	11.5

Table B7: Determination of 4 Point Bending Strength for Abura from the test data

Specimen Number	a (mm)	Fmax (N)	aFmax (Nmm)	b (mm)	H (mm)	h^2 (mm^2)	W (mm^3)	2W (mm^3)	f_m (N/mm^2)
ABR01	300	17.90	5370	41	75	5625	38438	76875	*69.85*
ABR02	300	17.50	5250	41	74	5476	37419	74839	*70.15*
ABR03	300	17.60	5280	40	74	5476	36507	73013	*72.32*
ABR04	300	17.30	5190	42	76	5776	40432	80864	*64.18*
ABR05	300	17.60	5280	42	75	5625	39375	78750	*67.05*
ABR06	300	17.80	5340	41	74	5476	37419	74839	*71.35*
ABR07	300	17.30	5190	40	76	5776	38507	77013	*67.39*
ABR08	300	17.90	5370	41	74	5476	37419	74839	*71.75*
ABR09	300	17.40	5220	42	76	5776	40432	80864	*64.55*
ABR10	300	17.30	5190	40	75	5625	37500	75000	*69.20*
ABR11	300	17.40	5220	40	75	5625	37500	75000	*69.60*
ABR12	300	17.60	5280	41	74	5476	37419	74839	*70.55*
ABR13	300	17.50	5250	41	73	5329	36415	72830	*72.09*
ABR14	300	17.20	5160	41	76	5776	39469	78939	*65.37*
ABR15	300	17.20	5160	40	75	5625	37500	75000	*68.80*
ABR16	300	17.50	5250	40	74	5476	36507	73013	*71.90*
ABR17	300	17.80	5340	42	74	5476	38332	76664	*69.65*
ABR18	300	17.60	5280	41	74	5476	37419	74839	*70.55*
ABR19	300	17.40	5220	40	73	5329	35527	71053	*73.47*
ABR20	300	17.20	5160	42	75	5625	39375	78750	*65.52*

Table B8: Determination of 4 Point Bending Strength for Afromosia from the test data

Specimen Number	a (mm)	Fmax (N)	aFmax (Nmm)	b (mm)	H (mm)	h^2 (mm^2)	W (mm^3)	2W (mm^3)	f_m (N/mm^2)
AFR01	300	23.40	7020	41	75	5625	38438	76875	*91.32*
AFR02	300	25.30	7590	41	74	5476	37419	74839	*101.42*
AFR03	300	24.40	7320	40	74	5476	36507	73013	*100.26*
AFR04	300	24.50	7350	42	76	5776	40432	80864	*90.89*
AFR05	300	25.40	7620	42	75	5625	39375	78750	*96.76*
AFR06	300	24.40	7320	41	74	5476	37419	74839	*97.81*
AFR07	300	24.60	7380	40	76	5776	38507	77013	*95.83*
AFR08	300	24.60	7380	41	74	5476	37419	74839	*98.61*
AFR09	300	24.60	7380	42	76	5776	40432	80864	*91.26*
AFR10	300	24.70	7410	40	75	5625	37500	75000	*98.80*
AFR11	300	24.40	7320	40	75	5625	37500	75000	*97.60*
AFR12	300	24.50	7350	41	74	5476	37419	74839	*98.21*
AFR13	300	24.70	7410	41	73	5329	36415	72830	*101.74*
AFR14	300	24.60	7380	41	76	5776	39469	78939	*93.49*
AFR15	300	24.10	7230	40	75	5625	37500	75000	*96.40*
AFR16	300	25.00	7500	40	74	5476	36507	73013	*102.72*
AFR17	300	24.70	7410	42	74	5476	38332	76664	*96.66*
AFR18	300	24.80	7440	41	74	5476	37419	74839	*99.41*
AFR19	300	24.50	7350	40	73	5329	35527	71053	*103.44*
AFR20	300	25.10	7530	42	75	5625	39375	78750	*95.62*

Table B9: Determination of 4 Point Bending Strength for Berlinia from the test data

Specimen Number	a (mm)	Fmax (N)	aFmax (Nmm)	b (mm)	H (mm)	h^2 (mm^2)	W (mm^3)	2W (mm^3)	f_m (N/mm^2)
BERLI01	300	14.9	4478	41	75	5625	38438	76875	58.3
BERLI02	300	14.7	4419	40	74	5476	37419	74839	59.0
BERLI03	300	14.8	4447	40	74	5476	36507	73013	60.9
BERLI04	300	14.6	4394	42	76	5776	40432	80864	54.3
BERLI05	300	14.8	4436	40	75	5625	39375	78750	56.3
BERLI06	300	14.9	4467	40	74	5476	37419	74839	59.7
BERLI07	300	14.7	4400	40	76	5776	38507	77013	57.1
BERLI08	300	14.9	4477	41	74	5476	37419	74839	59.8
BERLI09	300	14.7	4417	42	76	5776	40432	80864	54.6
BERLI10	300	14.6	4394	41	75	5625	37500	75000	58.6
BERLI11	300	14.7	4406	40	75	5625	37500	75000	58.7
BERLI12	300	14.8	4446	41	74	5476	37419	74839	59.4
BERLI13	300	14.7	4423	41	75	5625	36415	72830	60.7
BERLI14	300	14.6	4386	40	76	5776	39469	78939	55.6
BERLI15	300	14.6	4379	41	75	5625	37500	75000	58.4
BERLI16	300	14.7	4420	42	74	5476	36507	73013	60.5
BERLI17	300	14.9	4470	40	74	5476	38332	76664	58.3
BERLI18	300	14.8	4447	42	74	5476	37419	74839	59.4
BERLI19	300	14.7	4408	41	73	5329	35527	71053	62.0
BERLI20	300	14.6	4381	42	75	5625	39375	78750	55.6

Table B10: Determination of 3 Point Bending Strength for Abura from the test data

Specimen Number	L (mm)	Fmax (N) X1000	LFmax (Nmm)	b (mm)	h (mm)	h^2 (mm^2)	$f_m = \frac{3f_{max}.L}{2bh^2}$ (N/mm^2)
ABR01	900	15.80	14220	40	75	5625	94.80
ABR02	900	15.50	13950	40	74	5476	95.53
ABR03	900	15.60	14040	40	74	5476	96.15
ABR04	900	15.30	13770	40	76	5776	89.40
ABR05	900	15.60	14040	40	75	5625	93.60
ABR06	900	15.70	14130	40	74	5476	96.76
ABR07	900	15.30	13770	40	76	5776	89.40
ABR08	900	15.80	14220	41	74	5476	95.00
ABR09	900	15.40	13860	41	76	5776	87.79
ABR10	900	15.60	14040	41	75	5625	91.32
ABR11	900	15.40	13860	41	75	5625	90.15
ABR12	900	15.60	14040	41	74	5476	93.80
ABR13	900	15.50	13950	41	73	5329	95.77
ABR14	900	15.20	13680	41	76	5776	86.65
ABR15	900	15.20	13680	41	75	5625	88.98
ABR16	900	15.50	13950	42	74	5476	90.98
ABR17	900	15.80	14220	42	74	5476	92.74
ABR18	900	15.60	14040	42	74	5476	91.57
ABR19	900	15.40	13860	42	73	5329	92.89
ABR20	900	15.20	13680	42	75	5625	86.86

Table B11: Determination of 3 Point Bending Strength for Afromosia from the test data

