# RELIABILITY-BASED ANALYSIS AND CALIBRATION OF EUROCODE 5 DESIGN CRITERIA FOR A SOLID TIMBER PORTAL FRAME

**By**

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## Declaration

I hereby declare that the work in this dissertation titled "Reliability-based Analysis and Calibration of Eurocode 5 Design Criteria for a Solid Timber Portal Frame" was performed by me in the Department of Civil Engineering, under the supervision of Dr. Idris Abubakar, Professor Stephen Pinder Ejeh and Dr. Jaafar Abubakar Sadeeq. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this work has been presented for another degree or diploma at any university.

_____

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2014

## Certification

This dissertation titled "**Reliability-based Analysis and Calibration of Eurocode 5 Design Criteria for a Solid Timber Portal Frame**" by Jibrin Kaura MOHAMMED meets the regulations governing the award of the degree of Doctor of philosophy of Ahmadu Bello University, Zaria, and is approved for its contribution to knowledge and literary presentation.

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## Dedication

This dissertation is dedicated to the memory of my late parents, Alhaji Ummaru Usman Danbala and Hajiya Zainab Auwal Adisa, and to my late sisters, Hajiya Aishatu Mahmood Usman and Hajiya Rukayyat Ummar Danbala.

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**Abstract**

In this study, laboratory tests were conducted on some commonly used timber species (*Alstonia boonei, Triplochiton Scleroxylon, Terminalia Ivorensis, Terminalia superba and Lophira Alata*) in Nigeria. The tests were in accordance with, EN 408, EN 13153-1 and ASTM D-143). The test results were analysed in accordance with Eurocode 5. Eurocode 0, JCSS and EN 384, with the aid of Easyfit statistical package. The statistics of the reference properties (density, modulus of elasticity and bending strength) of each specie were generated, and EN 338 strength class assigned to each species after adjustment to 18% Nigerian reference moisture content. The mean values of the densities are: 360.76kg/m$^3$, 380.25kg/m$^3$, 472.10kg/m$^3$, 533.89kg/m$^3$, 955.93kg/m$^3$, for the *Alstonia boonei, Triplochiton Scleroxylon, Terminalia Ivorensis, Terminalia superba and Lophira Alata* respectively. The corresponding coefficients of variation are 4%, 16%, 18%, 19% and 4%. Likewise, the mean values of moduli of elasticity are 8192.0N/mm$^2$, 6137.80N/mm$^2$, 12161.0N/mm$^2$, 11614.0N/mm$^2$ and 22750.0N/mm$^2$, for *Alstonia boonei, Triplochiton Scleroxylon, Terminalia Ivorensis, Terminalia superba and Lophira Alata*, with the corresponding coefficients of variation of 6%, 27%, 22%, 21% and 12% respectively. Also, the mean values of bending strengths are 43.09N/mm$^2$, 54.07N/mm$^2$, 70.49N/mm$^2$, 83.63N/mm$^2$ and 97.79N/mm$^2$, for the *Alstonia boonei, Triplochiton Scleroxylon, Terminalia Ivorensis, Terminalia superba and Lophira Alata*, with the corresponding coefficients of variation of 12% 16%, 19%, 19%, and 16% respectively. Three theoretical distribution models (normal, lognormal and gumbel) were fitted to the reference material properties, using Kolmogorov Simonov test. Normal distribution was found to be the most fit for the timber density, and the best fit theoretical distribution

model for modulus of elasticity and bending strength is lognormal distribution. Based on the class limits of EN 338, strength class D18 was assigned to *Alstonia boonei, Triplochiton Scleroxylon and Terminalia Ivorensis. Terminalia superba,'* was assigned to strength class D24 and *Lophira Alata* was assigned to strength class D60. The derived material properties (tension and compression strengths parallel and perpendicular to grain, shear strength and shear modulus) were generated from the reference properties based on EN 384. The generated statistical data on the material properties and load statistics reported in various international references were used for the reliability-based analyses and calibration of the Eurocode 5 design criteria for a three-hinged solid timber portal frame. Thirty two failure modes were identified for the frame and limit state function developed for each failure mode. Component and system reliability analyses were implemented using first order reliability method and genetic algorithms. The rafter-column joint failure mode was found to be predominant. The implied safety for the predominant mode was found to correspond well with the frame implied system reliability. This is an indication that, the system reliability of timber structures is the reliability of its critical mode of failure. Uncertainty sensitive mathematical models for the prediction of material safety factors were developed. The models were recommended for use in a proposed Nigerian national annex to the Eurocode 5.

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## Notations

| | |
|---|---|
| $k_{mod}$ | Load duration factor to timber |
| $f_{d,fi}$ | Mechanical resistance of timber in fire |
| $k_{mode,fi}$ | Modification factor for fire resistance of timber |
| $f_{20}$ | 20% fractile value of mechanical resistance of timber in fire |
| $\gamma_{fi}$ | Partial safety factor for timber in fire |
| $E_d$ | Design action effect in normal temperature |
| $E_{d,fi}$ | Design action effect at elevated temperature |
| $\eta_{fi}$ | Reduction factor for the design load in fire situation |
| $G_k$ | Characteristics dead load |
| $Q_k$ | Characteristics imposed load |
| $W_k$ | Characteristics wind load |
| $\Psi_{fi}$ | Combination factor for frequent values of variable action in fire situation |
| $\gamma_g$ | Partial safety factor for dead load |
| $\gamma_q$ | Partial safety factor for imposed load |
| $\gamma_w$ | Partial safety factor for wind load |
| $D_{ef}$ | Effective charring depth of timber |
| $\beta_n$ | Charring rate |
| t | Time in minutes |
| $k_0$ | Charring rate parameter |
| GA | Genetic Algorithms |
| M | Performance function |
| G(x) | Limit state function |

| | |
|---|---|
| $X_i$ | Basic variable i |
| LSD | Limit state design |
| $P_f$ | Probability of failure |
| FORM | First Order Reliability Method |
| R | Resistance |
| S | Load action |
| $\mu_m$ | Mean of safety margin |
| $\sigma_m$ | Standard deviation of safety margin |
| ANOVA | Analysis of variance |
| $\mu_R$ | Mean of resistance |
| $\sigma_S$ | Mean of load action |
| X | Standardized variable |
| $X^*$ | Design point of basic variables |
| $\alpha^*$ | Direction cosines in the $X_i^*$ direction |
| $\beta_{HL}$ | Hasofer Lind reliability index |
| SORM | Second Order Reliability Method |
| PTM | Probabilistic Transformation Method |
| FEM | Finite Element Method |
| ANN | Artificial Neural Networks |
| $\gamma_m$ | Partial safety factor for material property |
| $\rho_b$ | Bulk density |
| $\rho_d$ | Dry density |
| $m_b$ | Bulk mass |

| | |
|---|---|
| $m_0$ | Oven dry mass |
| V | Volume of sample |
| $E_m$ | Bending modulus of elasticity. |
| L | Span of test timber beam |
| $F_2$-$F_1$ | Load increment |
| $w_2$-$w_1$ | Deflection increment |
| I | Moment of inertia |
| $f_m$ | Bending strength |
| $F_{max}$ | Maximum applied load |
| b | Width of section |
| h | Depth of section |
| W | Section modulus |
| a | Distance between one load and the nearest support in four point test setup |
| $k_h$ | Section depth adjustment factor |
| JCSS | Joint Committee on Structural Safety |
| $R_m$ | Reference bending strength |
| $E_m$ | Reference bending modulus of elasticity |
| $\rho_{den}$ | Reference density |
| $R_{t,0}$ | Derived tension strength parallel to grain |
| $R_{t,90}$ | Derived tension strength perpendicular to grain |
| $R_{c,0}$ | Derived compression strength parallel to grain |
| $R_{c,90}$ | Derived compression strength perpendicular to grain |
| $E_{t,0}$ | Derived tension modulus of elasticity parallel to grain |

| | |
|---|---|
| $E_{t,90}$ | Derived tension modulus of elasticity perpendicular to grain |
| $R_v$ | Derived shear strength |
| $G_v$ | Derived shear modulus |
| $R_{h,0}$ | Embedding strength parallel to grain |
| $R_{h,90}$ | Embedding strength perpendicular to grain |
| $f_{m,k}$ | Characteristic bending strength |
| $f_{t,0,k}$ | Characteristic tension strength parallel to grain |
| $f_{t,90,k}$ | Characteristic tension strength perpendicular to grain |
| $f_{c,0,k}$ | Characteristic compression strength parallel to grain |
| $f_{c,90,k}$ | Characteristic compression strength perpendicular to grain |
| $f_{v,k}$ | Characteristic shear strength |
| $\rho_k$ | Characteristic density |
| $E_{meam}$ | Mean modulus of elasticity |
| $E_{0,05}$ | Five percentile modulus of elasticity |
| $f_{m,12}$ | 12% moisture content adjusted bending strength |
| $E_{m,12}$ | 12% moisture content adjusted modulus of elasticity |
| $\rho_{12}$ | 12% moisture adjusted density |
| $f_{measured}$ | Measured density |
| $E_{measured}$ | Measured modulus of elasticity |
| $\rho_{measured}$ | Measured density |
| u | Moisture content at test |
| NCP | Nigerian code of practice |
| GOF | Goodness of fit |

| | |
|---|---|
| K-S | Kolmogorov Smirnov |
| ECDF | Empirical cumulative distribution function |
| pdf | Probability density function |
| $\xi$ | Reduction factor for permanent action |
| MOE | Modulus of elasticity |
| $N_{frame,k}$ | Characteristic value of axial load due to frame self weight |
| $N_{roof,k}$ | Characteristic value of axial load due to self weigh of roof |
| $N_{imposed,k}$ | Characteristic value of axial load due to imposed load |
| $N_{wind,k}$ | Characteristic value of axial load due to wind load |
| $M_{frame,k}$ | Characteristic value of bending moment due to frame self weight |
| $M_{roof,k}$ | Characteristic value of bending moment due to self weigh of roof |
| $M_{imposed,k}$ | Characteristic value of bending moment due to imposed load |
| $M_{wind,k}$ | Characteristic value of bending moment due to wind load |
| $c_p$ | Pressure coefficient |
| $q_p$ | Wind pressure |
| z | Height of building |
| $c_g(z)$ | Gust factor |
| $c_{r(z)}$ | Roughness factor |
| $h_h$ | Haunch depth |
| $X_k$ | Characteristic value of material property |
| $\eta$ | Mean value of the factor for load duration, volume and scale effect |
| $r_k$ | Characteristic value of resistance |
| $z_d$ | Vector of design variables |

| | |
|---|---|
| $\psi_i$ | Variable load i combination factor |
| $k_c$ | Buckling coefficient |
| $L_{ef}$ | Effective length |
| $K_r$ | Rotational stiffness |
| $K_u$ | Slip modulus |
| $r_i$ | Distance between single fastener and centre of connection |
| $K_{ser}$ | Joint slip |
| d | Diameter of fastener |
| $L_{ef,c}$ | Effective length of column |
| $L_{ef,r}$ | Effective length of rafter |
| $F_{v,c}$ | Load on bolt in column due to shear force |
| $F_{N,C}$ | Load on bolt in column due to normal force |
| $V_{v,c}$ | Shear force in column |
| $V_{n,c}$ | Normal force in column |
| $F_{v,r}$ | Load on bolt in rafter due to shear force |
| $F_{N,r}$ | Load on bolt in rafter due to normal force |
| $V_{v,r}$ | Shear force in rafter |
| $V_{n,r}$ | Normal force in rafter |
| $F_{d,c}$ | Total load of bolt in the axis of column |
| $F_{d,r}$ | Total load of bolt in the axis of rafter |
| $\tau_{v,c}$ | Shear stress in frame column |
| $\tau_{v,c}$ | Shear stress in frame rafter |
| $f_{h,c,k}$ | Characteristic embedding strength of bolt in column |

| | |
|---|---|
| $f_{h,r,k}$ | Characteristic embedding strength of bolt in rafter |
| $f_{v,d}$ | Characteristic shear strength in frame member. |
| $A_{net,v}$ | Net shear area in the parallel grain direction |
| $A_{net,t}$ | Net cross sectional area perpendicular to the grain |
| $F_{bs,rh}$ | Characteristic block or plug shear |
| $L_{net,t}$ | Net width of the cross section perpendicular to the grain |
| $L_{net,v}$ | Total net length of the shear fracture area. |
| $G_k$ | Characteristic Dead load |
| $Q_k$ | Characteristic Imposed load |
| $W_k$ | Characteristic wind load |
| $k_{def}$ | Creep factor |
| $\psi_2$ | Modification factor for creep due to various load |
| $\delta_L$ | Deflection limit |
| $\delta$ | Drift limit |
| $\Delta_f$ | Lateral deflection |
| $L_c$ | Column deflection |
| $\sigma_{active}$ | Standard deviation of active variable |
| m | Number of non-zero genes |
| $\Theta_s$ | Model uncertainty for load |
| $\Theta_R$ | Model uncertainty for resistance |

# CHAPTER ONE

# INTRODUCTION

## 1.1 Preamble

Many nations have either developed or are in the process of developing their own codes of practice, based on the most recent available database. Every country need to develop its design codes because of diversity in geographical and environmental conditions. In most of the Common Wealth countries, including Nigeria (Auta and Mastenikov, 2006; SNiP, 2004; Onundi, *et al,* 2009), the design or investigation of physical structures and facilities are in accordance with the requirements of the British Standard code of practice. The former British design code for timber structures, BS 5268, is based on permissible stress method. The code was withdrawn in April, 2010 and replaced with the Eurocode 5. The Eurocode 5 is based on the limit state concept. Consequently, since that date, BS 5268 was not maintained. The withdrawal of BS 5268 and the adoption of the Eurocode 5 has wide ramification to Nigeria, and other commonwealth countries. In Nigeria, the code of practice for the design of timber structures was largely based on the withdrawn permissible stress design code. It implied that the NCP 2 is now left without a basis.

The need for local contents in construction of infrastructure is a serious engineering challenge for developing countries (Aguwa and Sadiku, 2011). This is clear on the need for grading of the local timber from any country aspiring to use the new timber design approach embodied in the Eurocode. The available information from the NCP 2 cannot be used because it is not consistent with the criteria established in the Eurocode annexes. The development in the design standards is clear from the crude permissible sresses to the

advanced limit state design and partial coefficient methods. This is known as level 1 reliability method and takes into account some aspects on the randomness of actions and material properties. For steel and concrete, even more advanced fully probabilistic level 2 method have been largely brought into practice and are used to:

a) Calibrate level 1 codes to ensure a more uniform and consistent safety level.
b) Design of extraordinary structure.

For timber the development has been less impressive for several reasons.

1) The uncertainties in the material properties is much higher than for other building material. (this poses problems but also means that the advantages of introducing level 2 method is higher)
2) The properties and therefore the probability of failure, depend on the whole load and moisture history of the structure: the mechanical properties in a design situation depend on the load duration and moisture content.

All the timber species that appear in the Nigerian Code of Practice for timber structural design (NCP 2, 1973) are hardwoods. In this study, material safety factors for the limit state design format of the Eurocode 5 were calibrated using level 2 approach.. Material properties of five timber species namely: *Alstonia boonei* (Ahun), *Triplochiton scleroxylonI* (Obeche), *Terminalia ivorensis* (Black Afara), *Terminalia superba* (White Afara) *and Lophira Alata* (Ekki), that are coomonly used in Nigeria, were tested, and the results used in the calibration exercise.

## 1.2 Problem Statement and Justification of Research

NCP 2 (1973) is the design code for timber structures in Nigeria, that was developed with reference to CP 112 of Great Britain. Both the codes were based on permissible stress design approach. CP 112 was severally revised, with the major revision taking place in 1984, when it was replaced with BS 5268; also a permissible design code. BS 5268 was withdrawn and replaced with the Eurocode 5, on 31st March, 2010. The Eurocode 5, is based on the limit state design philosophy. However, the Nigerian Code of Practice for timber structural design (NCP 2) was never updated. There arouse the need for the revision of NCP 2, to meet with the current global best practices on the design of timber structures.

A country adopting the Eurocodes, is allowed to develop its National Annex, containing tropicalised design parameters, known as Nationally determined parameters. One of the most important Nationally determined parameters is the partial safety factors for material properties. The use of this factor is to accommodate uncertainties in the material properties of and ensure low probability of failure.

Among all the major construction materials (concrete, timber and steel), timber is the only one that is entirely natural. The material properties of timber, such as modulus of elasticity, bending strength, density, tension strength, compression strength, shear strength and shear modulus, are random, with varying degree of coefficients of variation. According to the Eurocode 0 (204), the current recommended safety factors in the Eurocodes, are deterministic in nature, developed based on the long term experience of

building tradition. The Eurocode 0 (2004) permit the calibration of the material safety factors using probabilistic method in order to properly and fully accommodate the inherent uncertainties in the material properties of timber. The approach to calibrate material safety factors for the proposed adoption of Eurocode 5, for Nigeria, is to use of probabilistic method. With, that factors is expected to be less deterministic.

### 1. 3 Aim and Objectives

The aim of this study was to perform reliability-based calibration of partial safety factors for material properties located in Nigeria, considering the Eurocode 5 design criteria of timber portal frame. The specific objectives include:

i. To test some Nigerian timber species in the laboratory according to EN 408 (2004) specifications.
ii. Assign EN 338 (2009) Strength classes to each of the species.
iii. To perform reliability analysis on a three hinged timber portal frame designed with five timber species, in accordance with the Eurocode 5, in order to asses the effect of uncertainties on the Eurocode 5 design criteria of timber structural systems.
iv. Assess the effect of fire and load duration on the Eurocode 5 design criteria of timber structures.
v. Conduct reliability-based calibration of partial safety factors for material properties.

## 1.4 Research Hypotheses

The hypotheses put forward in this study are as follows:

1. There is large variability in the material properties both within and between timber species.
2. The reliability-based calibration of timber portal frame structural system can be based on the reliability of its critical mode of failure.
3. The material safety factor in the Eurocode 5 is conservative with respect to Nigeria.