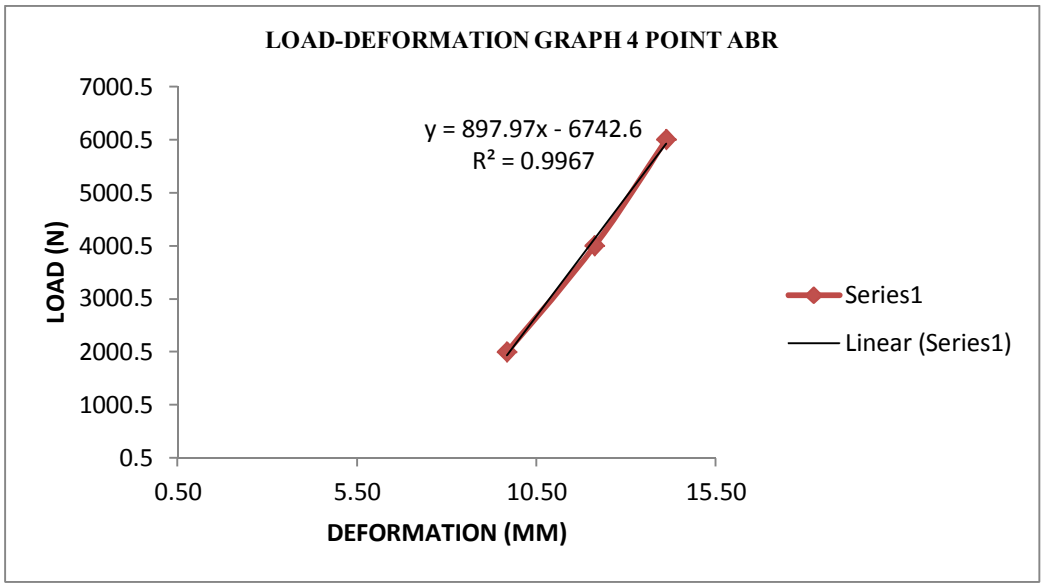
Specimen Number	L (mm)	Fmax (N) X1000	LFmax (Nmm)	b (mm)	h (mm)	h^2 (mm^2)	$f_m = \frac{3f_{max}L}{2bh^2}$ (N/mm^2)
AFR01	900	21.30	19170	41	75	5625	124.68
AFR02	900	21.30	19170	41	74	5476	128.08
AFR03	900	21.90	19710	40	74	5476	134.98
AFR04	900	21.40	19260	42	76	5776	119.09
AFR05	900	21.80	19620	42	75	5625	124.57
AFR06	900	21.90	19710	41	74	5476	131.68
AFR07	900	21.90	19710	40	76	5776	127.96
AFR08	900	21.90	19710	41	74	5476	131.68
AFR09	900	21.60	19440	42	76	5776	120.20
AFR10	900	21.40	19260	40	75	5625	128.40
AFR11	900	21.60	19440	40	75	5625	129.60
AFR12	900	21.70	19530	41	74	5476	130.48
AFR13	900	21.70	19530	41	73	5329	134.08
AFR14	900	22.20	19980	41	76	5776	126.55
AFR15	900	21.90	19710	40	75	5625	131.40
AFR16	900	21.70	19530	40	74	5476	133.74
AFR17	900	21.40	19260	42	74	5476	125.61
AFR18	900	22.00	19800	41	74	5476	132.28
AFR19	900	22.40	20160	40	73	5329	141.87
ABR20	900	25.10	22590	42	75	5625	143.43

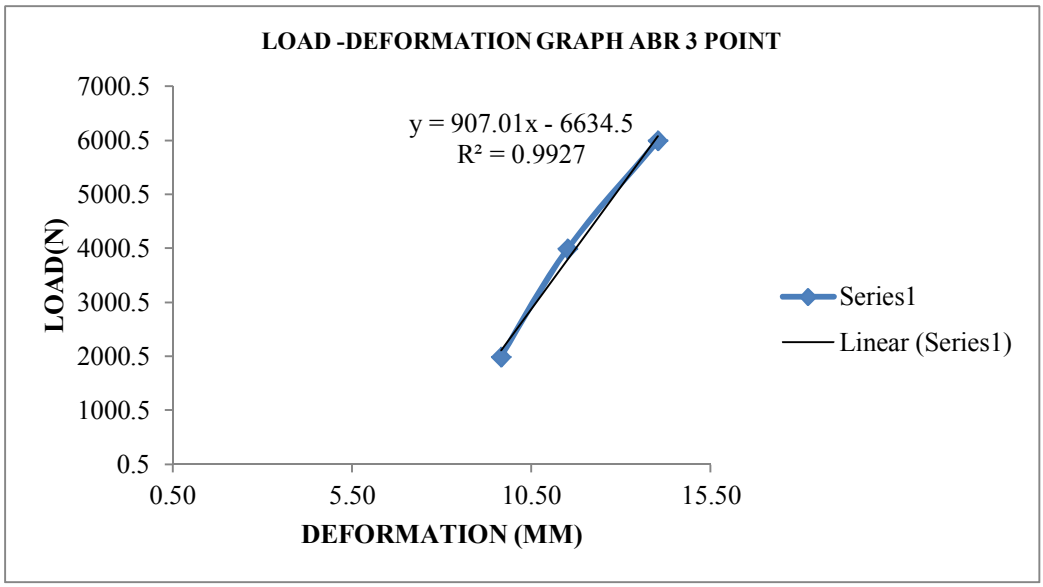
Table B12: Determination of 3 Point Bending Strength for Berlinia from the test data

Specimen Number	L (mm)	Fmax (N) X1000	LFmax (Nmm)	b (mm)	h (mm)	h^2 (mm^2)	$f_m = \frac{3f_{max}L}{2bh^2}$ (N/mm^2)
BERNI01	900	12.4	11157	40	75	5625	74.38
BERNI02	900	11.8	10655	41	74	5476	71.19
BERNI03	900	12.1	10895	42	74	5476	71.06
BERNI04	900	11.6	10450	40	76	5776	67.84
BERNI05	900	12.0	10805	41	75	5625	70.27
BERNI06	900	12.3	11061	42	74	5476	72.14
BERNI07	900	11.7	10501	41	76	5776	66.52
BERNI08	900	12.4	11145	41	74	5476	74.46
BERNI09	900	11.8	10642	40	76	5776	69.09
BERNI10	900	11.6	10447	42	75	5625	66.33
BERNI11	900	11.7	10551	40	75	5625	70.34
BERNI12	900	12.1	10883	42	74	5476	70.98
BERNI13	900	11.9	10695	41	73	5329	73.43
BERNI14	900	11.5	10385	40	76	5776	67.43
BERNI15	900	11.5	10321	41	75	5625	67.13
BERN16	900	11.9	10668	42	74	5476	69.58
BERNI17	900	12.3	11084	40	74	5476	75.90
BERNI18	900	12.1	10895	40	74	5476	74.61
BERNI19	900	11.7	10563	42	73	5329	70.79
BERNI20	900	12.30	11070	42	75	5625	70.29

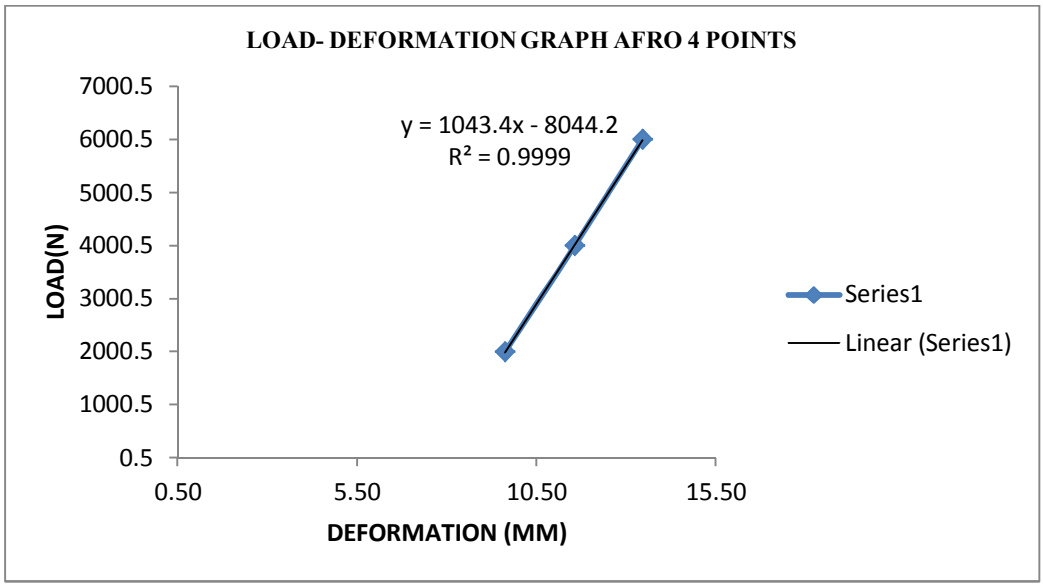
APPENDIX C
LOAD DEFORMATION GRAPHS

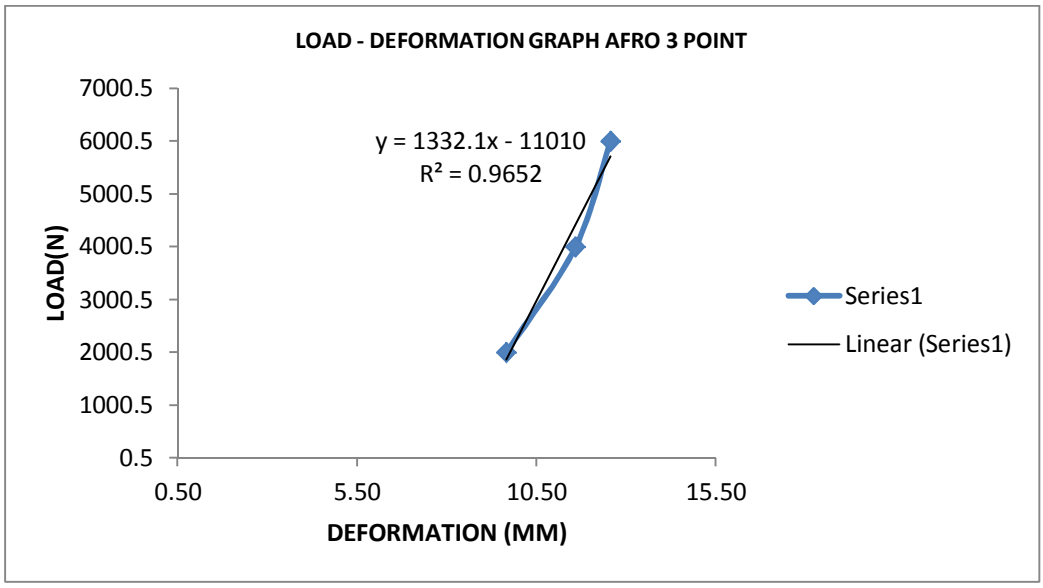
Abura Timber



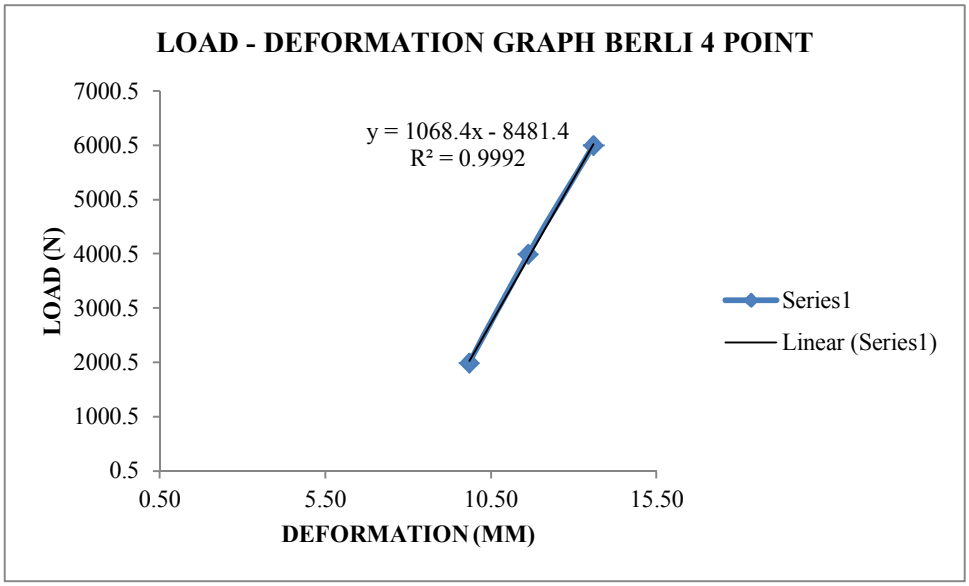


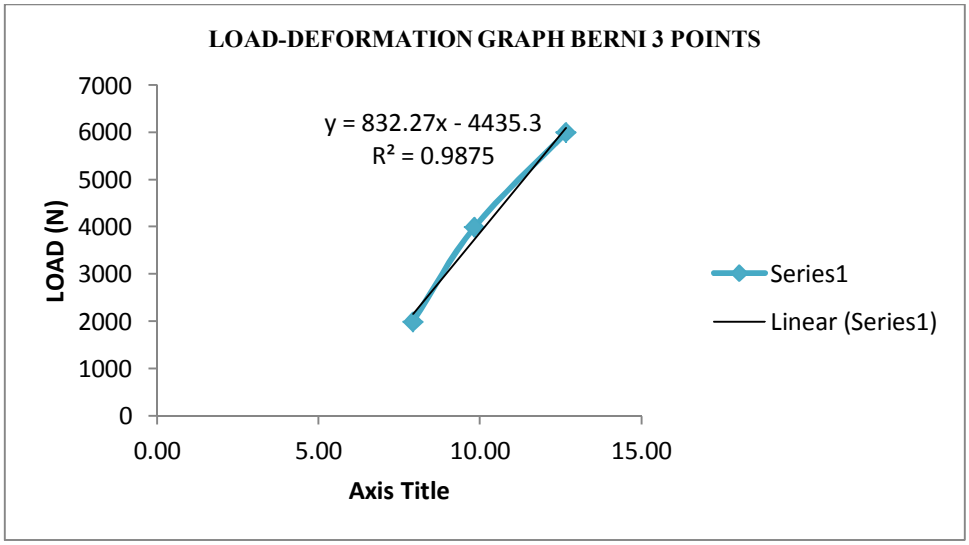
Afromosia Timber





Berlinia Timber





APPENDIX D: Table 1 of BS EN 338:2009(E)

Table 1: Strength classes –

		Softwood specie												Hardwood specie							
		C14	**C16**	**C18**	**C20**	**C22**	**C24**	**C27**	**C30**	**C35**	**C40**	**C45**	**C50**	**D18**	**D24**	**D30**	**D35**	**D40**	**D50**	**D60**	**D70**
Strength Properties (N/mm^2)																					
Bending	f_m	14	16	18	20	22	24	27	30	35	40	45	50	18	24	30	35	40	50	60	70
	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	27	30	11	14	18	21	24	30	36	42
Tension paral	$f_{t,90,k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Stiffness Properties (kN/mm^2)																					
Mean modulus of elasti	$E_{0,mean}$	7	8	9	9.5	10	11	11.5	12	13	14	15	16	9.5	10	11	12	13	14	17	20
	$E_{0.05}$	4.7	5.4	6.0	6.4	6.7	7.4	7.7	8.0	8.7	9.4	10.0	10.7	8.0	8.5	9.2	10.1	10.9	11.8	14.3	16.8
Density (Kg/m^3)																					
Density	ρ_k	290	310	320	330	340	350	370	380	400	420	440	460	475	485	530	540	550	620	700	900

NOTE 1: Values given above are for tension strength, compression strength, shear strength, 5% modulus of elasticity, mean modulus of elasticity perpendicular to grain and mean shear modulus have been calculated using equations given in Annex A.