## 1.5 Scope and Limitation

The resistance models in the Eurocode 5 are used in conjunction with the partial safety factors for the materials properties, derived from the long term experience of building tradition to account for uncertainties. The partial safety factors for loading are giving in a separate codes (Eurocode 0, 2004). The reliability-based calibration was therefore limited to the calibration of the partial safety factors for material properties in the Eurocode 5 (2004). The probabilistic models (expected values, coefficient of variation and distribution models) used in the reliability analysis and calibration are from data generated by subjecting five Nigerian timber species to test in the laboratory in accordance with standard procedures and global best practices (EN 408, 2004; EN 384, 2004; JCSS, 2006). The tested species were graded (EN 338, 2009) as necessary criteria for their utilization in timber structures designed in accordance with the Eurocode 5. The study is limited to only five species of timber that are found in Nigeria. Even though are common timbers in the market, there are several other types.

Three-hinged solid timber portal frame, with moment resisting connection between columns and sloping rafters was used for the reliability-based analyisi and calibration of the Eurocode 5, design criteria. The frame was subjected to gravity load (self weight and live load) and wind load. Wind load data generated from fourty two meteorogical stations across Nigeria, was considered in the specification of the wind load scenario.

# CHAPTER TWO

# LITERATURE REVIEW

## 2.1 Preamble

Timber is an efficient building material, not only in regard to its mechanical properties but also because it is a highly sustainable material. Timber is considered a sustainable building material because it is derived from a renewable source and has a low embodied energy (Chanakya, 2009; Porteous and Kermani, 2007; Robert, 2009). Embodied energy reflect the minimal non-renewable energy used in the production of timber and its application in construction. Timber has sound thermal properties, meaning that timber structures rely less on carbon-emitting heating and cooling appliances than building construction of other material. Wood is also durable, since many products, particularly hardwoods have a service life of greater than 50 years, and often require little energy in maintenance (Robert, 2009). Wood can also be recycled, which is important in terms of storing carbon through the life of a product and its transformation. Wood is inexpensive material. Forest is a wood factory which produces wood using only solar energy (Dorina *et al*, 2012).

Timber is a widely available natural resource in many countries; with proper management, there is a potential for a continuous and sustainable supply of raw timber material in the future. 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

(Porteous and Kermani, 2007). The concept of environmental performance becomes clearer when wood is compared with steel and concrete.

Recent research studies confirmed that houses made from wood present significantly lower risk for the environment. Wood-based buiding materials require less energy to be produced, emit less pollution to the air and water, contribute with lower amount of $CO_2$ to the atmosphere, are easily disposed of or recycled, and are derived from a renewable resource. It can be easily concluded that the environmental advantages of wood compared to steel and concrete are obvious. Life-cycle analysis results for the steel-framed versus wood-framed home (Dorina *et al*, 2012) showed that the steel-framed home use 17% more energy; had 26% more global warming potential; had 14% more air emissions; had over 300% more water emission and had about the same level of solid waste production than timber. Analysis results for the concrete-framed home used 165 more energy; had 31% more global warming potential; had 23% more air emission; had roughly the same level of water emission and produced 51% more solid waste.

Timber, is an advantageous building material due to its material properties. 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 (Porteous and Kermani, 2007). However, timber is still not utilized to its full potential in the building and construction sector considering its beneficial properties. Many building owners, 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. In addition to this there are still a number of issues which need to be further researched before timber materials can achieve the same recognition as a high quality building material such as e.g. steel and concrete.

Timber is an abundant, renewable and recyclable material, which has been used by humans for thousands of years. Its use in construction is still widespread, ranging from structural frames to floors, paneling, doors, interior and exterior woodwork, and furniture, among its multiple uses in an average dwelling (FPL 1999). Three polymeric materials make up the wood cells: cellulose, hemi-cellulose and lignin (Kollmann and Cote, 1968). Cellulose makes up the cell walls, and provides the tensile strength of the wood matrix. Hemicellulose is similar to cellulose, and grows around the cellulose fibres. Lignin gives rigidity to the wood, allowing trees to grow upright; it cements the cells together, thus accounting for the compressive and shear strengths of wood (Kollmann and Cote 1968; Moraes 2003). These three polymers form an inhomogeneous and anisotropic material, which exhibits great variability among different species.

In general tree species can be divided into two major groups, hardwoods and softwoods. Hardwoods are porous, and present greater hardness than softwoods (although some exceptions exist) (FPL, 1999). Timber structures have traditionally been built using heavy timber frames, with the walls being constructed of various materials such as interwoven branches and split logs in the very early versions of these types of structures (as early as 6500 BC), and later using plastered panels and bricks (Foliente, 2000). Other forms of timber structures are palisade-type

buildings and log-cabin constructions. Roof structures, as integral part of the building, have also traditionally been made in wood, and are still popular to this day especially in Nigeria. Timber roof structures have two basic forms: simple horizontal beams, or more complicated rafters being supported by the frame. Combinations of these two types have developed into hammer-beam roofs, which were used with great skill during the Renaissance, and into trusses, where originally the struts were also under flexion, unlike modern trusses where all the joints are assumed to be pinned (Foliente, 2000).

The coming of the industrial revolution marked the appearance of industrially built planks, boards and nails which spawned the appearance of new construction techniques, especially in the USA, where new framing methods reduced labour costs, increased the flexibility in construction and allowed for prefabrication (Foliente, 2000). These light frame construction methods are nowadays the predominant form of construction in residential and low-rise buildings. Other technological advances developed in this period are the glued laminated timber, first used in Bavaria in 1809 (Pedro, 2008) and improvements in wood trusses (Foliente, 2000). Glued laminated timber in its present form was developed in the twentieth century, as well as another important innovation, plywood (Foliente, 2000).

## 2.2 Basic Material Properties of Timber

### 2.2.1 Background

A base set of material properties as well as some influencing factors are needed for a probabilistic design (Toratti and Ranta Maunus, 2002). Density, modulus of elasticity and bending strength are the reference material properties required to be determined explicitly from laboratory (Yang *et al*, 2008; Ziwa *et al*, 2006). The propertries and distribution of other material properties are linked with EN 384 (2004) and JCSS (2006)

recommendations. As for the influencing factors, such as moisture content effect, load duration effect, size and system effect, use is made of the deterministic value of such properties.

In BS 5268 it is the strength that governed the design, but in the EC5 modulus of elasticity (MOE) and density are also important for both ultimate and serviceability limit states. The knowledge of MOE is of great importance for designing timber structures. It is normally the most important parameter for strength grading of sawn timber (Glos, 1995). The end-use of wood material, especially for structural timber is strongly related to MOE (Piter and Zerbino, 2003; Yang *et al*, 2008; Cave and Walker, 1994; Cown *et al*, 1999; Kennedy, 1995; Kliger *et al*, 1995; Kumar, 2004).

Modulus of elasticity defines the relationship between stress and strain within the elastic region. Timber is considered to be an isotropic material with three mutually perpendicular axes in the longitudinal, radial and tangential directions (Piter and Zerbino, 2003). The longitudinal modulus of elasticity is a quantitative measure of the stiffness of the wood along the grain. It is the most commonly measured elastic property and the most important elastic constant. Elasticity implies that deformations produced by the low stress are completely recoverable after loads are removed, and this is the accepted criterion of stiffness.

The values of MOE is of great importance for estimating deformation in timber structures, and for calculating timber columns (Thelanderson, 1995). The modulus of

elasticity and bending strength are determined in the laboratory by testing timber beams in bending. In the bending test of structural members, two forms of loading were employed, the choice depending upon the type specified in codes of practice for the particular material. In the first, a simply supported beam is subjected to a four-point loading system. Two concentrated loads are symmetrically applied to the test piece (beam) producing zero shear force and constant bending moment in the central span of the beam. The condition of pure bending is therefore achieved in the central span. The second form of loading system consists of a single concentrated load at mid-span. The loads are usually applied by a testing machine. Deflection is measured by a dial gauge placed underneath the beam, and load-deflection diagrams were plotted. BS 373 (1957) and ASTM D 143 (2006) set out method for measurement of MOE of small clear timber specimen based on the three point bending.

The European Standard EN 408 (2004) establishes a third point loading test for calculating the local MOE, free of shear (clause 9), in the central third, but the code additionally allows for obtaining the global MOE, over the whole span (clause 10). The mean values of MOE, and the characteristic values of bending strength and density allow to assign a timber specie to a strength class of the International Strength class System EN 338 (2008), and the other characteristic strength and stiffness properties can be calculated from those basic ones (Glos, 1995; Baltrusaitis and Pranckevocene, 2003; Antti *et al*., 2006; JCSS, 2006; Ridley-Ellis *et al*, 2008).

Wood density is often considered to be another very important wood property because of its strong influence on the quality of a wide range of solid wood and fiber products and has as such, received much attention. Bunn (1981) states that "density is probably the most important intrinsic wood property for most wood products", while Bamber and Burley (1983) pointed out that, "of all the wood properties, density is the most significant in determining end use". Similarly, Zobel and van Buijtenen (1989) states that "density largely determines the value and utility of wood and overshadows the importance of other wood properties". However, the importance of density as an indicator of wood quality alone is debatable because variability is due to many intrinsic factors, defects and variations inherent in basic wood structures.

### 2.2.2 Variability of stiffness and strength in structural timber

Most timber codes that were used throughout the world until the 1980s were in an allowable stress format. The permissible stresses for these codes were based on extensive research into the properties of clear wood samples tested in the first half of this century. Clear wood samples are small pieces of wood that are free from any sign of obvious strength-reducing characteristics. The absence of strength-reducing characteristics means that the properties are essentially those of the wood fibres aligned in the most favourable orientation. In general, the clear wood samples provide an indication of the upper limit of the expected performance of realistic timber pieces. (Ritter *et al*, 1996; Madsen, 1992). The commercial timber, however, does not have the same properties as clear wood at all. Today the strength capacity is determined from direct testing of structural timber, so called ingrade testing.

The variability between individual timber elements is significantly larger than those found for steel or reinforced concrete members. The coefficient of variation (COV) for strength properties of structural timber is 20 – 40 % where the higher value is found for brittle failure modes. The variability may also differ between wood species. The lengthwise variation of the modulus of elasticity has been studied since the mid-sixties. The within variability of bending stiffness was modelled by Czmoch (1991,1998) as a stationary random process. He used two models, one with random bending stiffness variation around a global mean for the whole population of beams. In the other model a variable mean was used for every beam and a stationary random process expressed the local fluctuation within a beam.

The variability of tensile strength parallel to the grain has been studied for example by Showalter *et al.* (1987) and Lam and Varoglu (1991a,b) with both experiments and mathematical models. Riberholt and Madsen (1979) presented a model of the lengthwise bending strength. This model is based on a constant bending strength between defects. The clear wood strength and the strength in weak zones are described by stochastic variables. A Poisson process describes the occurrence of a weak section. The model was further used by Czmoch (1991,1998), Thelandersson *et al*, (1999) in a study of load configuration and length effects. They suggested further investigations to get the input data to the model. Källsner and Ditlevsen (1994) presented a study on the variability of bending strength. Weak sections were cut out from 26 beams and were finger jointed to pieces of wood with expected higher strength. In total, 197 tests were carried out.

Unfortunately, only 34 percent of the failures were bending failure in the weak zone (most of the failures occurred in the finger joints).

A statistical model was developed to model the bending strength variation. The model was later improved to also include the failure of the finger joint. Furthermore the experimental verification was done in three phases. The first phase contained experiments with single weak zones. Phase two was performed for long beams containing several weak zones. The third phase studied the interaction between weak zones. The verification indicates that the bending strength was lower than the predicted strength from the model. The reasons for this may be statistical uncertainty, release of stored elastic energy during the test and possible interaction between weak zones, Källsner et al, 1997; Källsner and Ditlevsen, 1998, 1999).

### 2.2.3 Background of the Tested Timber Species

All the timber species listed in the NCP 2 (1973) are hardwoods. Generally, hardwoods are used in structures where both sound mechanical performance and good natural durability are required (Blass, 2005). Hardwoods offer certain advantages, particularly when using those timbers which are in the medium to high density ranges. Such material may allow the designer to use timber throughout a structures, rather than having to switch to other materials for longer spans or heavier buildings. Another well established building application for hardwoods is in the form of solid timber portal frames. The availability of larger dimensions, and higher grade, as well as the inherently greater strength and stiffness provided by many hardwoods makes it possible to consider hardwoods for

application where steel or concrete would otherwise be regarded as necessary. The additional density of many of the species which are considered suitable for structural purposes also provides additional fire resistance. This is acknowledged by the indication of slower charring rates in the structural timber design codes, BS 5268: part 4 and Eurocode 5: part 1-2. Five timber species were investigated in this study namely *Alstonia boonei* (Ahun), *Triplochiton scleroxylon* (Obeche), *Terminalia Ivorensis* (Black Afara), *Terminalia superba* (White Afara) and *Lophira Alata* (Ekki).

*Alstonia Boonei* (Ahun) is a large deciduous tree, up to 45m tall and 1.2m in diameter. In Nigeria, the tree occurs in moist lowland forest of the southern axis. They are also found in Angola, Benin, Cameroon, Central African Republic, Congo, Cote d'voire, democratic republic of Congo, Equatorial Guinee, Guinea-Bissau, Liberia, Senegal, Sierra-Leone, Sudan, Togo and Uganda (Cirad, 2009). The sapwood of alstonia boonei is very wide, up to 200mm soft and light in weight when dried. Nearly yellowish-white when freshly cut. It has a low luster and no characteristic odour or taste. The grain is generally straight, and the wood liable to staining. It work easily with hand and machine tools. Also, the wood can be glued easily.

*Triplochiton Scleroxylon* (Obeche), is a large deciduous forest tree commonly attaining 45m in height and 1.5m in diameter (Martins, 1984). The specie is geographically distributed in Benin, Cameroon, Central Africa Republic, Congo, Cote d'voire, Democratic Republic of Congo, Equatorial Guinea, Ghana, Guinea, Liberia, Sierra-Leone, Togo and Nigeria (Cirad, 2009). Although, the main uses of the specie are for

timber, it is also used for fibreboard and particleboard. The heartwood and sapwood are not clearly differentiated. There is usually an unpleasant odour when the specie is freshly cut, but the smell disappears after it is seasoned. The sapwood is permeable. It respond well to hand and machine tools in moulding and most operations.

The specie has good gluing qualities. It is light yellow in colour. It appeared in EN 350-2 (1994). It is a class 5 (non-durable) material, susceptible to termite attack, thereby requiring preservative treatment. It is produce poor connection with nails but good with glue. Based on the work at Cirad (2009), at 12% moisture content, the specie has a density of 380kg/$m^3$, crushing strength of 30MPa, static bending strength of 52MPa and modulus of elasticity of 7260MPa.

*Terminalia Ivorensis* (Black Afara) is a large deciduous forest tree ranging in height from 15 to 46m. It is branchless for up to 30m (Martins, 1984). The tree is native to Cameroon, Cote D'voire, Ghana, Guinea Bissau, Liberia, Sierra Leone and Nigeria (Cirad, 2009). The specie has a yellowish-brown wood. It is listed in EN 350-2 (1994). It is a class 3 (moderately durable) material. It is suaceptible to termite attack. It has good nailing and gluing capacity. Based on the work pf Cirad (2009), at 12% moisture content, the density of Terminalia Ivorensis is 500kg/$m^3$, crushing strength of 44MPa, static bending strength of 71MPa and modulus of elasticity of 11350MPa.

*Terminalia superba* (White Afara) is a large tree, up to 50m tall and 5m in girth. It is essentially a tree of deciduous forest (Martin, 1984). It is a native of Congo, Cameroon,

Angola, Benin, Central Africa republic, Cote d'voire, Democratic Republic of Congo, Equatorial Guinea, Gabon, Ghana, guinea, Liberia, Sierra Leone, and Nigeria (Cirad, 2009). It is light yellow in colour, and appeared in the EN 350-2 (1994). It is a class 4 (poorly durable) material, and susceptible to termite attack. It is good with both nail and glue. Based on the work at Cirad (2009), at 12% moisture content, the specie has a density of 540kg/$m^3$. Crushing strength of 47MPa, static bending strength of 80MPa. The modulus of elasticity is 11750MPa.

The specie is used as timber and fibre. It has potential importance in paper making, offering the capability of providing a relatively wide range of pulps. It is also used for the manufacture of cellulose, for paneling and also as particleboard. In the area of timber manufacturing, the wood is normally creamy white, with no noticeable distinction between the sapwood and heartwood.

*Lophira Alata* (Ekki) occurs in evergreen and moist decidous forest in freshwater swamp forest (NCP 2, 1973). The tree may attain a height of 50m. Trunk diameter range from 1.5 to 2.5m, without butress (Martin, 1884). *Lophira Alata* (Ekki) is dark red in colour. The specie appeared in the EN 350-2 (1994). It is a class 2 (durable), highly resistant to termite attack. It is non-permeable to preservative treatment. It has a good embedment to nails, but pre-boring is required when driving. Based on the study conducted at Cirad (2009), the species has a density of 1006kg/$m^3$, crushing strength of 96MPa, static bending strength of 162MPa and Modulus of elasticity of 21420MPa; all at 12% moisture

content. The tree is used for heavy duty contruction work, such as harbour works, heavy-duty flooring, rail road cross ties.

### 2.3 Strength Classes of Timber

Solid timber for structural application has to be strength graded prior to its use (Kisito *et al,* 2011). In order to remain economic, the grading process usually focuses on the most important physical and mechanical properties; density, modulus of elasticity (MOE) and bending strength (JCSS, 2006). Timber is assigned to its appropriate strength class based on respective limits given in EN 338 (Kisito *et al*, 2011; Steiger and Arnold, 2009; Herald *et al,* 2007; Baltrušaitis and Pranckevičienė, 2003). Additional mechanical properties such as tensile and compressive strength parallel and perpendicular to the grain are derived from the basic property values by standard empirical relationships (EN 384, 2004; JCSS, 2006).

#### 2.3.1 The Timber Strength Classificationn System

Prior to April, 2010 the structural design of timber members in the United Kingdom is related to Part 2 of BS 5268, and is based on permissible stress design philosophy. Elastic theory is used to analyse structures under various loading conditions to give the worst design case. Then timber sections are chosen so that the permissible stresses are not exceeded at any point on the structure. BS 5268 par 2, on which the design of structural timber is based, was originally published as CP 112 in 1952, revised later in 1967 and, extensively revised in 1971.

The 'basic stresses' introduced in CP 112 were determined from carrying out short-term loading tests on small timber specimens free from all defects. The data was used to estimate the minimum strength which was taken as the value below which not more than 1% of the test results fell. These strengths were multiplied by a reduction factor to give basic stresses. The reduction factor made an allowance for the reduction in strength due to duration of loading, size of specimen and other effects normally associated with a safety factor, such as accidental overloading, simplifying assumptions made during design and design inaccuracy, together with poor workmanship.