NOTE 2: The tabulated properties are compatible with timber at moisture content consistent with a temperature of 20^0C and relative humidity of 65%

Source: EN338 (2009)

APPENDIX E
First Order Reliability Programs for the Six Failure Modes

FAILURE MODE ONE: COMPRESSION CRITERION

```
PROGRAM COLUMN COMPRESSION
IMPLICIT DOUBLE PRECISION (A-H,O-Z)
COMMON XX(5),EX(5),IER
COMMON FX
```

```
      COMMON FORM5,ICRT
      COMMON/GKGOOD/BETA,ALPHA
      OPEN(200,FILE='AI.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(200,FILE='AI.RES',STATUS='NEW')
20    WRITE(200,30)
30    FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
     +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
     +5X,'BUCKLING ')
      WRITE(*,*)'   INPUT DEAD TO LIVE LOAD RATIO'
      READ(*,*)ALPHA
      WRITE(*,*)'   INPUT CHARACTERISTIC COMP STRENGTH IN N/MM '
      READ(*,*)FCOK
      WRITE(*,*)'   INPUT IMPOSED LOAD IN N'
      READ(*,*)QK
      WRITE(*,*)'   INPUT LENGTH OF PORTAL FRAME IN MM '
      READ(*,*)XL
      WRITE(*,*)'   INPUT WIDTH OF COLUMN IN MM'
      READ(*,*)XB
      WRITE(*,*)'   INPUT DEPTH OF COLUMN IN MM'
      READ(*,*)XH
C
      EX(1) = FCOK
      EX(2) = QK
      EX(3) = XL
      EX(4) = XB
      EX(5) = XH
```

```
      N5 =5
      CALL KGOOD(N5,EX)
      STOP
      END

      SUBROUTINE KGOOD(N5,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N5)
      DIMENSION XX(5),SX(5),VP(10,5),COV(5,5),ZES(3),
     +          UU(7),EIVEC(5,5),IV(2,5)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/5/,NC/5/,NE/5/,IRHO/0/
      SX(1) = EX(1)*1.3D-1
      SX(2) = EX(2)*4.0D-1
      SX(3) = EX(3)*6.0D-2
      SX(4) = EX(4)*6.0D-2
      SX(5) = EX(5)*6.0D-2
      NAUS = 200
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
      IV(1,1) = 3
```

```
      IV(1,2) = 7
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
```

```
      PRT = 'ZES'
      CALL YFAUS(NAUS,3,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(NAUS,*)BETA
      WRITE(ICRT,*)' THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
      COMMON/GKGOOD/BETA,ALPHA
      IF(XX(1).GT.0)THEN
      FX1 = (0.6*XX(1))/1.3
      FX2 = (XX(2)*XX(3)*(1.35*ALPHA+1.5))
      FX3 = (2.0*XX(4)*XX(5))
      FX = (FX1 - (FX2/FX3))
      IER = 0
      ELSE
      FX = 1.D+20
      IER = 1
      ENDIF
      RETURN
      END
```

FAILURE MODE TWO: BENDING CRITERION

```
      PROGRAM COLUMN BENDING
IMPLICIT DOUBLE PRECISION (A-H,O-Z)
COMMON XX(6),EX(6),IER
COMMON FX
COMMON FORM5,ICRT
COMMON/GKGOOD/BETA,ALPHA
OPEN(200,FILE='FIFI.RES',STATUS='OLD',ERR=10)
GOTO 20
10  OPEN(200,FILE='FIFI.RES',STATUS='NEW')
20  WRITE(200,30)
30  FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
   +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
   +5X,'BUCKLING ')
 WRITE(*,*)'  INPUT DEAD TO LIVE LOAD RATIO'
 READ(*,*)ALPHA
 WRITE(*,*)'  INPUT CHARACTERISTIC COMP STRENGTH IN N/MM '
 READ(*,*)FMK
 WRITE(*,*)'  INPUT LENGTH OF PORTAL FRAME IN MM'
 READ(*,*)XL
 WRITE(*,*)'  INPUT IMPOSED LOAD IN N'
 READ(*,*)QK
 WRITE(*,*)'  INPUT HEIGHT OF COLUMN IN MM'
 READ(*,*)Xh1
 WRITE(*,*)'  INPUT WIDTH OF COLUMN IN MM'
 READ(*,*)XB
 WRITE(*,*)'  INPUT DEPTH OF COLUMN IN MM'
```

```
      READ(*,*)XH
C
      EX(1) = FMK
      EX(2) = XL
      EX(3) = QK
      EX(4) = Xh1
      EX(5) = XB
      EX(6) = XH
      N6 =6
      CALL KGOOD(N6,EX)
      STOP
      END

      SUBROUTINE KGOOD(N6,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N6)
      DIMENSION XX(6),SX(6),VP(10,6),COV(6,6),ZES(3),
     +          UU(7),EIVEC(6,6),IV(2,6)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/6/,NC/6/,NE/6/,IRHO/0/
      SX(1) = EX(1)*2.9D-1
      SX(2) = EX(2)*6.0D-2
      SX(3) = EX(3)*4.0D-1
      SX(4) = EX(4)*6.0D-2
```

```
      SX(5) = EX(5)*6.0D-2
      SX(5) = EX(6)*6.0D-2
      NAUS = 200
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
      IV(1,1) = 3
      IV(1,2) = 7
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
```

```
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
      PRT = 'ZES'
      CALL YFAUS(NAUS,3,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(NAUS,*)BETA
      WRITE(ICRT,*)' THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
      COMMON/GKGOOD/BETA,ALPHA
      IF(XX(1).GT.0)THEN
      FX1 = (0.6*XX(1))/1.3
      FX2 = (3.0*XX(2)*XX(3)*XX(4)*(1.35*ALPHA+1.5))
      FX3 = (4.0*XX(5)*(XX(6)**2))
```

```
FX = (FX1 - (FX2/FX3))

IER = 0

ELSE

FX = 1.D+20

IER = 1

ENDIF

RETURN

END
```