Basic stress was defined as the stress that could be permanently sustained by timber free from any strength-reducing characteristics. Since 1967 there have been continuing and significant changes affecting the structural use of timber. Research studies in the UK and other countries had shown the need for a review of the stress values and modification factors given in the original code. With the introduction of BS 5268 in 1984, the concept of' basic stresses' was largely abandoned and the new approach for assessing the strength of timber moved somewhat in line with iimit states' design philosophy.

In 1996, Part 2 of BS 5268 was revised with a clear aim to bring this code as close as possible to, and to run in parallel with, Eurocode 5: Design of timber structures, Part 1.1 General rules and rules for buildings. The overall aim has been to incorporate material specifications and design approaches from Eurocode 5, while maintaining a permissible stress code with which designers, accustomed to BS 5268, will feel familiar and be able to use without difficulty.

The concept of grouping timber into strength classes was introduced into the UK with BS 5268: Part 2 in 1984. Strength classes offer a number of advantages both to the designer and the supplier of timber. The designer can undertake his design without the need to check on the availability and price of a large number of species and grades which he might use. Suppliers can supply any of the species/grade combinations that meet the strength class in a specification. The concept also allows new species to be introduced onto the market without affecting existing specifications for timber. The latest strength classes used in BS 5268: Part 2: 1996 relate to the European strength classes which are defined in EN 338 Structural timber. Strength classes.

There are a total of 16 strength classes, C14 to C40 for softwoods and D30 to D70 for hardwoods as given in Table 7 of BS 5268: Part 2 (1996). The number in each strength class refers to its 'characteristic bending strength' value, for example, C40 timber has a characteristic bending strength of $40N/mm^2$. Characteristic values larger than the grade stress values used in BS 5268: Part 2, as they do not include effects of long-term loading and safety factors

One benefits of using characteristic value of material properties rather than grade stress is that it will make it easier to sanction the use of new material properties and component for structural purposes, since such values can be utilized immediately, without first having to determine what reduction factors are needed to convert them to permissible or working values. In Table 2.1, typical grade stresses and characteristic values for timber strength class D70 are presented.

Table 2.1: Strength Properties of D70 Timber Based on BS 5268 and EN 338

| Material Property | Strength class D70 | |
|---|---|---|
| | Grade Stress (BS 5268) | EN 338 |
| Bending Sterngth Parallel to Grain | 23.0 | 70.0 |
| Tension Parallel to Grain | 13.8 | 42.0 |
| Compression Parallel to Grain | 23.0 | 34.0 |
| Shear Strength | 2.60 | 6.0 |

Source: BS 5268 (1996); EN 338 (2009)

### 2.3.2 Timber Grading System in the Nigerian Code of Practice (NCP 2)

NCP 2 (1973) is the structural timber design code in Nigeria. This code is based on the permissible stress design philosophy, with reference to the CP 112 (1967). With the introduction of BS 5268 CP 112 had been withdrawn in 1984. To date NCP 2 was never subjected to any revision, and remained the authoritative document to timber structural design in Nigeria. The basic stresses in the NCP 2, is similar to that in CP 112. The timber species are classified into 7 strength groups N1 to N7, with N1 given the strongest species. Each specie is also classified according to its ability to resist attack by fungi and insects.

The code also has five durability classes namely, very durable, durable, moderately durable, non-durable and perishable. As the ease with which the timber can be impregnated when it is used under condition which favour decay or attack by insects is important, five classes indicating the resistance of the heartwood to impregnation were

given in the code. These are; permeable, moderately resistant, resistant, very resistant and extremely resistant. Table 2.2 give the NCP 2, strength grades of the timber species tested in this study.

Table 2.2 Grades of the Tested Timber Species According to NCP 2 (1973)

| Timber Specie | Standard Name | Strength Group | Natural Durability | Resistant to Impregnation |
|---|---|---|---|---|
| *Alstonia boonei* (Ahun) | Alstonia | N7 | Perishable | Permeable |
| *Triplochiton Scleroxylon* (Obeche) | Obeche | N6 | Non-durable | Moderately Permeablr |
| *Terminalia Ivorensis* (Black Afara) | Idigbo | N4 | Durable | Extremely Permeable |
| *Terminalia superba* (White Afara) | Afara | N6 | Non-durable | Permeable |
| *Lophira Alata* (Ekki) | Ekki | N1 | Very Durable | Extemely permeable |

Source NCP 2 (1973)

### 2.4 Effect Load Duration on Timber Structures

Wood experience a significant loss of strength and stiffness when loaded over a period of time. The effect of ten years load duration may amount to a strength loss of approximately 40 % for solid wood (Toratti, 2004; Kohler and Svensson, 2002; Larsen, 2001). This phenomenon is known as creep-rupture – often called the duration of load (DOL) effect. The issue has been a subject of particular interest for everyone in the timber engineering community concerned with safe and efficient engineering design. Research on both clear wood (Wood, 1951) and structural timber (Hoffmeyer, 1990 and Madsen 1992) sustaining long-term constant load is well documented in the literature

### 2.4.1 The Eurocode 5 Load Duration Factor $K_{mod}$

Load duration factors to account for timber design under sustained load are given in Eurocide. The $k_{mod}$ is based on timber service and load duration classes. The service classes are defined in clause 2.3.1.3 of the Eurocode 5 as follows;

Service class 1 refers to timber used internally in a continuously heated building. The average moisture content likely to be attained in service condition is 12%. Service class 2 refers to timber used in a covered building. The average moisture content likely to be attained in service condition if building is generally heated is 15%, and if unheated, 18%. Service class 3 refers to timber used externally and fully exposed. The average moisture content likely to be attained in service condition is over 20%.

Table 2.3: Eurocode 5 Load Duration Classes

| Load duration class | Order of accumulated Duration of Characteristic Load | Example of loading |
|---|---|---|
| Permanent | Greater than 10 years | Self weight |
| Long-term | 6 month to 10 years | Storage |
| Medium-term | 1 week to 6 month | Imposed floor load |
| Short-term | Less than one week | Snow, maintenance |
| Instantaneous | Instantly | Wind, impact |

Source: Eurocode 5: Part 1.1 (2004)

The load duration classes are given in Table 1 of the National Anex to Eurocode 5, as presented in Table 2.3, and based on the service and load duration class, the $K_{mod}$ are established, as shown in Table 2.4.

Table 2.4: $K_{mod}$ for Solid Timber (Eurocode 5)

| Load Duration Class | Service Class | | |
|---|---|---|---|
| | Class 1 | Class 2 | Class 3 |
| Permanent | 0.60 | 0.60 | 0.50 |
| Long term | 0.70 | 0.70 | 0.55 |
| Medium term | 0.80 | 0.80 | 0.65 |
| Short term | 0.90 | 0.90 | 0.70 |
| Instantaneous | 1.10 | 1.10 | 0.90 |

Source: Eurocode 5: Part 1.1 (2004)

## 2.5 Timber Exposure to Fire

One of the most prevalent questions that arise with regards to timber buildings worldwide is the question of fire safety (James, 2009). A timber structure can be designed to be as economical, as structurally sound and as aesthetically pleasing as concrete and steel counterparts, but without addressing the issue of fire safety these facts hold little weight when a decision is made on what material to build with. The general perception of the public is that timber is combustible, therefore it is considered to be more dangerous to use than steel or concrete (Lina *et al*, 2010).

Fire is an emotive subject and is one of the first issues raised when timber frame construction is discussed. Comparative fire tests, showed that, when concrete has perished and steel melted in fire, timber can still take a large load (Ideris *et al*, 2008). Fig. 2.1 shows a typical timber frame after intense fire. Timber has a high and predictable performance in fire because timber char at a slow and known rate. In one important aspect of performance, namely the maintenance of strength with increasing temperature over time, wood performs well. This is due to the fact that timber chars at a constant rate throughout a fire; the formation of this char protects the un-burnt timber underneath. BS 5268 Part 4 gives the charring rates for softwioods and hardwoods and a method of calculating the fire resistance of timber components. Eurocode 1-2 also gives charring rates for softwood and hardwoods and a method of calculating the fire resistance of timber components that is not too dissimilar to that of BS 5268.



Fig. 2.1 Wooden frame after intense fire (Source: Larsen and Jorgen, 2011)

Great amounts of research have been destined on the performance of timber in fire, since the late years of the nineteenth century (Babrauskas and Williamson 1978; Ingberg and Mitchell 1941, Bamford, et al. 1946, Drysdale 1998, Buchanan 2000, White 2002). Good reviews covering the performance of timber structures in fires and their design are those by Schaffer (Schaffer 1984), Buchanan (Buchanan 2000), Benichou and Sultan (Benichou and Sultan 2000) and Poon (Poon 2003), while Lau and Barrett made assessment of the design process (Lau and Barrett 1994).

Structural failure of timber elements can occur due to two mechanisms. Mechanical resistance can decrease due to changes in wood properties caused by the temperature rise (Kollmann and Cote, 1968; Gerhards, 1982; Schaffer, 1984; Moraes, 2003). In particular, metallic connectors can fail at low temperatures because of a decrease in the embedding strength of wood (Moraes, *et al*.2005). At higher temperatures and longer times, timber structures can fail because of the reduction of the cross section of the timber elements due to charring (Eurocode 5, 2004).

Timber is weakened by temperature and moisture content increments (Kollmann and Cote 1968). This can cause structural failure even at temperatures below the charring temperature. Gerhards (1982) provides an excellent review of the particular changes of properties such as the modulus of elasticity, shear modulus, and tensile, compressive and shear strengths with variations in temperature and moisture content.

In general, wood is weakened by the increase of moisture and temperature, with the modulus of elasticity parallel to the grain decreasing to 80% of its value at equilibrium moisture conditions when it reaches the fibre saturation point (30% moisture content), and falling to 70% of its value when it reaches $200^0$C. The decrease in strength can reach values of more than 30% for shear and compressive strengths at temperatures of only $50^0$C (Gerhards, 1982), with respect to the values at $20^0$C. Osman (1985) studied the wood tensile strength in high temperatures and increasing moisture contents, reporting a drop close to 40% in the tensile strength at a temperature of $200^0$C, and a drop of 30% in the modulus of elasticity when the moisture content reaches a value of 30% (for wood parallel to grain). Moraes (2003) measured a decrease of about 20% in the embedding strength at similar temperature rise.

Embedding strength is a critical property in connector performance, since resistance to shear stresses is directly related to the embedding strength (Moraes *et al*., 2005). Moraes detected an initial drop\ in the embedding strength to about 70% when temperature reaches $80^0$C, both for loading parallel and perpendicular to grain. After $100^0$C, the embedding strength increases, reaching a relative maximum at $140^0$C, which is 10% lower than the initial value at ambient temperature. The strength then decreases, to reach a value close to 60% of the initial strength for a temperature of $200^0$C. This conduct is explained by the behaviour of the lignin matrix. Lignin will attain glass transition between $60^0$C and $80^0$C (Schaffer, 1973; Irvine, 1984; Moraes 2003) for moisture contents between 3 and 10%, which leads to a loss of binding strength between the fibres and to a consequent loss of embedding and compressive strengths of the timber element.

For dry conditions, the glass transition temperature increases, and Moraes states that the increase in the embedding strength at the temperatures mentioned above occurs because at those temperatures the samples are practically oven dry (Moraes et al. 2005).

Earlier, Noren (1996) reported that although there was no data on the evolution of embedding strength with temperature available at the time, it was expected that it would follow a similar behaviour to the compressive strength, because both are related. However, the dominant mechanical property in the behaviour of timber structures in fires is the compressive strength (Young and Clancy 2001).

Young and Clancy (2001) report reductions on the compressive strength of dry samples loaded parallel to grain of about 20% at a temperature of $200^0$C. They stated that the magnitude of the change with temperature of compression properties differs to that of tensile properties, which can thus be related to the lignin matrix. Finally, these researchers also expect an increase of the compressive strength between $160^0$C and $210^0$C. Since fires will increase both the temperature and moisture content, it is certain that these strength reductions will be faced by every timber element exposed to a fire. White and Schaffer (1981) have measured peak moisture contents of about 20% in wood samples heated following standard furnace testing curves (ISO 1999), and observed that the front of peak moisture content consistently moves ahead of the charring front.

The peak moisture content occurs at $200^0$C, and is 1.3 to 2.0 times the initial moisture content. Fredlund (1988), using a pressure model, calculated slightly higher values for the

moisture content, of up to 25%, in wood exposed to a fixed incident heat flux. The measured peak pressure rise is of the order of 100kPa (Fredlund 1988). As the temperature continues to rise, devolatilization of the wood polymers will commence. The loss of tensile strength in wood is attributed to the depolymerisation of cellulose, which begins at around $200^0$C, while hemicellulose suffers a rapid mass loss at around $180^0$C that is partly responsible for changes in compressive strength (Schaffer, 1973; Young and Clancy, 2001). Lignin, after a series of physical changes, will begin to lose mass at about $280^0$C. The products of this chemical transformation are pyrolysis gases which are mainly released to the exterior, and a char layer, which has insulating properties but also has a greater porosity.

The pressure rise generated by the release of pyrolysis gases is lower than that associated with the vaporization of water (Fredlund, 1988), and is of the order of 20 kPa (Tinney, 1965, Lee *et al.*, 1976). Char is more brittle than virgin wood, and due to shrinking or pressure build up, cracks will appear on its surface. Subsequent heating will increase the thickness of the char layer and eventually combustion, either flaming or surface combustion, will take place. It is the char layer that causes the most known mode of failure of timber structures under fire. As heating of the timber element continues for an extended period of time, the pyrolysis front will move into the virgin sections of wood located at deeper positions. The brittle layer of char is not able to support any loads, causing an increase in the stress of the reduced cross section of virgin wood. In addition, the virgin wood will heat up, which will cause a decrease in the strength as has already been described.

A series of methodologies have been developed to account for the speed at which the char layer progresses into the virgin wood and the loss of strength of the timber member (Eurocode 5, 2004; Schaffer, 1984; White, 2002; Poon, 2003). They consider a reduction in the cross section caused by charring, and normally the charring rate is taken as a constant. Thus, knowing the time of exposure of the timber element to a fire allows calculating the remaining cross section which is still load bearing. These methods also consider the loss of strength of the virgin wood due to the temperature rise.

### 2.5.1 BS 5268: Part 4-1978 Fire Design Requirements

Section 4.1 of BS 5268: Part 4-1978 sets out a method of assessing the performance of a structural timber exposed to fire. The charring rate of wood was taken as 0.67mm/min. For columns exposed on all sides and members in tension, the charring rates were increased by a factor of 1.25.

### 2.5.2 Fire Resistance Assessment According to Eurocode 5

Alternatively, three methods can be used for design and assessment of fire-exposed timber structure reliability according to the Eurocodes 5 part 1-2 (2004). They are reduced cross section method, reduced properties method, and common method. These methods are ordered upward according to calculation demands and subsequently even according to the resultant fire resistance of the structure. The principle of these methods consists in cross section charring depth calculation, or the change of physical and mechanical properties as well as the assessment of residual cross-section load-bearing capacity in the required fire resistance time.

Charring depth results from the design charring speed of the respective material and fire resistance. Design charring rate (range between 0.5 to 1mm/min) depends mainly on wood density or on density of similarly-based material (usually inversely proportional), and it is mentioned by means of a tabular characteristics values in Eurocode 5 par1-2 (2004). The design of structural element in fire conditions according to Eurocode 5, part 1-2 (2004) is given by the following procedures:

Mechanical resistance: $f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma}$ (2.1)

where $f_{20}$ is the 20% fractile value of the mechanical resistance distribution in normal temperature condition. $\gamma_{fi} = 1.0$ is the partial safety factor for timber in fire and $k_{mod,fi}$ is the modification factor for fire. The action is obtained from a simplified procedure as

$$E_{d,fi} = \eta_{fi} E_d \quad (2.2)$$

Where, $E_d$ is the design effect of action in normal temperature design and $\eta_{fi}$ is the reduction factor for the design load in fire situation. This is given by the following equation:

$$\eta_{fi} = \frac{(G_k + \psi_{fi} Q_{k,i})}{(\gamma_g G_k + \gamma_{q,i} Q_{k,i})} \quad (2.3)$$

Where $\gamma_g$ ang $\gamma_q$ are partial safety factors and $\psi_{fi}$ is the combination factor for frequent values of variable action in the fire situation. These depends on fire exposure. They are established using the charring rate equation, given by:

$$D_{ef} = \beta_n t + K_n d_n \quad (2.4)$$

Where $\beta_n = 0.7$ mm/min, $K_n = 1$ and $d_n = 7$mm.

## 2.6 Genetic Algorithms (GA)

Genetic Algorithms (GA) are adaptive heuristic search algorithm based on the evolutionary ideas of natural selection and genetics. They represent an intelligent exploitation of a random search used to solve optimization problems (Sivaraj and Ravichandran, 2011). The basic techniques of the Gas are designed to simulate processes in natural systems necessary for evolution; especially those that follow the Charles Darwin's principles of "Survival of the Fittest".

This principle is similar to competition among individuals for scanty resources in nature resulting in the fittest individuals dominating over the weaker ones. GA simulates the survival of the fittest individuals among all the individuals in the population over successive generations for solving a problem. Each generation consists of a population of character strings that are analogous to the chromosome that are seen in DNA (Deoxyribonucleic acid). Each individual represents a point in a search space and a possible solution. The individuals in the population are then made to go through a process of evolution.

GA is based on an analogy with the genetic structure and behavior of chromosomes within a population of individuals using the following foundations: Individuals in a population compete for resources and mates. Those individuals most successful in each 'competition' will produce more offspring than those individuals that perform poorly. Genes from `good' individuals propagate throughout the population so that two good

parents will sometimes produce offspring that are better than either parent. Thus each successive generation will become more suited to their environment (Goldberg, 1989).

### 2.6.1 Selection Operator

The key idea of selection operator is to give preference to better individuals (those that are nearer to the solution) by allowing them to pass on their genes to the next generation and prohibit the entrance of worst fit individuals into the next generations. The selection operator mainly works at the level of chromosomes. The goodness of each individual depends on its fitness. Fitness value may be determined by an objective function or by a subjective judgement specific to the problem. As the generations pass, the members of the population should get fitter and fitter.

### 2.6.2 Selection mechanisms

A critical parameter to be improved in GA is the selection pressure which is the process of selecting the best individuals for the next generation. If it is set too low, then the rate of convergence towards the optimum solution is too low. If the selection pressure is set too high, the system is likely to be stuck in a local optimum due to the loss of diversity in the population. Thus the selection methods controls the selection pressure, which in turn determines how fast the algorithms coverage. The speed of convergence of different selection schemes was first studied by Goldberg and Deb (1990). The selection mechanism should be chosen such that it converges to the global optimum solution by without being caught into local optimum and also should encompass the knowledge of the existing data.

### 2.6.3 Roulette Wheel Selection

In the Roulette wheel selection method (Holland, 1992), the first step is to calculate the cumulative fitness of the whole population through the sum of the fitness of all individuals. After that, the probability of selection `is calculated for each individual as being $p_{sel,i} = f_i/Pf_i$. Then, an array is built containing cumulative probabilities of the individuals. So, n random numbers are generated in the range 0 to $Pf_i$ and for each random number an array element which can have higher value is searched for. Therefore, individuals are selected according to their probabilities of selection.

### 2.6.4 Linear Ranking selection

In linear ranking selection (Baker, 1985), first the individuals are ranked according to the fitness values. Those with high fitness values will be ranked high and those with low fitness values will eventually have lower ranks. Then the individuals are selected with a probability that is linearly proportional to the rank of the individuals in the population.

### 2.6.5 Tournament selection

In tournanment selection variant, two individuals are chosen at random and the better of the two individuals is selected with fixed probability p, $0.5 < p \leq 1$ (Goldberg, 1989; Muhlenbein 1989).

### 2.6.6 Elitism

When creating a new population by crossover and mutation, we have a big chance, that we will loose the best chromosome. Elitism is the name of the method that first copies the best chromosome (or few best chromosomes) to the new population. The rest of the population is constructed in ways described above. Elitism can rapidly increase the performance of GA, because it prevents a loss of the best found solution.

### 2.6.7 Encoding

Encoding of chromosomes is the first question to ask when starting to solve a problem with GA. The most common method is the binary encoding, mainly because the first research of GA used this type of encoding and because of its relative simplicity. In binary encoding, every chromosome is a string of bits - 0 or 1, as depicted in Fig. 2.2.

| | |
|---|---|
| Chromosome A | 10110010110010101110010 1 |
| Chromosome B | 11111110000011000001111 1 |

Fig. 2.2 Example of chromosomes with binary encoding

### 2.6.8 Crossover and Mutation

Crossover and mutation are two basic operators of GA. Performance of GA depends on them very much (David et al, 1993). A crossover operator is used to recombine two strings to get a better string. In crossover operation, recombination process creates different individuals in the successive generations by combining material from two individuals of the previous generation. In reproduction, good strings in a population are

probabilistically assigned a larger number of copies and a mating pool is formed. It is important to note that no new strings are formed in the reproduction phase. In the crossover operator, new strings are created by exchanging information among strings of the mating pool.

Mutation adds new information in a random way to the genetic search process and ultimately helps to avoid getting trapped at local optima (Goldberg, 1989). It is an operator that introduces diversity in the population whenever the population tends to become homogeneous due to repeated use of reproduction and crossover operators. Mutation may cause the chromosomes of individuals to be different from those of their parent individuals. Mutation in a way is the process of randomly disturbing genetic information. They operate at the bit level; when the bits are being copied from the current string to the new string, there is probability that each bit may become mutated. This probability is usually a quite small value, called mutation probability

## 2.7 Structural Safety Concepts

The nature of uncertainties and the manner of dealing with them has been a topic of discussion by statisticians, Scientists, Engineers and other professionals for a long time (Der-Kiureghian and Ditlevsen, 2009; Benjamin and Cornell, 1970; Lindley, 2000; Pate-Cornell, 1996; Vrouwenvelder, 2003; Rao *et al*, 2007).

A structure under the study of a designer is not always under a set of predefined or approximate loadings, and since these approximations and uncertainties are not meant to

ignore, generally safety factors are considered to assure reliability. The safety factor ensures a more relaxed mind. However, by considering a safety factor, yet the variations of load (or structural resistance), distribution factor or changes of load probability density will introduce different failure probabilities, which indicate that safety factors do not account for the uncertainties existing within the load or even material properties. In some cases however, a smaller safety factor than unity may even handle the uncertainties due to failure probability, as well fulfilling economical purposes.

The uncertainties in structural parameters such as material properties, external loads, geometry, etc., have caused serious attentions to reliability in structural design and analysis (Ghasemi and Yousefi, 2011). Thus, reliability theory, as a branch of theory of probability, provides a firm framework which can introduce a proper factor of safety when required (Reynolds, 2008; ISSC, 2006; Faber and Sorensen, 2002; Smith, 1976). Thus, any system made of a satisfied reliability index, may be referred to as safe.

Early studies in the field of structural reliability were carried out by Freudenthal (1945), who brought the method of probability and statistics to bear on characterizing the nature of factor of safety. An illuminating account of the historical development of the subject is available in the book by Madsen *et al*, (1986). Besides, this book also provides a rigorous account of reliability index based approaches. Some of the other earlier books relevant to this field are those by Pugsley (1962), Bokotin (1969), Tichy and Vorlicek (1972), Benjamin and Cornell (1970), Ang and Tang (1975, 1984), Leporati (1979), Ditlevsen (1981), Elishakoff (1983), Augusti *et al*, (1984), Yao (1985), Thoft-Christensen and

Morutsu (1986) and Madsen (1988). Works of more recent vintage include those by Wen (1990), Soong and Grigoria (1993), Dai and Wang (1992), Nigam and Narayanan (1994) Melchets (1999) and Ditlevsen and Madsen (2005).

Several specialsed review articles have also been published. Thus, Shinozuka (1983) presents a discussion on definition and calculation of reliability indices. Methods for calculating probability of failure are critically reviewed by Scheller and Stix (1987). Bjerager (1990) presents a state of the art review of structural models and random field reliability models. Developments in the area of stochastic finite element method (SFEM) have been discussed by Nakagiri (1987), Benaroya and Rehak (1988), Shinozuks and Yamazaki (1988), Brenner (1991), Ghanem and Spanos (1991), Shinozuka (1991), Der Kiureghian et al., (1991) Kleiber and Hien (1992) and Liu et al., (1992). Melchers (1999) has outlined the developments of the reliability analysis of existing structures and discussed the needs for further research.

As in other branches of structural mechanics, the emergence of computational power in recent years has strongly influenced the developments in this subject. Other factors that have contributed to the growth of the subject are the increased availability of data.

### 2.7.1 Theoretical Framework on Structural Reliability

The study of structural reliability is concerned with the calculations and prediction of the probability of limit state violation for engineered structures (Melchers, 1999). Ditlevsen and Madsen (2005) considered structural reliability as a method that attempt to treat

rationally, the various sources of uncertainties. According to Tsompanakis and Papadrakakis (2000) structural reliability analysis is a tool that assist the design engineer to take into account all possible uncertainties during the design and construction phases and the lifetime of a structure in order to calculate its probability of failure. While Tsompanakis and Papadrakakis, (2000). To Stanley et al (1978) reliability-base design considers the probability that the structure will last a given length of time against the agents that can cause it to fail.

Reliability-based design is a probabilistic design process where the loads and the strengths of materials and sections represented by their known or postulated distributions, defined in terms of distribution type, mean and standard deviation. The probability of failure ($P_f$) for a specific design case is calculated as the probability that the maximum total load effect exceeds the resistance to failure. This is opposed to limit state design (LSD) method, which is a semi-statistical design process in which the probabilistic aspects are treated at the code development stage (Kadry and Smaili, 2007) in order to define characteristic values and partial safety factors for load and resistance that are used to ensure, on average an acceptably low probability of failure across a full spectrum of design cases.

In the current design codes in Europe, LSD is used as a practical method of incorporating reliability methods in the normal design process. This approach failed to fully address the statistical nature of basic design variable. Issues of uncertainties are lumped conservatively within partial safety factors. A limit state code is calibrated to ensure that

selected appropriate reliability levels, called target values, are attained in design. (Zimmerman *et al*, 1998).

In reliability-based concept, the performance function of a structural system according to a specified mission is given by:

$$M = \text{performance criterion} - \text{given criterion limit} = g(X_1, X_2, \ldots\ldots, X_n) \quad (2.5)$$

in which the $X_i$ ($i = 1, \ldots\ldots, n$) are the n basic random variables (input parameters), with $g(\ )$ being the functional relationship between the random variables and the failure of the system. The performance function can be defined such that the limit state of failure surface, is given by $M = 0$. The failure event is defined as the space where $M > 0$. Thus a probability of failure can be evaluated by the following integral.

$$P_f = \int\int \ldots \int f_x(x_1, \ldots\ldots, x_n) dx_i \ldots dx_n \quad (2.6)$$

Where $f_x$ is the joint density function of $x_1, x_2, \ldots, x_n$ and the integration is performed over the region where $M < 0$. Because each of the basic random variables has a unique distribution and they interact, the integral cannot be easily evaluated. Use is made of approximate method.

Traditionally, the concern of researchers were on the evaluation of structural reliability of steel and concrete structures and/or components. (Rosowsky *et al*, 2002; Jinquan and Baidurya, 2007; Afolayan and Abubakar, 2003; Afolayan and Opeyemi, 2008; Holicky and Retief, 2005; Hassan, 2006; Chinwedu, 2002). Due to the performance of timber structures during recent extreme events, such as the Northridge earthquake, and the

Hurricane Andrew (Mohammed, 2006), Much attention is being tailored on the reliability of wood frame components and assemblies under normal and accidental loading (Rosowsky and Ellingwood, 2002; Rosowsky, 2002; Filiatrault and Folz, 2001, 2002; Afolayan, 2005; van de Lindt, 2005; Foliente *et al*, 2001; Afolayan, 2004; Afolayan and Abdulkareem, 2005; Mohammed, 2006; van de Lindt and Mathew, 2003).

Study on the adequacy of the current design codes, that are based on limit state design philosophy are being assessed, as many attempt were made to calibrate these new generation codes (Holicky and Gurvanessian, 2002; Mike and David, 2002; Nowak, 1995; Ranta-maunus *et al*, 2001; Ranta-maunus and Tomi, 2002; Ranta-maunus, 2004; Trezos and Thomas, 2002; Girgio and Silvia, 2002). This research work is an additional efforts that utilizes the concept of artificial intelligenge, genetic algoritjms in particular.

### 2.7.2 Monte Carlo Simulation Method

One of the approximate methods of evaluating equation (2.9) is the method of Monte Carlo. The method is used to build probability density function (pdf) of the structural system response, as well as to assess the reliability of components or structures or to evaluate the sensitivity of parameter. Monte Carlo simulation consists of drawing samples of the basic variables according to their probabilistic characteristics and then feeding them into performance function (Rubistein, 1981; Kadry, and Smaili, 2007). The major advantages of the Monte Carlo method is that this method is valid for static, but also for dynamic and probabilistic models, with continuous or discrete variables. The main draw back of this method is that it requires often large number of calculations and

can be prohibitive when each calculation involved a long and onerous computer time (Kadry and Smaili, 2007).

To avoid the problem of long computer time in the method of Monte Carlo, it can be interesting to build an approximate mathematical model in the form of a polynomial function, called response surface method (Rajeshekhar, 1993). When a response surface has been determined, the system reliability can be easily assessed with Monte Carlo simulation. The practical problems encountered by the use of the response surface method are in the analysis of strongly non-linear phenomena where it is not obvious to find a family of adequate functions and in the analysis of discontinuous phenomena. Likewise, this approach may also be time consuming when there are larger number of random variables (Cordoso *et al*, 2008).

### 2.7.3 First and Second Order Reliability Method

First and Second Order Reliability Method (FORM/SORM) are other approximation methods. The Safety level of a structure is measured by a reliability index. There are different definitions of reliability index, β. The first was introduces by Cornell in the late 1960s (Cornell, 1969). Given the performance function as"

$$M = R - S = g(X_1, X_2, \ldots\ldots, X_n) \quad (2.7)$$

The mean and the standard deviation of the margin can be defined as

$$\mu_M = \mu_R - \mu_S \quad (2.8)$$

$$\sigma_M = \sqrt{(\sigma^2 + \sigma^2)} \quad (2.9)$$

and the safety index is given by:

$$\beta = \frac{\mu_M}{\mu_S} \tag{2.10}$$

This reliability index is defined as the shortest distance between the failure surface defined by G(x) = 0 and the standardized origin (Fig. 2.4).

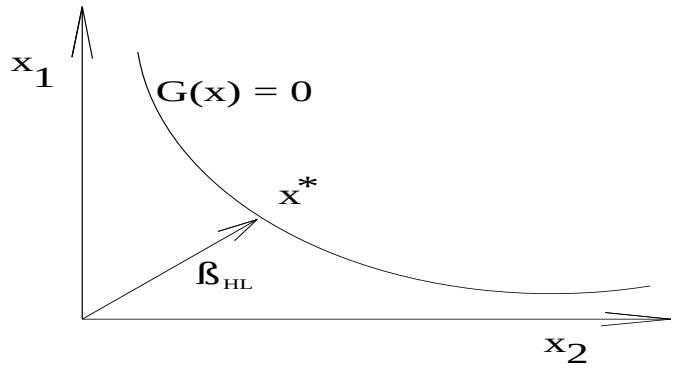



Fig. 2.3 Reliability index β and the limit state function of the standardized space

The standardized variables are expressed as:

$$X = \frac{X-\mu_x}{\sigma_x} \tag{2.11}$$

The point $x^*$ on the failure surface is of considerable importance. It is the design point. If a design is carried out using values from the design point this will result in the highest probability of failure, thereof the name. If the limit state function is linear and the variables are non correlated, the shortest distance, that is $\beta_{HL}$, can be found directly using basic algebra. However, this is seldom the case. When the limit state function is non linear, approximate methods must be considered. For example, geometrical optimization of the problem which can be solved numerically or analytically. The analytical methods

are usually based on a first order Taylor approximation of the limit state function. That is why the method is called First Order Reliability Method (FORM) (Thoft-christensen and Baker, 1982).

An iterative method that can be used to find $\beta_{HL}$, $x^*_i$ and an estimate of the probability of failure is the Rackwitz algorithm (Ang and Tang, 1984). The design point can be expressed in scalar form.

$$x_i^* = -\alpha_i^* \beta_{HL} \tag{2.12}$$

where $\alpha_i^*$ are the direction cosines (the unit vector) in the $x_i^*$ direction.

$$\alpha_i^* = \frac{A}{B} \tag{2.13}$$

$$\text{where } A = \frac{\partial g}{\partial x_i^*} \tag{2.14}$$

$$\text{and} \quad B = \sqrt{[\sum (\frac{\partial g}{\partial x_i^*})^2]} \tag{2.15}$$

First Order Reliability Method (FORM) and the Second Order Reliability Method (SORM) are based on first order approximation of the limit state at the design point (Cheng, 2007; Melchers, 1999; Ditlevsen and Madsen, 2005; Rackwitz, 2000). FORM/SORM require the calculations of the performance function and/or its partial derivatives with respect to the basic random variables at each iteration step.

A large number of calculation of the performance function and/or its partial derivatives are therefore needed (Deng *et al*, 2005). Such calculations might be performed efficiently

when the performance function g (x) can be expressed as an explicit form or analytical form in terms of the basic variables X. When the performance function are implicit, however, such calculations require additional efforts and might be time consuming (Deng *et al*, 2005). In many cases of practical importance, particularly for complicated structures, the limit state functions are usually implicit in terms of the random variables. Therefore, derivatives of the limit state functions are not readily available. Another issue of concern is the capability of the conventional structural reliability methods to detect the global minimum during the search for the reliability index (Rackwitz, 2000).

### 2.7.4 Probabilistic Transformation Method (PTM)

The probabilistic Transformation Method (PTM) developed by Kadry *et al*, 2006 and Kadry and Chateauneuf 2006 is a combination between the finite-element method and the probabilistic transformation method. It evaluate the probability density function (pdf) of a function by multiplying the input pdf by the Jacobian of the inverse function. The idea of PTM is based on the following formula (Hogg and Craig, 1978).

$$F_u(u) = f_p(p) \,.\, \left| J_{p,u} \right|$$

$$= f_b(p) \,.\, \left| \frac{\partial \varphi^{-1}(u)}{\partial u} \right| \qquad (2.16)$$

Where p is the input parameter, u is the response (solution), and $\varphi^{-1}(u)$ is the inverse transformation, which is determined either analytically or numerically.

One advantage of the PTM-FEM technique in the context of reliability analysis is the evaluation of the pdf, of the response in a closed form as opposed to other numerical

methods which give only the first and second moment of response under some condition (Seifedine and Khaled, 2007).

### 2.7.5 Structural Reliability Using Genetic Algorithms

Many human inventions were inspired by nature (Dyer, 2003), notably, the use of artificial intelligence in civil engineering domain. Advancement in artificial intelligence and its applications have led to the development of comprehensive search algorithms that lend themselves to immediate use in the reliability evaluation of structural systems and the identification of dominant failure modes. Areas that attracted considerable research outputs are the genetic algorithms and artificial neural networks (Lagaros and Papadopoulus, 2006).

According to Rackwitz (2001), genetic algorithms (GA) is known to be capable of detecting global minima during the search for the reliability index. The Genetic algorithm is an adaptive heuristic search method based on population genetics. Genetic algorithm were introduced by John Holland in the early 1970s (Manoj *et al*, 2010; Black, 1996) It is an iterative procedure that consists of a constant size population of individuals, each one represented by a finite string of symbols, known as 'genomes', encoding a possible solution in a given problem space. This space referred to as the search space, comprises all possible solution to the problem at hand.

GA-base structural reliability method was first developed by Shao and Murotsu (1999). It consists of generating search directions in the space formed by the random variables

affecting the reliability of a structural system. By following a search path until failure is reached, the reliability index, β, corresponding to the search direction is obtained. Genetic search algorithm simulates the biological process of evolution in an attempt to identify the parameters that is genes that define an optimum system.

Genetic algorithm is an iterative procedure that consists of a constant-size population of individuals, each one represented by a finite string of symbols, known as the "genome", encoding a possible solution in a given problem space, comprises all possible solutions to the problem at hand.