FAILURE MODE THREE: FLEXURAL BUCKLING CRITERION

```
      PROGRAM COLUMN FLEXURAL BUCKLING
     IMPLICIT DOUBLE PRECISION (A-H,O-Z)
     COMMON XX(7),EX(7),IER
     COMMON FX
     COMMON FORM5,ICRT
     COMMON/GKGOOD/BETA,ALPHA
     OPEN(200,FILE='ABU.RES',STATUS='OLD',ERR=10)
     GOTO 20
10    OPEN(200,FILE='ABU.RES',STATUS='NEW')
20    WRITE(200,30)
30    FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
     +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
     +5X,'BUCKLING ')
     WRITE(*,*)'   INPUT DEAD TO LIVE LOAD RATIO'
     READ(*,*)ALPHA
     WRITE(*,*)'   INPUT LENGTH OF PORTAL FRAME IN MM'
     READ(*,*)XL
     WRITE(*,*)'   INPUT IMPOSED LOAD IN N'
     READ(*,*)QK
     WRITE(*,*)'   INPUT HEIGHT OF COLUMN IN MM '
     READ(*,*)Xh1
     WRITE(*,*)'   INPUT WIDTH OF COLUMN IN MM'
     READ(*,*)XB
     WRITE(*,*)'   INPUT DEPTH OF COLUMN IN MM'
     READ(*,*)XH
     WRITE(*,*)'   INPUT BENDING STRENGTH PARALLEL TO GRAIN'
```

```
      READ(*,*)FMD
      WRITE(*,*)'   INPUT COMPRESSIVE STRENGTH PARALLEL TO GRAIN'
      READ(*,*)FCOD
C
      EX(1) = XL
      EX(2) = QK
      EX(3) = Xh1
      EX(4) = XB
      EX(5) = XH
      EX(6) = FMD
      EX(7) = FCOD
      N7 = 7
      CALL KGOOD(N7,EX)
      STOP
      END

      SUBROUTINE KGOOD(N7,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N7)
      DIMENSION XX(7),SX(7),VP(10,7),COV(7,7),ZES(3),
     +          UU(7),EIVEC(7,7),IV(2,7)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/7/,NC/7/,NE/7/,IRHO/0/
      SX(1) = EX(1)*6.0D-2
```

```
      SX(2) = EX(2)*4.0D-1
      SX(3) = EX(3)*6.0D-2
      SX(4) = EX(4)*6.0D-2
      SX(5) = EX(5)*6.0D-2
      SX(6) = EX(6)*3.7D-2
      SX(7) = EX(7)*1.3D-1
      NAUS = 200
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
      IV(1,2) = 7
      IV(1,6) = 3
      IV(1,7) = 3
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
```

```
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
      PRT = 'ZES'
      CALL YFAUS(NAUS,3,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(NAUS,*)BETA
      WRITE(ICRT,*)' THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
```

```
COMMON/GKGOOD/BETA,ALPHA
IF(XX(1).GT.0)THEN
FX1 = (3*XX(1)*XX(3)*(1.35*ALPHA+1.5))
FX2 = (4*XX(4)*(XX(5)**2)*XX(6))
FX3 = (XX(2)*XX(1)*(1.35*ALPHA+1.5))
FX4 = (2*XX(4)*XX(5)*0.12*XX(7))
FX = 1.0 - ((FX1/FX2) + (FX3/FX4))
IER = 0
ELSE
FX = 1.D+20
IER = 1
ENDIF
RETURN
END
```

FAILURE MODE FOUR: LATERAL TORSIONAL BUCKLING CRITERION

```
      PROGRAM COLUMN BUCKLING
     IMPLICIT DOUBLE PRECISION (A-H,O-Z)
     COMMON XX(7),EX(7),IER
     COMMON FX
     COMMON FORM5,ICRT
     COMMON/GKGOOD/BETA,ALPHA
     OPEN(100,FILE='ABUBAKAR.RES',STATUS='OLD',ERR=10)
     GOTO 20
10    OPEN(100,FILE='ABUBAKAR.RES',STATUS='NEW')
20    WRITE(100,30)
30    FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
     +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
     +5X,'BUCKLING ')
     WRITE(*,*)'   INPUT DEAD TO LIVE LOAD RATIO'
     READ(*,*)ALPHA
     WRITE(*,*)'   INPUT IMPOSE LOAD IN NEWTON'
     READ(*,*)QK
     WRITE(*,*)'   INPUT WIDTH OF COLUMN IN mm'
     READ(*,*)XB
     WRITE(*,*)'   INPUT DEPTH OF COLUMN IN mm '
     READ(*,*)XH
     WRITE(*,*)'   INPUT LENGHT OF PORTAL FRAME IN MM'
     READ(*,*)XL
     WRITE(*,*)'   INPUT BENDING STRENGTH PARALLEL TO GRAIN'
     READ(*,*)FMD
     WRITE(*,*)'   INPUT HEIGHT OF COLUMN IN MM'
```

```
      READ(*,*)Xh1
      WRITE(*,*)'   INPUT COMPRESSIVE STRENGHT PARALLEL TO GRAIN'
      READ(*,*)FCK
C
      EX(1) = QK
      EX(2) = XB
      EX(3) = XH
      EX(4) = XL
      EX(5) = FMD
      EX(6) = Xh1
      EX(7) = FCK
      N7 = 7
      CALL KGOOD(N7,EX)
      STOP
      END

      SUBROUTINE KGOOD(N7,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N7)
      DIMENSION XX(7),SX(7),VP(10,7),COV(7,7),ZES(3),
     +          UU(7),EIVEC(7,7),IV(2,7)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/7/,NC/7/,NE/7/,IRHO/0/
      SX(1) = EX(1)*4.0D-1
```

```
      SX(2) = EX(2)*6.0D-2
      SX(3) = EX(3)*6.0D-2
      SX(4) = EX(4)*6.0D-2
      SX(5) = EX(5)*3.7D-2
      SX(6) = EX(6)*6.0D-2
      SX(7) = EX(7)*1.3D-1
      NAUS = 100
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
      IV(1,1) = 7
      IV(1,5) = 3
      IV(1,7) = 3
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
```

```
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
      PRT = 'ZES'
      CALL YFAUS(NAUS,3,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(100,*)BETA
      WRITE(ICRT,*)'  THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
```

```
COMMON/GKGOOD/BETA,ALPHA
IF(XX(1).GT.0)THEN
FX1 = (3.0*XX(4)*XX(1)*XX(6)*(1.35*ALPHA+1.5)**2)
FX2 = (4.0*XX(2)*(XX(3)**2)*0.17*XX(5))**2
FX3 = (XX(1)*XX(4)*(1.35*ALPHA+1.5))
FX4 = (2.0*XX(2)*XX(3)*0.12*XX(7))
FX = 1.0 - ((FX1/FX2) + (FX3/FX4))
IER = 0
ELSE
FX = 1.D+20
IER = 1
ENDIF
RETURN
END
```

FAILURE MODE FIVE: CARRYING CAPACITY OF DOWEL CRITERION

```
      PROGRAM CARRYING CAPACITY OF DOWEL
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      COMMON XX(3),EX(3),IER
      COMMON FX
      COMMON FORM5,ICRT
      COMMON/GKGOOD/BETA,ALPHA
      OPEN(200,FILE='ZAINAB.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(200,FILE='ZAINAB.RES',STATUS='NEW')
20    WRITE(200,30)
30    FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
     +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
     +5X,'BUCKLING ')
      WRITE(*,*)'   INPUT DEAD TO LIVE LOAD RATIO'
      READ(*,*)ALPHA
      WRITE(*,*)'   INPUT LENGTH OF PORTAL FRAME'
      READ(*,*)XL
      WRITE(*,*)'   INPUT INPOSED LOAD IN N'
      READ(*,*)QK
      WRITE(*,*)'   INPUT HEIGHT OF PORTAL FRAME'
      READ(*,*)Xh1
C
      EX(1) = XL
      EX(2) = QK
      EX(3) = Xh1
      N3 =3
```

```
      CALL KGOOD(N3,EX)
      STOP
      END

      SUBROUTINE KGOOD(N3,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N3)
      DIMENSION XX(3),SX(3),VP(10,3),COV(3,3),ZES(3),
     +          UU(7),EIVEC(3,3),IV(2,3)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/3/,NC/3/,NE/3/,IRHO/0/
      SX(1) = EX(1)*6.0D-2
      SX(2) = EX(2)*4.0D-1
      SX(3) = EX(3)*6.0D-2
      NAUS = 200
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
      IV(1,2) = 7
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
```

```
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
      PRT = 'ZES'
      CALL YFAUS(NAUS,3,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(NAUS,*)BETA
```

```
      WRITE(ICRT,*)' THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
      COMMON/GKGOOD/BETA,ALPHA
      IF(XX(1).GT.0)THEN
      FX1 = (XX(1)*XX(2)*XX(3)*(1.35*ALPHA+1.5)*504)
      FX2 = (8.0*(22*(504**2)+17*(384**2)))
      FX3 = FX1/FX2
      FX4 = (1000*XX(2)*(1.35*ALPHA+1.5))/(22+17)
      FX5 = (FX3+FX4)**2
      FX6 = (XX(1)*XX(2)*(1.35*ALPHA+1.5))/(2.0*(22+17))
      FX7 = (FX6**2)
      FX8 = (2.0*0.6*21340)/1.3
      FX = FX8- (FX5+FX7)**0.5
      IER = 0
      ELSE
      FX = 1.D+20
      IER = 1
      ENDIF
      RETURN
      END
```