The standard genetic algorithm proceeds as follows (Black, 1996): an initial population of individuals is generated at random or heuristically. Every evolutionary step, known as generation, the individuals in the current population are decoded and evaluated according to some predefined quality criterion, referred to as fitness. The fundamental principle for building an artificial system that reproduces and mimics the working of evolution date back to the biologist Barricelli who formulated an article about artificial methods to realize evolutionary processes (Baricelli, 1957).

The issue of efficiency of GA, which is of concern in all types of applications, is the subject of continuous research (Goldberg and Samtani, 1986; Goldberg, 1989; Grigoriu and Turkstra, 1979). Deng *et al* (2005) proposed a method for improving the efficiency of genetic algorithm during the search for the minimum reliability index of a structural

system. This is based on a splitting technique that shreds a chromosome into many small fragments, which can be easily identified.

A dominant structural failure mode normally occurs when a set of structural members fail. Also, the failure of subsets of those members would lead to the occurrence of 'partial failure' can be identified, then some of these are very likely to contribute to the dominant failure mode. Conversely, a possible solution that does not contain any important 'partial failure' is unlikely to be the solution that would lead to a dominant failure mode. Linkage shredding genetic algorithm for reliability of structural system was proposed by Wang and Ghosn (2005).

The genetic algorithm (GA) that differs from other classical optimization in four ways (Goldberg, 1989) is a part of evolutionary computational technique and probabilistic and global search method. Due to these advantages, the GA has been preferred in wide ranges of optimization problems. In order to apply the genetic algorithm, a population of solutions within a search space is initialized on the contrary of the traditional optimization methods that starts from a single point solution.

The population can be viewed as points in the search space of all solutions to the optimization problem. Each individual in population has a fitness value defined by a fitness function. Then the artificial evolution processes called the genetic loop which mimic natural evolution are applied to produce new candidate solutions. At the end of the process, the newly created generation replaces previous generation and revolution is

repeated until a satisfying solution to the problem is obtained ensuring certain design criteria are satisfied or a maximum number of generations are reached.

### 2.7.6 Reliability Analysis Using Artificial Neural Networks (ANN)

Another search technique inspired by nature is the Artificial Neural Networks ANN. To address the issues of implicit performance functions that are sometimes encountered in structural systems, Deng *et al* (2005) presented three artificial neural network (ANN) based reliability analysis methods. ANN-based Monte Carlo simulation method, ANN-based first order reliability method and ANN-based second order reliability method. These methods employ multi-layer feed forward ANN technique to approximate the implicit performance function.

El-Hewy and Mesbahi (2006) proposed an artificial neural network ANN-based response surface method. The method established the relation between the random variables (input) and structural responses using reliability (FORM) and second order reliability method (SORM), or Monte Carlo simulation (MCS) method, to predict the failure probability. In another development Cordoso *et al* (2008) and Joao *et al* (2008) proposed methodologies for computing the probability of failure by combining Neural Network (NN) and Monte Carlo Simulation (MCS).

The proposed methodology makes use of the capability of ANN to approximate a function for computation of performance measure at lower cost. Cheng *et al* (2008) proposed a method of reliability of structures using ANN-based response surface method.

This method proved to be much more economical to achieve reasonable accuracy when dealing with problems where closed-form failure functions are not available or the estimated probability is extremely small.

### 2.7.7 Properties Affecting Reliability Analysis and Design of Timber Structures

Some peculiarities related to the reliability of timber structures were mentioned in Hoffmeyer *et al* (1999), Forest Products Laboratory, (1999), Gustrafsson *et al* (1998) and Gustafsson (2001). These are:

a. Structural timber is not a manufactured but a graded, natural material
b. The behaviour varies not only between members but also within members
c. The properties depend on the moisture history
d. The properties depend on the loading history
e. The material is anisotropic and failure may be influenced by brittle failure in tension perpendicular to grain.

To take these facts into account it is important that the properties of timber are determined by standardised test and evaluation methods.

## 2.8 Probabilistic Calibration of Material Safety Factors

For the past several years, structural performance has always been measured by the use of a factor of safety approach in design (Detlevsen and Madsen, 2005). As long as the strength of a component exceeds the stress on it, safety is conjectured. But, then, the larger the gap that exists in the strength-stress interference, the greater the reliability and

of course, the heavier a structure becomes. Conversely, the smaller the gap, the lower the reliability, but the lighter the structure too (Detlevsen and Madsen, 2005). Although, the concept of safety factor design has often produced an adequate gap between stress and strength, the reliability has always remained unmeasured. There is also the possibility of a safety factor being too large or too small in a design in spite of the historical success of the philosophy. In order to offer a structure that is neither too heavy nor too light and at the same time satisfy a target reliability while accounting for randomness in the intervening variables, full probabilistic thinking has been described as  a good explicit (Andre and Antonio, 2010).

# CHAPTER THREE

# MATERIALS AND METHODS

## 3.1 Preamble

Reliable information regarding the strength and stiffness of timber in Nigeria, is one of the major obstacles to their use in building construction application, based on the requirements of the EN 338 and Eurocode 5. The allocation of a specie to a strength class allows engineers to use the mechanical properties of the strength class in limit state design of timber structures.

One of the objectives of the testing programme in this study was to allocate each of the five timber species selected to appropriate strength classes. The possible strength classes are those contained in EN 338 (2009).

The scientific names of the species selected for test in the study, are *Alstonia boonei* (Ahun), *Triplochiton Scleroxylon* (Obeche), *Terminalia Ivorensis* (Black Afara), *Terminalia superba* (White Afara) and *Lophira Alata* (Ekki). These timber species are commercially available on the open market of Nigeria, and are widely used especially in roof truss, rail road cross-ties and timber bridge construction.

The number and choice of required tests were determined by the requirements of EN 384 (2004). The programme of test was designed to determine the bending strength, stiffness (Modulus of elasticity), density and moisture content of the five species. Testing was conducted in accordance with EN 408 (2004) in the department of civil engineering

heavy structures laboratory.. The Test data has been used to derive characteristic values for bending strength, modulus of elasticity and density in accordance with BS EN 384 (2004).

### 3.2 Materials

The programme of tests was designed to determine the bending strength, stiffness, density and moisture content of the five timber species. Four point bending tests was conducted in accordance with EN 408 (2004). For the purpose of comparison, three point bending tests were also conducted in accordance with ASTM D 143 (2006). Fourty samples were used for each test as prescribed in EN 384 (2004). The fourty samples were made of four sets of ten sub-samples. Each sample was drawned randomly from a different supplier, based on the assumption that, the samples were independent. The samples under test were obtained from Kaduna and Sabon gari Zaria timber sheds. Four independent samples of each species were therefore made available for the tests. The sample from Kaduna were first prepared for the required tests and delivered at the Civil Engineering Department laboratory, Ahmadu Bello University, Zaria on 24$^{th}$ June, 2009, while the Zaria samples were delivered on 6$^{th}$ July, 2009 after preparation.

The test specimens were first subjected to protected exterior drying for three months, for the samples to attain equilibrium moisture content environmentally. The samples were then exposed to laboratory condition. Conditioning requirement is that, prior to testing, specimens must be conditioned to (20±2) $^{0}$C and (65±5) % relative humidity (EN 384,

2004). Testing commenced on 12th March, 2010 at the concrete and material laboratory of the department of civil engineering, Ahmadu Bello University, Zaria.

There were two groups of test samples; test beams for bending test, and small sections for density and moisture content test. Each of the bending test beams was nominally 50mm wide and 75mm depth, and 1500 mm long for the four point bending test, and 50 mm x 75 mm x 750 for the three-point bending test. The test pieces for density and moisture content test, are 70mm x 40mm x 40mm. The test specimens were prepared at the wood workshop of school of engineering technology, Nuhu Bamalli polytechnic Zaria. Plate I shows the researcher preparing samples for bending tests.

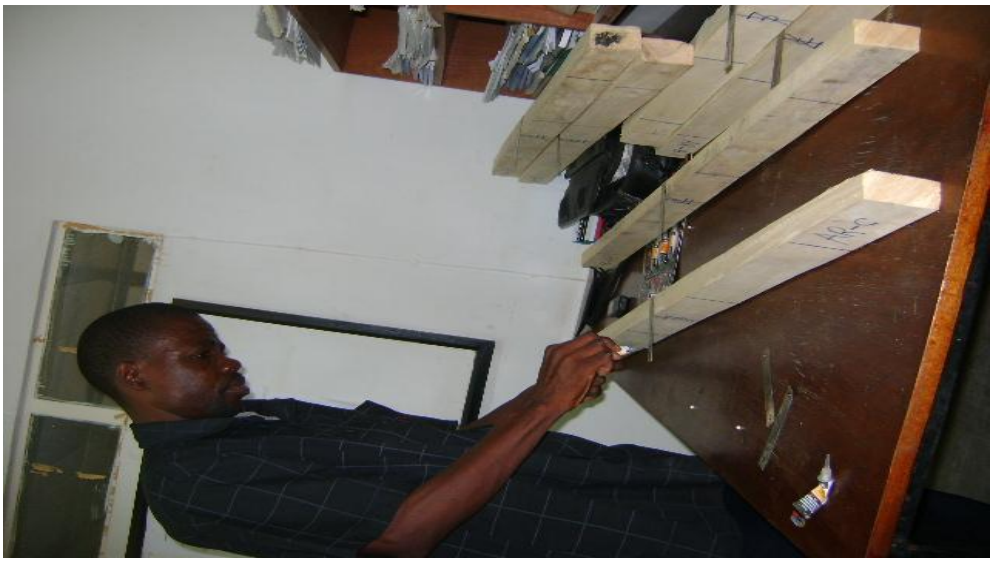


Plate I. Samples Preparation

### 3.3 Methods of Laboratory Experiments and Data Analysis

#### 3.3.2 Determination of Density and Moisture Content

The density and moisture content tests were done in accordance with EN 13153-1 (2002), The specimens were kept in an electric oven at $105^0$C for 24 hours. The laboratory equipment and apparatus used for the tests include: High precision electronic weighing balance with accuracy of 0.01%. Plastic bags for temporary storage of samples prior to test. Drying chamber (Oven).

The density of samples were determined using equations (3.1) and (3.2).

$$\text{Wet density } \rho_b = \frac{m_b}{V} \qquad (3.1)$$

$$\text{Dry density } \rho_d = \frac{m_o}{V} \qquad (3.2)$$

The moisture content of samples were obtained using equation. 3.3.

$$\text{Moisture content (\%)} = \frac{m_b - m_0}{m_0} \text{x}100 \qquad (3.3)$$

Where, $m_u$ is the bulk mass of sample in kg. $M_0$ is the oven dry mass in kg. V is the volume of sample in m. $\rho_b$ is the bulk density. $\rho_d$ is the dry density in kg/m$^3$. $\rho_d$ is the dry density in kg/m$^3$.

Plate II shows the specimens in air tight polythene bags prior to testing. Also, Plate III shows the oven dring procedure of the test specimens. The generated data on moisture content and density is presented in Tables 3.1 to 3.4.



Plate II Specimens in an air tight polythene bags



Plate III: Placement of Specimens in a drying Oven

Table 3.1 Moisture content (Sampled timber pieces set A)

| Sample Mark | Moisture Content (%) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia Ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 32.0 | 12.8 | 19.9 | 25.4 | 11.9 |
| 2 | 18.4 | 10.8 | 19.5 | 19.1 | 13.5 |
| 3 | 23.4 | 13.3 | 14.1 | 28.5 | 14.4 |
| 4 | 27.3 | 13.1 | 17.8 | 24.8 | 16.9 |
| 5 | 15.5 | 28.5 | 17.5 | 16.8 | 14.7 |
| 6 | 32.6 | 20.1 | 12.1 | 26.1 | 13.5 |
| 7 | 24.7 | 7.7 | 21.1 | 16.7 | 15.7 |
| 8 | 8.4 | 14.7 | 17.5 | 22.1 | 15.1 |
| 9 | 12.7 | 15.3 | 22.8 | 17.9 | 17.1 |
| 10 | 17.6 | 16.5 | 25.7 | 22.0 | 15.3 |
| 11 | 25.2 | 15.3 | 17.5 | 16.1 | 12.2 |
| 12 | 2.3 | 14.1 | 20.3 | 21.6 | 17.0 |
| 13 | 31.9 | 22.0 | 18.7 | 21.7 | 16.2 |
| 14 | 13.7 | 16.7 | 23.0 | 19.2 | 15.5 |
| 15 | 28.1 | 19.1 | 12.3 | 17.8 | 12.5 |
| 16 | 21.6 | 17.4 | 20.2 | 23.4 | 15.7 |
| 17 | 28.4 | 13.0 | 18.9 | 19.3 | 14.5 |
| 18 | 8.9 | 16.6 | 21.8 | 25.1 | 14.9 |
| 19 | 17.1 | 25.0 | 22.8 | 20.4 | 15.9 |
| 20 | 27.3 | 14.8 | 16.5 | 27.5 | 16.2 |

Table 3.2 Moisture content (Sampled timber pieces set B)

| Sample Mark | Moisture Content (%) | | | | |
|---|---|---|---|---|---|
| | *Alstonia boonei* | *Triplochiton scleroxylon* | *Terminalia Ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 27.1 | 19.9 | 16.7 | 21.7 | 15.3 |
| 2 | 17.2 | 15.8 | 16.5 | 22.6 | 17.3 |
| 3 | 33.2 | 20.2 | 25.5 | 20.0 | 15.6 |
| 4 | 31.1 | 15.7 | 16.4 | 19.8 | 16.3 |
| 5 | 23.7 | 19.7 | 14.1 | 17.9 | 16.1 |
| 6 | 22.1 | 26.4 | 24.7 | 32.5 | 16.2 |
| 7 | 21.2 | 16.4 | 13.9 | 20.0 | 18.8 |
| 8 | 15.3 | 14.3 | 26.1 | 19.1 | 17.0 |
| 9 | 26.1 | 21.8 | 23.7 | 24.8 | 16.5 |
| 10 | 15.9 | 15.2 | 16.3 | 21.7 | 14.9 |
| 11 | 11.6 | 20.5 | 17.8 | 11.7 | 14.5 |
| 12 | 2.0 | 26.4 | 10.7 | 25.2 | 13.6 |
| 13 | 32.8 | 17.6 | 12.8 | 24.8 | 13.3 |
| 14 | 16.2 | 19.0 | 11.7 | 21.8 | 14.2 |
| 15 | 11.4 | 15.0 | 16.4 | 23.6 | 14.2 |
| 16 | 23.2 | 19.6 | 14.6 | 15.8 | 15.8 |
| 17 | 17.3 | 26.4 | 15.4 | 23.3 | 13.9 |
| 18 | 12.2 | 8.6 | 20.3 | 14.8 | 15.7 |
| 19 | 18.0 | 17.9 | 20.1 | 22.9 | 17.7 |
| 20 | 29.5 | 14.7 | 23.3 | 13.0 | 15.8 |

Table 3.3 Density (Sampled timber pieces set A)

| Sample Mark | Density ($kg/m^3$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia boonei* | *Triplochiton scleroxylon* | *Terminalia Ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 365.7 | 434.0 | 418.1 | 562.4 | 923.0 |
| 2 | 378.9 | 351.4 | 384.1 | 456.0 | 929.3 |
| 3 | 363.1 | 405.1 | 405.3 | 617.6 | 994.9 |
| 4 | 371.1 | 356.9 | 696.0 | 369.9 | 917.1 |
| 5 | 370.1 | 367.1 | 368.3 | 465.0 | 992.4 |
| 6 | 358.7 | 303.1 | 434.5 | 580.1 | 967.7 |
| 7 | 351.1 | 475.7 | 481.1 | 299.4 | 925.1 |
| 8 | 348.3 | 340.8 | 539.9 | 509.7 | 958.0 |
| 9 | 356.0 | 578.1 | 503.4 | 554.6 | 971.7 |
| 10 | 344.2 | 443.8 | 520.7 | 558.5 | 1010.6 |
| 11 | 369.4 | 323.1 | 465.1 | 475.1 | 1026.4 |
| 12 | 363.9 | 389.0 | 490.2 | 525.8 | 908.2 |
| 13 | 361.7 | 418.5 | 398.8 | 645.5 | 906.6 |
| 14 | 358.2 | 450.5 | 353.4 | 375.0 | 890.3 |
| 15 | 354.1 | 253.8 | 469.6 | 485.9 | 970.7 |
| 16 | 372.3 | 339.9 | 537.1 | 593.2 | 873.5 |
| 17 | 367.5 | 428.6 | 508.8 | 624.7 | 925.3 |
| 18 | 361.4 | 363.7 | 377.3 | 604.9 | 990.3 |
| 19 | 375.0 | 482.6 | 614.7 | 612.2 | 1056.2 |
| 20 | 359.5 | 268.3 | 596.8 | 515.4 | 839.8 |

Table 3.4 Density (Sampled timber pieces set B)

| Sample Mark | Density ($kg/m^3$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia Ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 376.3 | 319.0 | 432.8 | 583.0 | 1020.6 |
| 2 | 359.6 | 370.6 | 549.8 | 499.7 | 915.5 |
| 3 | 367.5 | 391.7 | 535.1 | 560.3 | 1002.7 |
| 4 | 360.5 | 391.7 | 400.9 | 506.7 | 1076.4 |
| 5 | 347.2 | 424.4 | 426.7 | 469.9 | 958.0 |
| 6 | 360.1 | 426.6 | 485.1 | 684.7 | 1030.6 |
| 7 | 346.3 | 255.8 | 545.4 | 666.5 | 963.6 |
| 8 | 373.1 | 448.6 | 352.0 | 476.6 | 1011.3 |
| 9 | 364.2 | 418.9 | 460.8 | 514.6 | 973.5 |
| 10 | 381.1 | 414.7 | 386.2 | 547.8 | 946.8 |
| 11 | 346.3 | 386.8 | 511.5 | 452.2 | 993.3 |
| 12 | 379.0 | 458.7 | 365.3 | 515.4 | 949.8 |
| 13 | 350.6 | 360.6 | 286.2 | 495.1 | 933.1 |
| 14 | 348.2 | 428.8 | 367.4 | 400.3 | 1007.5 |
| 15 | 385.5 | 346.0 | 338.4 | 485.4 | 910.6 |
| 16 | 339.8 | 356.7 | 481.9 | 433.4 | 929.3 |
| 17 | 340.0 | 399.5 | 464.3 | 606.3 | 969.1 |
| 18 | 351.8 | 394.6 | 498.0 | 529.1 | 990.8 |
| 19 | 352.8 | 408.0 | 405.5 | 523.7 | 870.7 |
| 20 | 358.3 | 405.7 | 472.4 | 400.4 | 865.2 |

### 3.3.2 Three-point Bending Test Method

Three-point bending test is based the central deflection of test piece relative to the support according to ASTM D 143 (2006). The was first used to generate test data for computation of modulus of elasticity and bending strength. The test set-up is shown in Fig. 3.1.

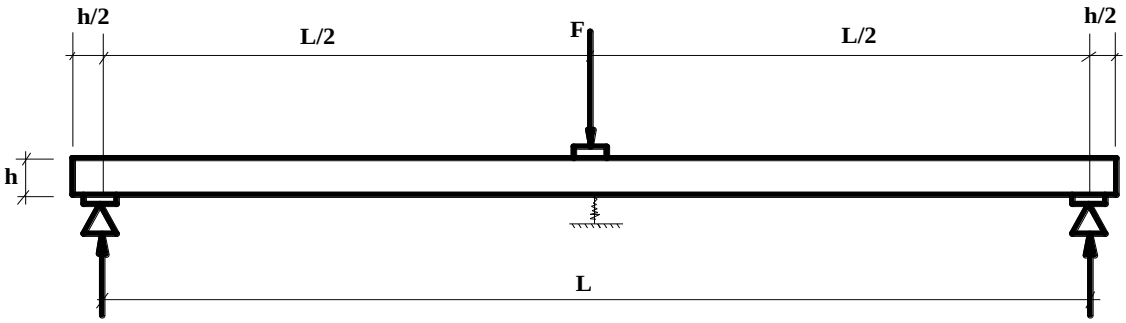



Fig. 3.1: Three-point bending test set-up (ASTM D 143, 2006)

The Modulus of Elasticity, $E_m$, was calculated for each specimen based on linear regression analysis of the load-deflection data collected between applied load levels of 1000N and 5000N. The bending strength $f_m$ was calculated from the maximum recorded load.

$$E_m = \frac{L^3(F_2-F_1)}{48I(w_2-w_1)} \quad (3.4)$$

Where $\frac{(F_2-F_1)}{(w_2-w_1)}$ = the slope of the least square regression line. L = span between the beam supports in mm. $F_2-F_1$ = the increment of load on the straight line of the load-deformation least square regression line in N. $w_2-w_1$ = the increment of deformation corresponding to $(F_2-F_1)$ in mm. I = the moment of inertia (second moment of area) in $mm^4$

The bending strength $f_m$ is computed using equation 3.5.

$$f_m(N/mm^2) = \frac{3F_{max}L}{2bh^2} \quad (3.5)$$

where L = span between the beam supports in mm. $F_{max}$ = Maximum load at piece rupture in Newton. b = with of test piece in mm. h = depth of test piece in mm. The generated three-point bending tests data are presented in Tables 3.5 to 3.8.

### 3.3.3 The Four Point Bending Test Method

The determination of global modulus of elasticity of timber using four-point bending test, is based on the application of loading at the central third of the test piece. Local Modulus of Elasticity was first recommended in the previous edition of EN 408 (2004). This method for the determination of local modulus of elasticity do not give consistent and reliable results (Bosttron *et al*, 1996; Bostron 1999; Solli 1996), because the determination of the MOE is based on measurement of a deflection over a relatively short span between the loading point. The deflections are small, often less than 1 mm. Hence the method is sensitive to measurement errors. Such errors can be caused by twisting of the timber during the test (Solli 1996).

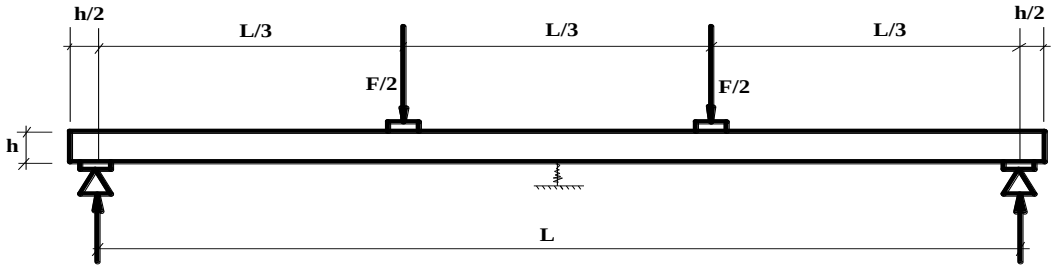



Fig. 3.2 Four-point bending test set-up (EN 408, 2004)

In the current version of EN 408 (EN 408, 2004) a method, that consider bending deflection of the entire span of the timber specimen relative to the end supports was additionally specied. The Modulus of Elasticity obtained based on this method is known as Global Modulus of Elasticity $E_{global}$. The testing arrangement is shown in Figure 3.2 (EN 408, 2004). The Global Modulus of Elasticity and the Bending Strength were obtained from equations 3.6 and 3.7 as follows:

$$E_m = \frac{L^3(F_2-F_1)}{4.7bh^3(w_2-w_1)} \tag{3.6}$$

$$f_m = \frac{aF_{max}}{2W} \tag{3.7}$$

where $(F_2-F_1)$, L, b, h. $(w_2-w_1)$ and $F_{max}$ are as defined above. a is the distance between one load and the nearest support in mm. W is the section modulus of the beam in $mm^3$. The generated four-point bending tests data are presented in Tables 3.9 to 3.12.

Table 3.5 Three-point Modulus of Elasticity Test results (Sampled timber pieces set A)

| Piece Number | Modulus of Elasticity ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 6488.9 | 5537.2 | 10979.7 | 11628.8 | 21639.6 |
| 2 | 6118.7 | 6634.9 | 11463.1 | 15610.1 | 18405.6 |
| 3 | 6197.4 | 6799.7 | 14182.8 | 12685.9 | 16448.7 |
| 4 | 5923.7 | 7092.0 | 11693.1 | 15653.8 | 15833.7 |
| 5 | 7996.7 | 8264.1 | 12209.7 | 8165.6 | 17814.9 |
| 6 | 6884.0 | 7658.2 | 10178.7 | 12274.6 | 17370.2 |
| 7 | 7306.7 | 7050.7 | 11613.8 | 12471.7 | 20028.1 |
| 8 | 7769.1 | 7760.4 | 12102.5 | 14614.3 | 21108.1 |
| 9 | 8042.2 | 9364.6 | 14133.3 | 16052.9 | 19611.3 |
| 10 | 7458.6 | 6908.5 | 12352.2 | 13600.7 | 19947.4 |
| 11 | 7135.8 | 5814.8 | 12667.0 | 10737.2 | 19233.5 |
| 12 | 7833.8 | 8892.2 | 13215.5 | 11832.6 | 16952.7 |
| 13 | 6360.6 | 7060.2 | 9098.1 | 13729.8 | 17133.5 |
| 14 | 7070.4 | 5683.2 | 8441.6 | 12193.9 | 17655.1 |
| 15 | 7528.8 | 5983.5 | 10695.9 | 12540.6 | 15493.7 |
| 16 | 7682.2 | 7905.5 | 12305.9 | 13388.1 | 17646.7 |
| 17 | 7500.9 | 7042.2 | 10176.1 | 12227.4 | 22603.2 |
| 18 | 8574.7 | 8045.1 | 13151.3 | 13081.5 | 16104.4 |
| 19 | 8011.4 | 7548.3 | 9944.9 | 10983.4 | 21007.4 |
| 20 | 6691.0 | 7911.5 | 13330.3 | 12176.0 | 14821.3 |

Table 3.6 Three-point Modulus of Elasticity Test results (Sampled timber pieces set B)

| Piece Number | Modulus of Elasticity ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 6710.4 | 6410.1 | 11472.6 | 13483.4 | 17935.8 |
| 2 | 7291.4 | 7485.8 | 12022.3 | 12173.7 | 21063.7 |
| 3 | 7425.9 | 9047.9 | 12310.2 | 16026.6 | 14254.9 |
| 4 | 6714.8 | 6871.0 | 10815.0 | 11075.4 | 14240.5 |
| 5 | 7452.0 | 5424.6 | 12470.1 | 12375.9 | 22594.7 |
| 6 | 7054.2 | 6583.5 | 9838.6 | 11892.7 | 13740.0 |
| 7 | 8139.7 | 7533.0 | 11373.4 | 15194.9 | 19068.2 |
| 8 | 7092.5 | 7675.7 | 12731.4 | 13031.9 | 15276.2 |
| 9 | 6501.4 | 5470.6 | 12865.9 | 9712.9 | 21660.0 |
| 10 | 6790.0 | 4638.2 | 11843.7 | 11137.9 | 15617.5 |
| 11 | 7959.9 | 6916.5 | 12213.1 | 16868.5 | 16826.4 |
| 12 | 7765.2 | 9014.7 | 11246.3 | 19291.4 | 16984.7 |
| 13 | 7294.7 | 6978.2 | 13930.0 | 14640.9 | 17876.2 |
| 14 | 7385.0 | 9183.7 | 10787.9 | 8786.4 | 17795.8 |
| 15 | 7983.1 | 5626.6 | 11644.3 | 17850.1 | 19265.8 |
| 16 | 7844.3 | 5873.6 | 12523.0 | 19288.4 | 16701.4 |
| 17 | 7223.8 | 5576.9 | 11968.9 | 14395.9 | 14965.8 |
| 18 | 7877.4 | 3853.4 | 8519.0 | 9683.9 | 21100.1 |
| 19 | 8074.7 | 5056.8 | 13359.2 | 11877.8 | 18017.5 |
| 20 | 7229.4 | 6979.4 | 11715.7 | 15283.8 | 18317.1 |

Table 3.7 Three-point Bending Strength Test results (Sampled timber pieces set A)

| Piece Number | Bending Strength ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 36.8 | 60.5 | 96.0 | 87.7 | 110.4 |
| 2 | 49.0 | 53.2 | 42.6 | 66.9 | 91.8 |
| 3 | 43.9 | 50.1 | 73.6 | 86.9 | 122.4 |
| 4 | 42.3 | 45.4 | 44.8 | 81.6 | 88.8 |
| 5 | 39.7 | 55.9 | 77.7 | 69.7 | 72.2 |
| 6 | 36.7 | 52.0 | 78.4 | 106.7 | 86.7 |
| 7 | 36.0 | 46.9 | 53.3 | 92.1 | 87.9 |
| 8 | 36.9 | 45.4 | 81.3 | 81.6 | 76.8 |
| 9 | 39.1 | 56.7 | 57.4 | 86.3 | 101.6 |
| 10 | 33.6 | 58.1 | 59.6 | 76.9 | 62.8 |
| 11 | 28.1 | 41.2 | 70.1 | 76.3 | 77.5 |
| 12 | 50.2 | 54.3 | 64.6 | 65.1 | 108.8 |
| 13 | 34.2 | 55.6 | 46.1 | 97.3 | 101.5 |
| 14 | 23.8 | 44.9 | 63.7 | 51.8 | 75.2 |
| 15 | 39.5 | 44.1 | 77.6 | 90.6 | 60.5 |
| 16 | 34.8 | 52.6 | 46.0 | 65.8 | 98.6 |
| 17 | 31.3 | 47.1 | 65.9 | 80.3 | 74.0 |
| 18 | 41.5 | 53.8 | 62.6 | 69.7 | 78.6 |
| 19 | 29.3 | 53.2 | 68.0 | 76.4 | 103.2 |
| 20 | 43.6 | 50.6 | 53.7 | 59.2 | 97.7 |

Table 3.8 Three-point Bending Strength Test results (Sampled timber pieces set B)

| Piece Number | Bending Strength ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 32.3 | 46.5 | 65.0 | 70.4 | 105.4 |
| 2 | 47.4 | 49.6 | 68.4 | 86.0 | 87.7 |
| 3 | 28.3 | 51.6 | 59.0 | 105.7 | 113.8 |
| 4 | 36.2 | 45.6 | 70.1 | 76.3 | 69.8 |
| 5 | 45.3 | 47.5 | 67.1 | 68.3 | 100.1 |
| 6 | 28.5 | 58.0 | 90.3 | 59.3 | 86.6 |
| 7 | 41.8 | 55.8 | 50.3 | 76.7 | 67.1 |
| 8 | 49.4 | 58.4 | 53.1 | 69.3 | 77.2 |
| 9 | 24.5 | 46.9 | 54.8 | 89.8 | 111.1 |
| 10 | 44.5 | 42.1 | 58.6 | 93.1 | 113.5 |
| 11 | 34.7 | 51.8 | 55.0 | 80.5 | 104.7 |
| 12 | 36.8 | 50.0 | 55.9 | 78.6 | 116.9 |
| 13 | 47.0 | 49.6 | 67.2 | 89.2 | 83.1 |
| 14 | 40.1 | 53.5 | 36.8 | 104.7 | 73.1 |
| 15 | 37.8 | 48.8 | 75.2 | 89.7 | 88.4 |
| 16 | 30.3 | 50.1 | 87.4 | 81.6 | 119.4 |
| 17 | 36.9 | 55.8 | 63.7 | 68.7 | 87.0 |
| 18 | 37.9 | 51.1 | 62.1 | 72.5 | 91.3 |
| 19 | 36.2 | 52.6 | 85.0 | 90.6 | 100.1 |
| 20 | 43.2 | 54.4 | 77.3 | 80.7 | 103.3 |

Table 3.9 Four-point Modulus of Elasticity Test results (Sampled timber pieces set A)

| Piece Number | Modulus of Elasticity ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 7986.6 | 4388.3 | 11913.0 | 10089.0 | 26487.2 |
| 2 | 8009.6 | 6146.0 | 11377.3 | 11556.2 | 22250.2 |
| 3 | 8418.6 | 5870.2 | 13621.0 | 9623.2 | 20852.2 |
| 4 | 7928.9 | 5707.4 | 14286.4 | 11304.0 | 22088.6 |
| 5 | 9202.5 | 10594.4 | 10394.3 | 10609.2 | 20823.6 |
| 6 | 8342.2 | 4601.3 | 9622.2 | 9048.4 | 24960.7 |
| 7 | 8786.8 | 9716.3 | 10461.9 | 11662.5 | 16998.2 |
| 8 | 7795.6 | 6215.2 | 17141.3 | 11135.2 | 21894.6 |
| 9 | 8267.0 | 3436.4 | 14791.3 | 12123.4 | 27892.3 |
| 10 | 8665.0 | 7379.1 | 13978.2 | 11941.3 | 21570.4 |
| 11 | 8450.2 | 6408.5 | 14897.3 | 15351.3 | 23500.4 |
| 12 | 8812.2 | 6449.2 | 15812.1 | 11433.6 | 25060.6 |
| 13 | 8387.0 | 7751.4 | 11204.9 | 15125.5 | 22815.1 |
| 14 | 8468.3 | 5560.7 | 7199.1 | 14313.7 | 19386.7 |
| 15 | 7986.2 | 5635.6 | 15986.9 | 7822.9 | 25298.4 |
| 16 | 7799.6 | 4341.9 | 10282.8 | 15482.3 | 23562.8 |
| 17 | 8046.8 | 6745.6 | 16742.1 | 8942.7 | 18271.4 |
| 18 | 7836.7 | 6553.8 | 11252.6 | 10962.3 | 24186.4 |
| 19 | 8255.8 | 3982.3 | 9418.4 | 10790.8 | 22453.9 |
| 20 | 8645.4 | 4811.1 | 11344.6 | 11808.7 | 22919.3 |

Table 3.10 Four-point Modulus of Elasticity Test results (Sampled timber pieces set B)

| Piece Number | Modulus of Elasticity ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 8399.2 | 3291.2 | 12769.3 | 16957.6 | 29686.2 |
| 2 | 8202.6 | 6437.6 | 9810.8 | 8045.3 | 21546.2 |
| 3 | 8695.3 | 5773.8 | 16165.5 | 11221.0 | 22120.4 |
| 4 | 7993.4 | 5678.0 | 10969.0 | 8898.1 | 22288.1 |
| 5 | 8069.1 | 7428.9 | 16237.5 | 15599.3 | 23467.7 |
| 6 | 8105.2 | 8289.8 | 12356.7 | 8233.8 | 22105.8 |
| 7 | 8184.9 | 7395.4 | 9705.7 | 13119.1 | 22336.2 |
| 8 | 7331.8 | 8113.6 | 13932.9 | 11633.7 | 21354.4 |
| 9 | 7546.9 | 3368.8 | 9584.4 | 12697.3 | 19940.2 |
| 10 | 8346.4 | 6260.4 | 11881.5 | 10811.6 | 24159.8 |
| 11 | 7836.6 | 4883.0 | 13444.8 | 10075.6 | 24699.1 |
| 12 | 8398.8 | 8623.8 | 9751.3 | 13762.6 | 21602.2 |
| 13 | 6751.5 | 7035.9 | 13576.5 | 14900.2 | 16454.1 |
| 14 | 8424.0 | 6610.3 | 12771.5 | 14110.7 | 20550.4 |
| 15 | 8385.8 | 7141.4 | 7237.9 | 8610.8 | 24402.9 |
| 16 | 7787.1 | 3510.2 | 6639.2 | 6078.1 | 20549.5 |
| 17 | 9108.2 | 5678.1 | 10650.3 | 12304.9 | 26104.5 |
| 18 | 7704.6 | 6131.3 | 12444.8 | 13198.7 | 27087.7 |
| 19 | 8353.2 | 5622.6 | 11841.4 | 10804.4 | 25142.4 |
| 20 | 7965.3 | 5943.2 | 12943.9 | 12388.5 | 21119.0 |