FAILURE MODE SIX: SHEAR STRESS CRITERION

```
PROGRAM SHEAR STRESS FAILURE
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      COMMON XX(4),EX(4),IER
      COMMON FX
      COMMON FORM5,ICRT
      COMMON/GKGOOD/BETA,ALPHA
      OPEN(200,FILE='MARYAM.RES',STATUS='OLD',ERR=10)
      GOTO 20
10    OPEN(200,FILE='MARYAM.RES',STATUS='NEW')
20    WRITE(200,30)
30    FORMAT(5X,'A PROGRAM BY ABUBAKAR MAMMAN',/,
     +5X,'PROBABILISTIC SAFETY CHECK ON TB COLUMN',/,
     +5X,'BUCKLING ')
      WRITE(*,*)'   IMPUT DEAD TO LIFE LOAD RATIO'
      READ(*,*)ALPHA
      WRITE(*,*)'    IMPUT BENDING MOMENT'
      READ(*,*)VM
      WRITE(*,*)'   IMPOSED LOAD IN N'
      READ(*,*)QK
      WRITE(*,*)'   WIDTH OF COLUMN'
      READ(*,*)Xb
      WRITE(*,*)'   DEPTH OF COLUMN'
      READ(*,*)Xh
C
      EX(1) = VM
      EX(2) = QK
```

```
      EX(3) = Xb
      EX(4) = Xh
      N4=4
      CALL KGOOD(N4,EX)
      STOP
      END

      SUBROUTINE KGOOD(N4,EX)
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
C     LIMIT STATE FUNCTION IS DECLARED EXTERNAL
      EXTERNAL QGOOD
      DIMENSION EX(N4)
      DIMENSION XX(4),SX(4),VP(10,4),COV(4,4),ZES(4),
     +          UU(7),EIVEC(4,4),IV(4,4)
      CHARACTER*10 PRT
      COMMON/GKGOOD/BETA,ALPHA
      DATA N/4/,NC/4/,NE/4/,IRHO/0/
      SX(1) = EX(1)*1.5D-1
      SX(2) = EX(1)*4.0D-1
      SX(3) = EX(2)*6.0D-2
      SX(4) = EX(3)*6.0D-2
      NAUS = 200
C     PRINT TO SCREEN
      ICRT = 0
C     PRESETING THE VARIABLES VP,COV,AND IV DONE
C     USING YINIT
      CALL YINIT(N,IV,VP,IRHO,COV,NC)
```

```
      IV(1,1) = 3
      IV(1,2)= 7
      DO 50 I = 1,N
50    XX(I) = EX(I)
      V1 = 1.0D0
      BETA = 1.0D0
      WRITE(NAUS,5000)
5000  FORMAT(////,5X,70('-'),30X,'FORM5',/,5X,70('-'),/,5X,
     +'RELABILITY-BASE DESIGN')
C     STOCHASTIC MODEL IS PRINTED USING "YKOPF"
      CALL YKOPF(NAUS,N,IV,EX,SX,VP,IRHO)
C     PRINT ALSO TO SCREEN
      WRITE(ICRT,*)'   START OF FORM5'
      WRITE(ICRT,*)'   STOCHASTIC MODEL'
      CALL YKOPF(ICRT,N,IV,EX,SX,VP,IRHO)
C     THE MATRIX COV IS PRINTED USING "YMAUS"
      PRT = ' COV '
      CALL YMAUS(NAUS,NC,N,COV,PRT)
C     THE MAIN SUBROUTINE IN FORM IS CALLED
      CALL FORM5(N,IV,EX,SX,VP,QGOOD,IRHO,COV,NC,
     +          EIVEC,NE,V1,NAUS,BETA,XX,UU,ZES,IER)
C     THE COORDINATES OF THE BETA-POINT ARE PRINTED
C     WITH THE TITLE UU
      PRT = 'UU'
      CALL YFAUS(NAUS,N,UU,PRT)
C     COMPUTE SAFETY INDEX AND CORRESPONDING PROBABILITY
C     OF FAILURE
```

```
C     THE VECTOR ZES IS PRINTED WITH THE TITLE ZES
      PRT = 'ZES'
      CALL YFAUS(NAUS,4,ZES,PRT)
      WRITE(ICRT,*)'   END OF FORM5: IER =',IER
      WRITE(NAUS,*)BETA
      WRITE(ICRT,*)' THE FOLLOWING IS THE SAFETY INDEX'
      WRITE(ICRT,*)BETA
      RETURN
      END

      SUBROUTINE QGOOD (N,XX,FX,IER)
C     LIMIT STATE FUNCTION
      IMPLICIT DOUBLE PRECISION (A-H,O-Z)
      DIMENSION XX(N)
      COMMON/GKGOOD/BETA,ALPHA
      IF(XX(1).GT.0)THEN
      FX1 = (0.6*2.0)/1.3
      FX2 = (2.0*XX(1)-(XX(2)*(1.35*ALPHA+1.5)*1000))
      FX3 = (4.0*XX(3)*XX(4))
      FX = (FX1 -((3/2)* (FX2/FX3)))
      IER = 0
      ELSE
      FX = 1.D+20
      IER = 1
      ENDIF
      RETURN
      END
```

APPENDIX F
Established Tables of Safety Indices

ABURA TIMBER

Table F1:Abura Timber Specie Compression Criterion β-Values (α=0.8)

	B(width of column)							
H(depth of column mm	300	350	400	450	500	550	600	650
900	2.88	3.92	4.7	5.33	5.86	6.32	6.74	7.11
950	3.02	4.06	4.85	5.48	6.02	6.49	6.91	7.28
1000	3.14	4.2	4.99	5.63	6.18	6.65	7.07	7.45
1050	3.27	4.33	5.12	5.78	6.32	6.8	7.23	7.61
1100	3.39	4.46	5.26	5.91	6.47	6.95	7.38	7.76
1150	3.5	4.58	5.39	6.05	6.61	7.09	7.52	7.91
1200	3.61	4.7	5.51	6.18	6.74	7.23	7.66	8.05
1250	3.72	4.81	5.63	6.3	6.87	7.36	7.79	8.19

Table F2:Abura Timber Specie Bending Criterion β-Values (α=0.8)

	B (width of column mm)							
H (depth of column mm)	100	150	200	250	300	350	400	450
900	-0.22	-0.12	0.46	1.04	1.61	2.17	2.72	3.24
950	-0.30	0.25	0.68	1.28	1.87	2.45	3.02	3.57
1000	-0.33	0.41	0.87	1.48	2.09	2.68	3.27	3.84
1050	-0.54	0.72	1.03	1.65	2.27	2.87	3.47	4.66
1100	-0.92	0.54	1.16	1.79	2.40	3.02	3.62	4.22
1150	0.23	0.65	1.27	1.89	2.51	3.12	3.72	4.32
1200	0.12	0.74	1.35	1.97	2.58	3.18	3.79	4.39
1250	0.21	0.81	1.42	2.02	2.62	3.22	3.82	4.41

Table F3 :Abura Timber Specie Flexural Buckling Criterion β -Values (α=0.2)