Table 3.11 Four-point Bending Strength Test results (Sampled timber pieces set A)

| Piece Number | Bending Strength ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 36.8 | 60.5 | 96.0 | 87.7 | 66.5 |
| 2 | 49.0 | 53.2 | 42.6 | 66.9 | 50.7 |
| 3 | 43.9 | 50.1 | 73.6 | 86.9 | 65.9 |
| 4 | 42.3 | 45.4 | 44.8 | 81.6 | 61.9 |
| 5 | 39.7 | 55.9 | 77.7 | 69.7 | 52.9 |
| 6 | 36.7 | 52.0 | 78.4 | 106.7 | 81.0 |
| 7 | 36.0 | 46.9 | 53.3 | 92.1 | 69.9 |
| 8 | 36.9 | 45.4 | 81.3 | 81.6 | 61.9 |
| 9 | 39.1 | 56.7 | 57.4 | 86.3 | 65.5 |
| 10 | 33.6 | 58.1 | 59.6 | 76.9 | 58.4 |
| 11 | 28.1 | 41.2 | 70.1 | 76.3 | 57.9 |
| 12 | 50.2 | 54.3 | 64.6 | 65.1 | 49.4 |
| 13 | 34.2 | 55.6 | 46.1 | 97.3 | 73.8 |
| 14 | 23.8 | 44.9 | 63.7 | 51.8 | 39.3 |
| 15 | 39.5 | 44.1 | 77.6 | 90.6 | 68.7 |
| 16 | 34.8 | 52.6 | 46.0 | 65.8 | 49.9 |
| 17 | 31.3 | 47.1 | 65.9 | 80.3 | 61.0 |
| 18 | 41.5 | 53.8 | 62.6 | 69.7 | 52.9 |
| 19 | 29.3 | 53.2 | 68.0 | 76.4 | 57.9 |
| 20 | 43.6 | 50.6 | 53.7 | 59.2 | 44.9 |

Table 3.12 Four-point Bending Strength Test results (Sampled timber pieces set B)

| Piece Number | Bending Strength ($N/mm^2$) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia ivorensis* | *Terminalia superba* | *Lophira Alata* |
| 1 | 32.3 | 46.5 | 65.0 | 70.4 | 53.4 |
| 2 | 47.4 | 49.6 | 68.4 | 86.0 | 65.3 |
| 3 | 28.3 | 51.6 | 59.0 | 105.7 | 80.2 |
| 4 | 36.2 | 45.6 | 70.1 | 76.3 | 57.9 |
| 5 | 45.3 | 47.5 | 67.1 | 68.3 | 51.9 |
| 6 | 28.5 | 58.0 | 90.3 | 59.3 | 45.0 |
| 7 | 41.8 | 55.8 | 50.3 | 76.7 | 58.2 |
| 8 | 49.4 | 58.4 | 53.1 | 69.3 | 52.6 |
| 9 | 24.5 | 46.9 | 54.8 | 89.8 | 68.1 |
| 10 | 44.5 | 42.1 | 58.6 | 93.1 | 70.6 |
| 11 | 34.7 | 51.8 | 55.0 | 80.5 | 61.1 |
| 12 | 36.8 | 50.0 | 55.9 | 78.6 | 59.7 |
| 13 | 47.0 | 49.6 | 67.2 | 89.2 | 67.7 |
| 14 | 40.1 | 53.5 | 36.8 | 104.7 | 79.4 |
| 15 | 37.8 | 48.8 | 75.2 | 89.7 | 68.1 |
| 16 | 30.3 | 50.1 | 87.4 | 81.6 | 61.9 |
| 17 | 36.9 | 55.8 | 63.7 | 68.7 | 52.1 |
| 18 | 37.9 | 51.1 | 62.1 | 72.5 | 55.0 |
| 19 | 36.2 | 52.6 | 85.0 | 90.6 | 68.8 |
| 20 | 43.2 | 54.4 | 77.3 | 80.7 | 61.2 |

### 3.3.4 Adjustment Factors for Bending Strength

EN 408 (2004) required that, the test piece shall be systematically loaded in bending over a span 18 times its depth (Clause 10.2). The reference depth is 150mm. For test pieces that do not meet the the EN 408 (2004) dimesnsion specification, EN 384 (2004) recommended adjustment factors for bending and tensile strengths only (Clause 5.3.4.3). According to EN 384 (2004), the bending and tensile strength shall be adjusted to 150mm reference depth by deviding by an adjustment factor $k_h$ given by:

$$k_h = \frac{}{} 0.2 \qquad (3.8)$$

Also, for length other that specified in EN 408 (2004), the bending strength be adjusted by:

$$K_l = \left(\frac{l_{es}}{l_{et}}\right)^{0.2} \qquad (3.9)$$

Where, h is the actual depth of the test peace in mm. $l_{es}$ and $l_{et}$ are calculated as follows:

$$l_{es} \text{ and } l_{et} = l + 5a_f \qquad (3.10)$$

where $a_f$ and l are the distance between the point load and the overall length of the piece respectively, having the values of the standard test procedure and for the practical test piece.

### 3.3.5 Derived Material Properties

The modulus of elasticity, bending strength and density obtained from the laboratory are known as the reference properties of timber. Other strength and stiffness properties are calculated from the reference properties in accordance with the recommendation of EN 384 (2004) and JCSS, (2006). The other material properties include:

1. $R_{t,0}$ (Tension Strength parallel to grain)

2. $R_{t,90}$ (Tension Strength perpendicular to grain)
3. $E_{t,0}$ (Tension Modulus of Elasticity parallel to grain)
4. $E_{t,90}$ (Tension Modulus of Elasticity perpendicular to grain)
5. $R_{c,0}$ (Compression strength parallel to grain)
6. $R_{c,90}$ (Compression strength perpendicular to grain)
7. $E_{c,0}$ (Compression Modulus of Elasticity parallel to grain)
8. $E_{c,90}$ (Compression Modulus of Elasticity perpendicular to grain)
9. $R_v$ (Shear Strength)
10. $G_v$ (Shear Modulus)
11. $R_{h,0}$ (Embedding Strength parallel to grain)
12. $R_{h,90}$ (Embedding Strength perpendicular to grain.

The relationships between the reference properties of timber (bending strength, bending modulus of elasticity and density) and other timber properties (tension parallel and perpendicular to grain, compression parallel and perpendicular to grain etc) were given as (EN 384, 2004; JCSS, 2006):

$$R_{t,0} = 0.6R_m \quad (3.11)$$

$$R_{t,90} = 0.015\rho_{den} \quad (3.12)$$

$$E_{t,0} = R_m \quad (3.13)$$

$$E_{t,90} = E_m/90 \quad (3.14)$$

$$R_{c,0} = 5(R_m)^{0.45} \quad (3.15)$$

$$R_{c,90} = 0.08\rho_{den} \quad (3.16)$$

$$G_v = E_m/16 \quad (3.17)$$

$$R_v = 0.2(R_m)^{0.8} \quad (3.18)$$

Where, $R_m$ is the bending strength parallel to the grain. $E_m$ is the modulus of elasticity and $\rho_{den}$ thr density. The three are the reference material properties generated from the laboratory test data.

### 3.3.6 Moisture Adjustment Factors

It is well documented that most properties of timber vary with variation in moisture content below fibre saturation point (Ranta-Maunus, 2000). Following EN 384 (2004), Strength class values are derived from timber at about 12% moisture content, which is taken as a reference moisture content. The test results in this study indicated that the equilibrium moisture contents of the species are above 12%. This is expected, because the reference moisture content specified in the Nigerian code of practice (NCP 2, 1973) is 18%. Therefore, adjustments to results are desirable. Bending strength and density values are adjusted to 12% reference moisture content using equations (3.19) to (3.20) (Kucera, 1992; Bostron, 1994; Heikkila and Hijarvi, 2008). Bending strength is adjusted as follows:

$$f_{m,12\%} = \frac{f_{measured}}{1+0.0295(12-u)} \quad (3.19)$$

were $f_{m,12\%}$ is the bending strength at 12% moisture content. $f_{measured}$ is the bending strength at the measured moisture content. u = is the measured moisture content in %. The 12% moisture content adjustment equation for modulus of elasticity is given by

$$E_{m,12} = \frac{E_{measured}}{1+0.0143(12-u)} \quad (3.20)$$

For density, the 12% moisture adjustment factor is given as:

$$\rho_{12} = \rho_w \left[1-\frac{0.5(w-12)}{100}\right] \quad (3.21)$$

The reference material properties were also adjuted to Nigerian standard moisture content (NCP 2, 1973) of 18%, using linear interpolation.

### 3.3.7 Distributions Fitting and Tests of Goodness of Fit

In the assignment of distribution models to a random variable, two or more distribution may appear to be plausible probability distribution models. In that case, test of goodness of fit can be used to delineate the relative degree of validity of the different distribution (Ang and Tang, 1975; Engineering Statistics, 2006). Test of Goodness of Fit (GOF) may be undertaken using any of the following methods:

1. Anderson Darling test method (Stephen, 1974)
2. Chi-square test method (Snedecor and Cochran, 1989)
3. Kolmogorov Smirnov test method (Chakravart et al, 1969)

Each of the three tests is used to test if a sample data came from a population with a specific distribution.

The Anderson darling has one major disadvantage, that the critical value at a given significance level depend on the type of distribution being tested. This implied that critical values must be calculated for each distribution.

The chi-square distribution on the other hand requires a sufficient sample size in order for the chi-squre approximation to be valid.

The Kolmogorov smirnov test has the following advantages over the Anderson darling and the Chi-square tests:

1. Kolmogorov smirnov test do not depend on the specific distribution being tested.
2. Kolmogorov smirnov test is an exact test. This has overcome the disadvantage of Chi-square test, in which sufficient sample size is required for the Chi-squre approximation to be valid.

Kolmogorov Smirnov (K-S) test was adopted in this study. The test was performed on the density, modulus of elasticity and bending strength, for each of the five species, in order to access if each data set comes from a population with a specific distribution. 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 (Ang and Tang, 1975).

The hypothesis is that "The data follow a specified distribution". The hypothesis regarding the distributional form is rejected if the test statistics, is greater than the critical value. The Kolmogorov Smirnov test of goodness of fit was implemented using Easyfit software.

### 3.3.8 Probability Density Function

Probability density function (pdf) of random variable is the probability that a variate has a value x. given by:

$$F(x) = P(X = x) \tag{3.22}$$

For a continous distribution, the pdf is expressed in terms of an integral between tow points a and b:

$$\int_a^b f(x)dx = P(a \leq X \leq b) \tag{3.23}$$

The empirical pdf for a given set of data is displayed as a histogram consisting of equal-width vertical bars, each representing the number of samples data value (falling into the corresponding interval divided by the total number of data points). The theoretical pdf is displayed as a continous curve properly scaled depending on the number of intervals.

The probability density function pdf of a normally distributed random variable has a standard bell curve shape. The precice evaluation of the normality of any random variables is established using two numerical measures of shape. These measures are Skeweness and Kurtosis (Kelvin and MacGillivray, 1988). For a given data of timber material properties, $x_1, x_2, \ldots, x_n$ the formula of skeweness is:

$$\text{Skeweness} = \frac{\sum(x_i - \mu_x)^3}{(N-1)s^3} \tag{3.24}$$

Where $\mu_x$ is the mean value of the material property, s is its standard deviation, and N is the number of data points.

Kurtosis is given by equation 3.25:

$$\text{Kurtosis} = \frac{\sum(x_i - \mu_x)^4}{(N-1)s^4} \quad (3.25)$$

Skeweness tells the amount and direction of skew (departure from horizontal symmetry), and Kurtosis tells how tall and sharp the central peak is, relative to a standard bell curve of probability density function for the normal distribution. If Skeweness of a given data is positive, the data are said to be positively skewes right, meaning that, right tail of the distribution is longer than the left. If Skeweness is negative, the data are negatively skewed or skewed left, meaning that the left tail is longer. If Skeweness is 0, the data are perfectly symemetrical. Data is said to be highly skewed for Skeweness value ranging from -1 or +1. For moderately skewed data, the skewenes lies between -1 and -0.5, or +0.5 and +1. Data with with Skeweness ranging between 0 and -0.5 or 0 and +0.5 is said to be approximately symmetric.

The kurtosis of normal distribution is 3 not 0 (Engineering Statistics, 2006). Therefore kurtosis is not a suitable measure for statistical inference. What is normally used in drawing statistical inference is the Excess Kurtosis. When Kurtosis is 3, the excess kurtosis is equal to 0. A normal distribution therefore has Skeweness and Excess Kurtosis of 0. The excess kurtosis is given by equation 3.26.

$$\text{Excess kurtosis} = \frac{\sum(x_i - \mu_x)^4}{(N-1)s^4} - 3 \quad (3.26)$$

Data with Excess kurtosis of 0 is said to be mesokurtic. When the Excess kurtosis is less than 0, the data is said to be platykurtic. In this case, the central peak of the probability density plot is lower and broader, and its tails are shorter and thinner. Howecer, if the

Excess Kurtosis is greater than 0, the data is said to be leptokurtic. The central peak is higher and sharper, and its tails are longer and flatter.

### 3.3.9 Characteristic Values of Material Properties

The methods of computing bending strengths were presented in the international literature. (ISO 12491, 1997; ASTM 2916, 1994; Ranta-Maunus, 2004; EN 384, 2004; Eurocode 1-1, 2004; Eurocode 5 1-1, 2004). The characteristic values of material properties were generated in this study based on the procedure specified in EN 384 (2004).

### 3.3.10 Percentile Values of Material Properties

Given a data set that has been ordered in increasing magnitude, the median, first quartile and third quartile can be used to split the data into four pieces. The first quartile is the point at which one fourth of the the data lies below it. The median is located exactly in the middle of the data set, with half of all of the data below it. The third quartile is the place where three fouth of the data lies below it. To split the data to more than just two pieces, the idea of the quartile is generalized to that of percentile. The nth percentile of a set of data is the point where n% of the data is below it. The characteristic values of material properties reported in EN 338 (2009) are based on the 5 percentile values (EN 384, 2003). The percentile values of Density, Modulus of elasticity and Bending Strength were genereated from Easyfit statistical package.

### 3.3.11 Analysis of Variance (ANOVA) of the Reference Material Properties

Four set of independent samples of each of the five tested timber species were used for bending test. To be able to use the generated material properties, there is the need to carry out statistical test with the view to ascertain the homogeneity of the samples, to check for variability between and within samples. Two-way Analysis of variance (ANOVA) was employed in this study for the statistical test. Analysis of variance is a statistical technique whereby total variance in a set of data is partitioned into several components. Associated with each of these components is specific source of variation, so that in the analysis, it is possible to ascertain the magnitude of the contribution of each of these sources to the total variation. Variation in this context relies on a sum of squared deviation of values from their means. The total sum of squares which can be computed from the set of data is the sum of squared deviation of each observation from the mean of all the observations taken together. Analysis of varianve can therefore be used with confidence in this study, to investigate whether any significant difference exist in the material properties between and within timber species. For this analysis to be carried out, it is required that, the variance of material property of the considered samples are equal. The equality of this variance can be checked using Bartlet's statistics. In Bartlets test, a null hypothesis is tested on the equality of variance of the four sets of data for each reference material property.

### 3.3.12 The Two-way Analysis of Variance (Two-way ANOVA)

F-tests are used to compare the between- and within- timber species properties variances to determine if they are different. A hypothesis test is shown which tests the hypothesis

that the specie samples have different means. The *p*-value shows the probability of rejecting the null hypothesis, that the samples have the same mean, when it is in fact true. A significant p-value implies that at least two samples have different means.

**Table 3.13 Limiting Values for Classification into Hardwood Classes (EN 338, 2009)**

| S/No. | Strength Class | Limits of Characteristic Bending Strenght ($N/mm^2$) | Limits of Characteristic Density ($kg/m^3$) | Limits of Mean Bending Strenght ($N/mm^2$) |
|---|---|---|---|---|
| 1 | D18 | $f_{m,k} \leq 18$ | $\rho_k \leq 475$ | $E_{mean} \leq 9.5$ |
| 2 | D24 | $18 < f_{m,k} \leq 24$ | $475 < \rho_k \leq 485$ | $9.5 < E_{mean} \leq 10$ |
| 3 | D30 | $24 < f_{m,k} \leq 30$ | $485 < \rho_k \leq 640$ | $9.5 < E_{mean} \leq 10$ |
| 4 | D35 | $30 < f_{m,k} \leq 35$ | $640 < \rho_k \leq 670$ | $9.5 < E_{mean} \leq 10$ |
| 5 | D40 | $35 < f_{m,k} \leq 40$ | $670 < \rho_k \leq 700$ | $10 < E_{mean} \leq 11$ |
| 6 | D50 | $40 < f_{m,k} \leq 50$ | $700 < \rho_k \leq 780$ | $11 < E_{mean} \leq 14$ |
| 7 | D60 | $50 < f_{m,k} \leq 60$ | $780 < \rho_k \leq 840$ | $14 < E_{mean} \leq 17$ |
| 8 | D70 | $70 \leq f_{m,k}$ | $1080 \leq \rho_k$ | $20 \leq E_{mean}$ |

### 3.3.13 Allocation of Strength Classes

Strength classes are sets of material properties that can be assigned to specific combinations of timber species and strength graded. The use of strength classes is intended to simplify the specification of timber, since it groups species in pre-defined categories. Characteristic values are taken from bending strength, bending stiffness and density. The allocation to a strength class is governed by the lowest strength class

applicable to one of these properties. The classification of the species into various hardwood strength classes was made using the characteristic bending strength, mean value of density and mean values of modulus of elasticity. The limiting values of the reference material properties from EN 338 (2009) are presented in Table 3.13.

### 3.4 Methodology for the Reliability Analysis and Aalibration

The deterministic approach to structural assessment has been replaced by semi-probabilisitic method using partial safety factors (PSF) which represents individual variables affecting the reliability. Each random variable is represented by a characteristic value, which is modified by partial factors. This approach does not allow direct evaluation of probability of failure.

The partial safety factor method is used in current national and international specifications. The development of computer technology allows for considering a transition to application of fully probabilistic concepts. Such transition resulted to re-engineering of the entire design procedure, re-arrangement of standards, development of database and more. An attempt was made in this study to calibrate the material partial safety factors for timber structural design using the Eurocode 5.

The partial safety factors for materials properties in the Eurocode 5, were based on subjective judgment, derived from the long time experience of building tradition, of which their conceptions were not based on consistent tropicalisation to accommodate uncertainties. This often led to unequal reliability of structures or structural members

made of different building materials and exposed to different combination of actions (Andre and Antonio, 2010). The application of probabilistic method in the derivation of partial safety factors within the framework of code calibration is mentioned in the Eurocode 0 (2004).

This study is based on the statistical properties generated from experimental data on the five timber species considered. This is in addition to the international data on statistical properties of loading (Ellingwood and Galambos, 1980; Andre and Antonio, 2010).
A three-hinged pitched timber portal frame was considered for the investigation. The base and the ridge of the frame are pinned, resulting to a statically determinate structural system. The presented reliability analysis provides reliability verification of the frame designed according to recommendations given in the Eurocodes (Eurocode 0, 2004; Eurocode 1-1, 2004; Eurocode 1-4, 2004; Eurocode5, 2004).