H(depth of column mm)	B (width of column mm)							
	100	150	200	250	300	350	400	450
1100	1.86	2.26	2.61	2.92	3.21	3.47	3.71	3.93
1150	1.98	2.38	2.73	3.04	3.33	3.59	3.83	4.06
1200	2.09	2.49	2.88	3.16	3.44	3.71	3.96	4.19
1250	2.2	2.6	2.95	3.27	3.56	3.83	4.07	4.31
1300	2.3	2.7	3.06	3.38	3.67	3.94	4.19	4.42
1350	2.4	2.8	3.16	3.48	3.78	4.05	4.3	4.54
1400	2.49	2.9	3.26	3.58	3.88	4.15	4.41	4.65
1450	2.59	2.99	3.36	3.68	3.98	4.25	4.51	4.75

Table F4:Abura Timber Specie Lateral Torsional Buckling Criterion β-Values (α=0.2)

	B(width of column)							
H(Depth of column	300	350	400	450	500	550	600	650
1100	1.9	2.29	2.64	2.95	3.23	3.49	3.73	3.96
1150	2.01	2.41	2.75	3.07	3.35	3.61	3.85	4.08
1200	2.12	2.51	2.87	3.18	3.47	3.73	3.97	4.2
250	2.23	2.62	2.97	3.29	3.58	3.84	4.09	4.32
1300	2.33	2.73	3.08	3.4	3.69	3.96	4.2	4.44
1350	2.43	2.83	3.18	3.5	3.8	4.06	4.31	4.55
1400	2.52	2.92	3.28	3.6	3.89	4.17	4.42	4.66
1450	2.61	3.01	3.37	3.7	3.99	4.27	4.53	4.77

AFROMOSIA TIMBER

Table F5: Afromosia Timber Specie Compression Criterion β -Values (α=0.8)

	B(width of column)							
H(depth of column)	100	150	200	250	300	350	400	450
900	4.17	5.4	6.36	7.15	7.82	8.41	8.94	9.42
950	4.33	5.58	6.55	7.34	8.03	8.62	9.16	9.64
1000	4.48	5.75	6.73	7.53	8.22	8.83	9.44	9.85
1050	4.63	5.91	6.9	7.71	8.41	9.02	9.57	10.06
1100	4.77	6.06	7.06	7.89	8.59	9.21	9.76	10.25
1150	4.9	6.21	7.23	8.06	8.77	9.39	9.94	10.44
1200	5.04	6.38	7.3	8.22	8.94	9.57	10.12	10.62
1250	5.16	6.5	7.53	8.38	9.1	9.74	10.3	10.8

Table F6: Afromosia Timber Specie Bending Criterion β-Values (α=0.8)

	B(width of column)							
H(depth of column)	100	150	200	250	300	350	400	450
900	-0.15	0.62	1.33	2.03	2.72	3.4	4.07	4.71
950	0.3	0.78	1.49	2.2	2.9	3.59	4.27	4.94
1000	0.19	0.91	1.62	2.33	3.03	3.72	4.41	5.08
1050	0.3	1.01	1.71	2.41	3.11	3.8	4.48	5.16
1100	0.39	1.08	1.77	2.46	3.15	3.82	4.51	5.18
1150	0.47	1.14	1.81	2.49	3.16	3.82	4.49	5.15
1200	0.52	1.18	1.84	2.49	3.14	3.8	4.45	5.1
1250	0.57	1.21	1.84	2.48	3.12	3.75	4.39	5.02

Table F7:Afromosia Timber Specie Flexural Buckling Criterion β-Values (α=0.2)

	B(width of column mm)							
H(depth of column mm)	300	350	400	450	500	550	600	650
1100	2.39	2.79	3.15	3.43	3.76	4.03	4.28	4.52
1150	2.5	2.9	3.27	3.59	3.89	4.16	4.42	4.66
1200	2.62	3.02	3.38	3.71	4.01	4.29	4.54	4.79
1250	2.72	3.13	3.5	3.83	4.13	4.41	4.67	4.91
1300	2.83	3.24	3.61	3.94	4.24	4.52	4.79	5.03
1350	2.93	3.34	3.71	4.05	4.35	4.64	4.9	5.15
1400	3.03	3.44	3.82	4.15	4.46	4.75	5.01	5.27
1450	3.12	3.54	3.92	4.26	4.57	4.86	5.13	5.38

Table F8:Afromosia Timber Specie Lateral Torsional Buckling Criterion β -Values (α=0.2)

	B(width of column mm)							
H(depth of column)	300	350	400	450	500	550	600	650
1100	2.41	2.81	3.17	3.49	3.78	4.05	4.3	4.54
1150	2.53	2.93	3.29	3.61	3.9	4.18	4.43	4.67
1200	2.64	3.04	3.4	3.73	4.03	4.3	4.56	4.8
1250	2.74	3.15	3.51	3.84	4.14	4.42	4.68	4.92
1300	2.85	3.26	3.62	3.95	4.26	4.54	4.8	5.04
1350	2.95	3.36	3.73	4.06	4.37	4.65	4.91	5.16
1400	3.04	3.46	3.83	4.17	4.47	4.76	5.03	5.28
1450	3.14	3.56	3.93	4.27	4.58	4.87	5.13	5.39

BERLINIA TIMBER SPECIE

Table F9: Berlinia Timber Specie Compression Criterion β -Values (α=0.8)

	B(width of column)							
H(depth of column)	100	150	200	250	300	350	400	450
900	4.2	5.74	6.72	7.53	8.22	8.82	9.36	9.85
950	4.64	5.92	6.91	7.73	8.33	9.04	9.58	10.07
1000	4.8	6.09	7.1	7.92	8.43	9.25	9.79	10.29
1050	4.95	6.26	7.27	8.11	8.63	9.44	10	10.5
1100	5.09	6.42	7.44	8.29	9.01	9.64	10.19	10.69
1150	5.23	6.57	7.61	8.46	9.19	9.82	10.38	10.88
1200	5.36	6.72	7.77	8.63	9.36	10	10.56	11.07
1250	5.49	6.87	7.92	8.79	9.53	10.17	10.74	11.25

Table F10:Berlinia Timber Specie Bending Criterion β -Values (α=0.8)

	B(width of column)							
H(depth of column)	100	150	200	250	300	350	400	450
900	0.57	0.8	1.54	2.27	2.99	3.7	4.4	5.07
950	0.2	0.95	1.68	2.42	3.14	3.86	4.57	5.26
1000	0.33	1.06	1.79	2.52	3.24	3.95	4.66	5.36
1050	0.42	1.14	1.86	2.57	3.29	3.99	4.7	5.4
1100	0.5	1.2	1.9	2.6	3.3	4	4.69	5.38
1150	0.56	1.24	1.93	2.61	3.29	3.97	4.64	5.32
1200	0.61	1.27	1.93	2.6	3.26	3.92	4.58	5.24
1250	0.64	1.29	1.93	2.57	3.21	3.85	4.49	5.13

Table F11:Berlinia Timber Specie Flexural Buckling Criterion β -Values (α=0.2)

	B(width of column)							
H(depth of column)	300	350	400	450	500	550	600	650
1100	2.51	2.91	3.26	3.59	3.88	4.16	4.41	4.65
1150	2.62	3.02	3.39	3.71	4.01	4.29	4.55	4.79
1200	2.73	3.14	3.5	3.83	4.13	4.41	4.67	4.92
1250	2.84	3.25	3.62	3.95	4.25	4.54	4.8	5.04
1300	2.94	3.36	3.73	4.06	4.37	4.65	4.92	5.17
1350	3.04	3.46	3.84	4.17	4.49	4.77	5.04	5.29
1400	3.14	3.56	3.94	4.29	4.59	4.88	5.15	5.4
1450	3.24	3.66	4.04	4.38	4.7	4.99	5.26	5.52

Table F12:Berlinia Timber Specie Lateral Torsional Buckling Criterion β -Values (α=0.2)

	B(width of column mm)							
H(depth of column)	300	350	400	450	500	550	600	650
1100	2.52	2.93	3.28	3.61	3.9	4.17	4.43	4.67
1150	2.64	3.05	3.4	3.73	4.03	4.3	4.56	4.8
1200	2.73	3.16	3.52	3.85	4.15	4.43	4.69	4.93
1250	2.86	3.27	3.63	3.96	4.27	4.55	4.81	5.06
1300	2.96	3.38	3.74	4.08	4.38	4.67	4.93	5.18
1350	3.06	3.48	3.85	4.19	4.49	4.78	5.05	5.3
1400	3.16	3.58	3.95	4.29	4.6	4.89	5.16	5.41
1450	3.25	3.68	4.05	4.39	4.71	5	5.27	5.53

Carrying Capacity Of Dowel

Table F13:Carrying Capacity Of Dowel For The Three Timber Species(α=0.2)

	L(length of portal frame mm)							
Q(imposed load KN)	6500	7000	7500	8000	8500	9000	9500	10000
15.5	3.62	3.44	3.26	3.1	2.95	2.8	2.67	2.54
16	3.53	3.35	3.17	3.01	2.86	2.72	2.58	2.46
16.5	3.45	3.26	3.09	2.93	2.78	2.64	2.5	2.38
17	3.36	3.18	3	2.84	2.7	2.56	2.42	2.3
17.5	3.28	3.09	2.92	2.77	2.62	2.48	2.35	2.22
18	3.2	3.02	2.85	2.69	2.52	2.4	2.27	2.15
18.5	3.12	2.94	2.77	2.62	2.47	2.33	2.2	2.07
18.75	3.09	2.91	2.74	2.58	2.43	2.29	2.16	2.04

Table F14:Carrying Capacity Of Dowel For The Three Timber Species(α=0.4)

	L(length of portal frame mm)							
Q(imposed load KN)	6500	7000	7500	8000	8500	9000	9500	10000
15.5	3.44	3.22	2.87	2.71	2.56	2.42	2.29	2.17
16	3.13	2.95	2.78	2.62	2.48	2.34	2.21	2.08
16.5	3.05	2.87	2.7	2.54	2.4	2.26	2.13	2
17	2.96	2.79	2.62	2.46	2.32	2.18	2.05	1.92
17.5	2.89	2.71	2.54	2.39	2.23	2.1	1.97	1.85
18	2.81	2.63	2.47	2.31	2.17	2.03	1.9	1.78
18.5	2.73	2.56	2.39	2.23	2.09	1.96	1.83	1.7
18.75	2.7	2.52	2.36	2.2	2.06	1.92	1.79	1.67

Table F15:Carrying Capacity Of Dowel For The Three Timber Species(α=0.6)

	L(length of portal frame mm)							
Q(imposed load kN)	6500	7000	7500	8000	8500	9000	9500	10000
15.5	2.88	2.7	2.53	2.38	2.31	2.09	1.96	1.84
16	2.79	2.61	2.45	2.29	2.15	2.01	1.88	1.76
16.5	2.71	2.53	2.37	2.21	2.07	1.93	1.8	1.68
17	2.63	2.45	2.29	2.13	1.99	1.85	1.72	1.6
17.5	2.55	2.37	2.21	2.06	1.91	1.78	1.65	1.53
18	2.47	2.3	2.14	1.98	1.84	1.7	1.57	1.45
18.5	2.4	2.23	2.06	1.91	1.77	1.63	1.5	1.38
18.75	2.37	2.19	2.03	1.88	1.73	1.6	1.47	1.35

Table F16:Carrying Capacity Of Dowel For The Three Timber Species(α=0.8)

	L(length of portal frame mm)							
Q(imposed load)	6500	7000	7500	8000	8500	9000	9500	10000
15.5	2.58	2.4	2.24	2.09	1.94	1.81	1.68	1.55
16	2.5	2.32	2.16	2	1.86	1.72	1.59	1.47
16.5	2.41	2.24	2.07	1.92	1.78	1.64	1.54	1.39
17	2.33	2.16	2	1.84	1.7	1.56	1.44	1.31
17.5	2.26	2.08	1.92	1.77	1.62	1.49	1.36	1.24
18	2.18	2.01	1.85	1.69	1.55	1.42	1.29	1.17
18.5	2.11	1.94	1.74	1.62	1.48	1.35	1.22	1.09
18.75	2.22	2.14	1.98	1.83	1.57	1.42	1.31	1.26

Shear Stress Failure

Table F17:Shear Stress Failure For The Three Timber Specie(α=0.2)

	Vm (force at joint KN)							
B(width of column)	130	135	140	145	150	155	160	161.36
50	2.75	2.65	2.56	2.47	2.39	2.7	2.24	2.21
100	5.16	4.97	4.79	4.62	4.47	4.33	4.19	4.16
150	7.55	7.27	7.01	6.77	6.55	6.34	6.14	6.09
200	9.92	9.56	9.22	8.9	8.61	8.33	8.08	8.01
250	12.23	11.79	11.38	10.99	10.63	10.3	9.98	9.88
300	14.44	13.93	13.45	13.01	12.59	12.2	11.83	11.73
350	16.48	15.92	15.39	14.9	14.43	14	13.58	13.47
400	18.28	17.69	17.13	16.61	16.11	15.63	15.19	15.08

Table F18:Shear Stress Failure For The Three Timber Specie(α=0.4)

	Vm (force at joint KN)							
B(width of column mm)	130	135	140	145	150	155	160	161.36
50	2.44	2.35	2.26	2.18	2.11	2.04	1.98	1.96
100	4.52	4.36	4.2	4.06	3.92	3.79	3.68	3.64
150	6.6	6.36	6.13	5.92	5.72	5.54	5.37	5.32
200	8.66	8.35	8.05	7.78	7.52	7.28	7.05	6.99
250	10.69	10.3	9.94	9.6	9.29	8.99	8.72	8.64
300	12.65	12.2	11.78	11.38	11.01	10.67	10.34	10.26
350	14.49	13.99	13.52	13.07	12.66	12.27	11.9	11.81
400	16.17	15.63	15.12	14.64	14.19	13.77	13.37	13.26

Table F19:Shear Stress Failure For The Three Timber Specie(α=0.6)

	Vm (force at joint)							
B(width of column mm)	130	135	140	145	150	155	160	161.36
50	2.19	2.11	2.04	1.97	1.9	1.84	1.78	1.77
100	4.04	3.89	3.75	3.62	3.5	3.39	3.28	3.11
150	5.88	5.66	5.46	5.1	5.09	4.93	4.78	4.74
200	7.7	7.42	7.16	6.91	6.68	6.47	6.27	6.21
250	9.5	9.16	8.83	8.53	8.25	7.99	7.74	7.68
300	11.26	10.85	10.47	10.12	9.79	9.48	9.19	9.11
350	12.93	12.47	12.05	11.65	11.27	10.92	10.59	10.51
400	14.48	13.99	13.52	13.09	12.68	12.29	11.93	11.84

Table F20:Shear Stress Failure For The Three Timber Specie(α=0.8)

	Vm(force at joint KN)							
B(width of column mm)	130	135	140	145	150	155	160	161.36
50	2	1.93	1.86	1.8	1.74	1.68	1.63	2.08
100	3.65	3.52	3.39	3.28	3.17	3.06	2.97	2.94
150	5.3	5.1	4.92	4.75	4.59	4.45	4.31	4.27
200	6.94	6.68	6.45	6.22	6.02	5.82	5.64	5.6
250	8.56	8.25	7.95	7.68	7.43	7.19	6.97	6.91
300	10.14	9.78	9.43	9.11	8.82	8.54	8.27	8.2
350	11.67	11.25	10.87	10.35	10.16	9.85	9.55	9.47
400	13.11	12.65	12.23	11.83	11.45	11.1	10.77	10.68