The material and geometrical characteristics were monitored, and probabilistic assessments of the timber were performed. An integral part of reliability analysis is the sensitivity analysis, determining which variables, the monitored output is most sensitive to, and which is not sensitive to. Probabilistic and sensitivity studies represent important tools for the development and verification of more sophisticated but yet simple and lucid provision utilized in structural design practice.

The reliability indices as basic indicators of the level of reliability were determined using First Order Reliability Methods with Genetic Algorithms, in order to multiply solutions at

a time and simplify the application to implicit limit state functions. The implementation was made using Computer programs developed in MATLAB programming language (MATLAB, 2007). Thirty two failure modes of the portal frame were identified, and reliability functions were developed for mode of failure.

### 3.4.1 Failure Modes

One of the most important requirements of structural reliability analysis and design of structural systems is the identification of the possible modes of failure. Eurocode 5 like all other limit state design codes is based on single component performance, where each limit state is related to mode of failure of a single component (Hansson and Ellegaard, 2006). The portal frame under investigation is a structural system consisting of components (rafters, columns and connections) designed to make the final structure function as a unit. With reference to Fig. 3.3, the components considered for analysis are: Column AB, Column DE, Rafter BC, Rafter CD, Moment resisting connection at B, Moment resisting connection at D, Hold down connection at A, Hold down connection at E, and frame apex connection between sloping rafters at C. Each of these components consist of nested modes of failure.

### 3.4.2 Ultimate Limit State Modes of Failure

Related to ultimate limit state, thirty different failure modes were considered.

1. Failure due to compressive stress in member AB
2. Failure due to compressive stress in member DE
3. Failure due to compressive stress in member BC

4. Failure due to compressive stress in member CD
5. Failure due to bending stress in member AB
6. Failure due to bending stress in member DE
7. Failure due to bending stress in member BC
8. Failure due to bending stress in member CD
9. Failure due to flexural buckling of member AB
10. Failure due to flexural buckling of member DE
11. Failure due to flexural buckling of member BC
12. Failure due to flexural buckling of member CD
13. Fracture in member AB in the axis of member AB at joint B
14. Fracture in member BC in the axis of member AB at joint B
15. Fracture in member AB in the axis of member BC at joint B
16. Fracture in member BC in the axis of member BC at joint B
17. Shear stress in member AB around fastener group at joint B
18. Shear stress in member BC around fastener group at joint B
19. Fracture in member DE in the axis of member DE at joint D
20. Fracture in member CD in the axis of member DE at joint D
21. Fracture in member DE in the axis of member CD at joint D
22. Fracture in member CD in the axis of member CD at joint D
23. Shear stress in member DE around fastener group at joint D
24. Shear stress in member CD around fastener group at joint D
25. Block shear at end of member AB (joint A) due to tensile stress.
26. Block shear at end of member AB (joint A) due to shear stress.

27. Block shear at end of member DE (joint E) due to tensile stress.

28. Block shear at end of member DE (joint E) due to shear stress.

29. Failure of the frame apex connection between sloping rafters at C

30. Failure due to lateral displacement of the frame.

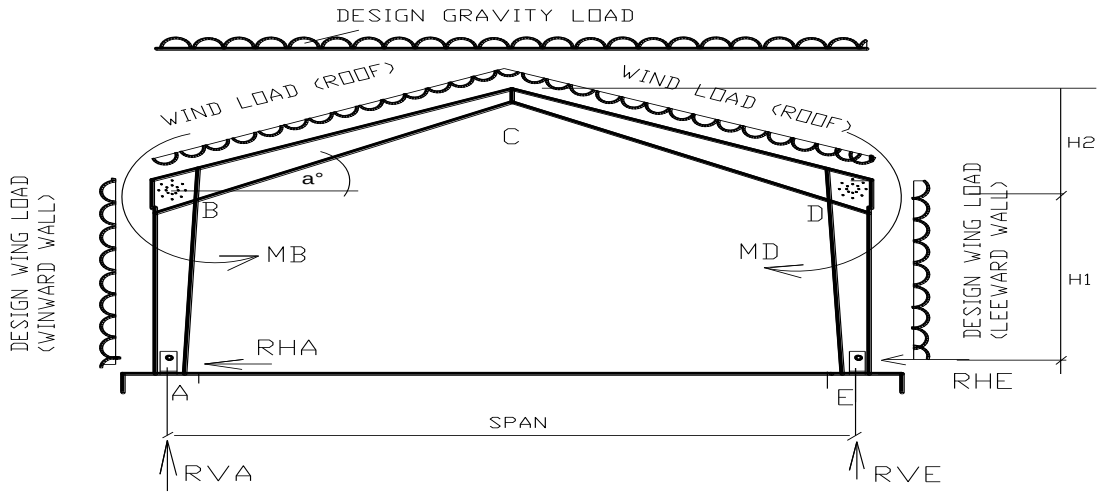



Fig. 3.3 Three pin Pitched portal frame subjected to gravity and wind loading

### 3.4.3 Service Limit State Mode of Failure

In addition to the ultimate limit state modes of failure, two failure modes for service limit states were considered, to capture beam and column deflections.

31. Failure due to deflection of frame Rafter.

32. Failure due to deflection of frame Column

The thirty two failure modes were modeled according to the resistance models of the Eurocode 5.

### 3.4.4 Frame Geometry

The portal frame specifically analysed in this study is a three pinned frame with moment resisting connection in the frame corners (Fig.3.3). The span of the frame is 12.0 m. The height of the structure is 6.0 m. The slope of the roof is approximately $19.80^0$. The maximum loading width is 3.0 m. The frame geometry was adopted from Blass *et al*, (1995). The column and rafter members were assumed to be of the same thickness of 200 mm. The maximum cross-sectional height of members was taken as the design criteria as done in most studies in reliability-based analysis and design of timber structures and components such as Afolayan (2005).

### 3.4.5 Effect of Actions and Load Combination

The frame is exposed to the self-weight of roof, varable actions. The action effects of the actions considered in the analysis consist of an axial force, N and bending moment M. In the design calculation, the axial force and bending moment were represented by the design values $N_d$ and $M_d$. The combination of action is determined considering expression (6.10b) given in the Eurocode 1990 (Eurocode 0, 2004). Considering the imposed load as the leading variable action, it follows that:

$$N_d = \xi\gamma_G(N_{frame,k}\ N_{roof,k}) + \gamma_Q(N_{imposed,k} + \psi_{0,w}N_{wind,k}) \quad (3.27)$$

$$M_d = \xi\gamma_G(M_{frame,k}\ M_{roof,k}) + \gamma_Q(M_{imposed,k} + \psi_{0,w}M_{wind,k}) \quad (3.28)$$

Where $\xi = 0.85$ is the reduction factor for permanent actions, $\gamma_G = 1.35$ is the partial factor for permanent actions, $\gamma_Q = 1.5$ is the partial factor for variable actions, and $\psi_{0,w} = 0.6$ is the factor for the combination value of the wind action. These parameters were

based on the Eurocodes. $N_{wind,k}$ is the characteristic value of the axial force due to the wind action. Following the recommendations provided in the Eurocode 1 part 4 (Eurocode 1-4, 2004), the characteristic value of the wind pressure $w_k$ is given as:

$$W_k = c_p q_p(z) \quad (3.29)$$

Where $c_p$ is the pressure coefficient dependent on the building geometry and for the size of the loaded area (in this study, the loaded area is assumed to be longer than $10m^2$). It describes the outside pressure and suction combined with either the inside suction or the outside overpressure. In this case, more unfavourable effects are caused by a combination of outside pressure and inside suction. The peak velocity pressure can be written as:

$$q_p(z) = c_g(z)c_r(z)^2\frac{1}{2}\rho v_b^2 \quad (3.30)$$

where $c_g(z)$ is the gust factor corresponding to the height z, of the structure and terrain category II ( open terrain with isolated obstacles) (Eurocode 1-4, 2004). $C_r(z)$ is the roughness factor, also for the terrain category II. The air density $\rho$ is taken as $1.26kg/m^3$. The design values of the bending moments were derived from the same assumptions as for the design values of the axial forces.

### 3.4.6 Structural Analysis

The structure analysed in this study has been modeled as a three pinned portal frame. This idealization results into a statically determinate structure of which analysis was made by considering its force and moment equilibrium. Bending moment is zero at the supports and the ridge, and critical at the rafter-column joints. Both the rafter and the

column were designed to resist axial forces, bending moment and flexural buckling. The structural analysis procedure is as follows:

1. Determine the main sizes and critical values for the relevant loads and climatic conditions.
2. Sketch the approximate centre lines of the frame based on the following experience-based figures:
    a. Cross-sectional depth at haunch $h_h \approx \frac{h}{15} + \frac{L}{30}$, where h is the column height, L is the span of the portal frame.
    b. Cross-sectional depth at support, $h_s = 0.7h_h$
    c. Cross-sectional depth at ridge, $h_r = 0.3h_h$
3. Calculate the support reactions and internal forces at the foot and ridge for various combinations of loading.
4. Determine the bending moment and the internal forces at the hauch for various combinations of loading.

### 3.4.7 The Eurocode 5 Design Values

In the Eurocodes, material strength is represented by a characteristic value defined as the 5% fractile value. The structural stiffness parameters are represented by a mean value, except in the case of instability. The design value of a material property is expressed as:

$$X_d = \eta \frac{X_k}{\gamma_m} \tag{3.31}$$

Where $X_k$ is the characteristic value of the material property, $\eta$ is the mean value of the conversion factor taking into account the effect of the duration of the load, volume, and

scale effects, effects of moisture and temperature and any other relevant parameter. $\gamma_m$ is the partial safety factor of the material property which take into account of the possibility of unfavorable deviations of material properties from their characteristic values.

The design resistance for material property is expressed in the following form provided that resistance is a linear function of material strength.

$$R_d = \frac{1}{\gamma_{Rd}} R\{X_{d,I}; a_d\} = R\{\eta_i \frac{X_{ki}}{\gamma_{Mi}}; a_d\} \quad (3.32)$$

Where $\gamma_{Mi} = \gamma_{Rd} \times \gamma_{mi}$, the random part of $\eta_i$ is included in $\gamma_{mi}$ is a partial safety factor covering uncertainty in the resistance model, plus geometric deviations if these are not modeled explicitly.

Typical format for the verification of a structural timber component in EC5 is given as a design equation in the following form:

$$k_{mod} z_d \frac{r_k}{\gamma_m} - \sum \psi_i \gamma_{s,i} S_{i,k} = 0 \quad (3.33)$$

where $r_k$ is the characteristic value for the resistance. $z_d$ is a vector of design variables (e.g. the cross section of the timber beam). $S_{i,k}$ are the characteristic values of load effects, which are considered in the design. $\gamma_m$ and $\gamma_s$ are partial safety factors for the resistance and loads respectively. $K_{mod}$ is the modification factor taking into account the effect of duration of load and moisture content

### 3.4.8 Limit State Functions for the Various Failure Modes

A traditional notion of the safety limit is associated with the ultimate limit states. For example, a beam fails if the moment due to loads exceeds the moment carrying capacity

(Nowak, 2004). The limit state functions for the portal frame were developed for each of the thirty two failure modes. In addition, the uncertainty model coefficients are used to accommodate the inaccuracy of the resistance model for each failure mode.

The design axial compression resistance of a timber member according to Eurocode 5 is given by:

$$f_{c,0,d} = \frac{(k_{mod} f_{c,0,k})}{\gamma_m} \tag{3.34}$$

The limit state function for compression is as follows:

$$G(x) = \frac{(k_{mod} f_{c,0,k})}{\gamma_m} - \sigma_{c,d} \tag{3.35}$$

where $k_{mod}$ is a modification factor to take care of variation in density and moisture content. $f_{c,0,k}$ is the characteristic compressive strength parallel to grain. $\gamma_m$ is the timber material partial safety factor for strength, $\sigma_d$ is the design compressive stress.

The design bending resistance of a timber member according to Eurocode 5 is given by:

$$f_{m,d} = \frac{(k_{mod} f_{m,k})}{\gamma_m} \tag{3.36}$$

The following is the limit state function for bending is as followins:

$$G(x) = \frac{(k_{mod} f_{m,k})}{\gamma_m} - \sigma_{m.d} \tag{3.37}$$

$f_{m,k}$ is the characteristic bending strength. $\sigma_{m.d}$, is the design bending strength.

Stability is often a sticky point in structural design, and timber structures are no exception. In the real world, instability results in extensive displacements which in turn produce high bending stresses that will cause material failure (Bell and Eggen, 2001).

When a slender column is loaded in pure compression, there is tendency for it to deflect sideways. This type of instability is called "flexural buckling". The Eurocode 5, design criteria for flexural buckling included bending in its section about column, and thus has no provision for buckling due to pure compression. Therefore, flexural buckling in this presentation referred to buckling due to both compression and bending. The design buckling resistance of a timber member according to the EC5 is given by:

$$\frac{\sigma_{md}}{f_{md}} + \frac{(\sigma_{c,0,d})}{(k_c f_{m,d})} \leq 1 \qquad (3.38)$$

The limit state function for buckling is as follows:

$$G(x) = 1 - \left( \frac{\sigma_{md}}{f_{md}} + \frac{(\sigma_{c,0,d})}{(k_c f_{m,d})} \right) \qquad (3.39)$$

Where $k_c = \frac{1}{(k+\sqrt{(k^2-\lambda_{rel}^2)})}$, $k = 0.5[1+\beta_c(\lambda_{rel}-0.3)+\lambda_{rel}^2]$, $\lambda_{rel} = \sqrt{\frac{(f_{c.0,k})}{(\sigma_{c,crit})}}$, $\sigma_{c,crit} = \pi^2 \frac{E_{0.05}}{\lambda^2}$ $\beta_c$ is a constant ($\beta_c$ = 2.0 for solid timber). The slenderness is a function of the member effective buckling length. This is the distance between two consecutive points of contra-flexure of the member (Blass *et a,l* 1995).

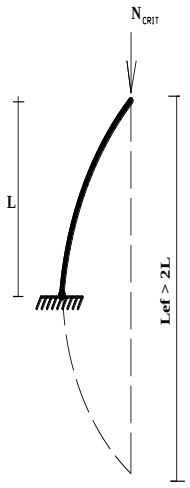



Fig 3.4. Effective Length of a Clamped Column

The columns for the frame in this study, is a clamped column (Fig. 3.4). Development of completely rigid connection between the column and the rafter is impossible. The connections are usually assumed to be semi-rigid. The rotations in semi-rigid joints are taken into account in the computation of buckling length. The rotational stiffness $K_r$ of a semi rigid connection is defined as the moment necessary to cause an angle rotation of one radian. With the slip modulus $K_u$ of the fastener, the rotational stiffness $K_u$ of a semi-rigid connection is calculated as

$$K_r = \sum_{i=1}^{n} K_u r_i^2 \qquad (3.40)$$

Where $r_i$ denotes the distance between the single fastener and the centre of the connection.

The slip modulus $K_u$ is given by:

$$K_u = \frac{2K_{ser}}{3} \qquad (3.41)$$

Where $K_{ser}$, the joint slip according to Eurocode 5 is given by:

$$K_{srr} = \frac{\rho_k^{1.5} d}{23} \tag{3.42}$$

d is the bolt diameter. $\rho_k$ is the density of the timber member.

With reference to Fig. 3.5, Equations 3.43 and 3.44 give the buckling length of the frame column and rafter respectively.

$$L_{ef,c} = h\sqrt{(4 + 3.2\frac{Is}{T_0 h} + 10\frac{Is}{T_0 h})} \tag{3.43}$$

$$L_{ef,r} = h\sqrt{(4 + 3.2\frac{Is}{T_0 h} + 10\frac{Is}{T_0 h})}\sqrt{\frac{I_0 N}{I N_0}} \tag{3.44}$$

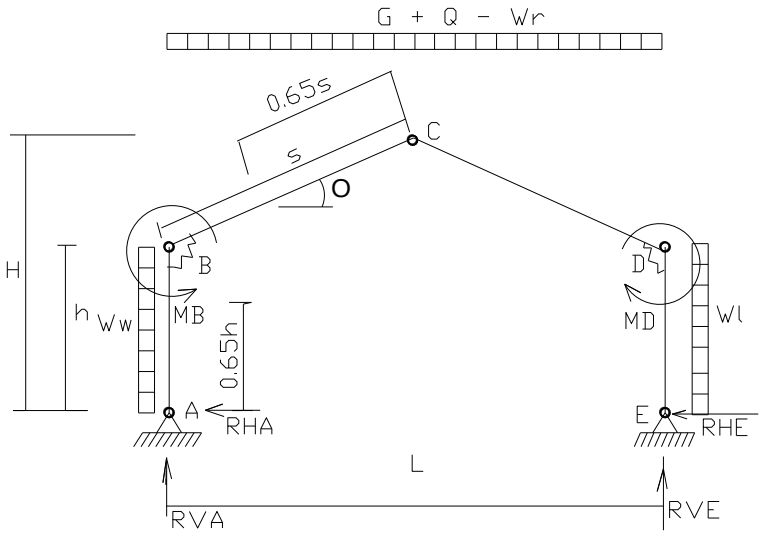



Fig 3.5. Three-hinged Frame with Semi-rigid Frame Corners

Where, N and $N_0$ denote the axial forces in the column and the rafter respectively. I is the second moment of area of the column. $I_0$ is the second moment of area of the rafter. S is

the length of the rafter. H is the height of the column. E is the five percentile member elastic modulus. $K_r$ is the beam-column joint rotational stiffness.

In the literature, often the reliability analysis of portal frames is carried out by assuming that the connections between the rafters and the columns are either fully rigid or ideally pinned (Wang, and Ghosn, (2005). The fully rigid connection assumption implied that full slope continuity exists between the adjoining members and that the full gravity moment is transferred from the beams to the columns. On the other hand, the ideally pinned connection assumption implies that there is no restraint for rotation between the beams and the columns and the columns carry no gravity moment transferred from the beams. These assumptions simplify the structural analysis but may not mimic the observed structural behaviour. It has been indicated that stiffness of the current practical connections falls between that of fully rigid and that of ideally pinned. In other words, the connections are semi-rigid connections (Chen et al. 1996).

In this study, bolted moment resisting connection at joints B and D (Fig. 3.5) are adopted. The connection was designed with double leg members. Fig. 3.6 shows the free body diagram of the moment resisting connection. The bolts are placed with regard to the Eurocode 5 regulations on minimum distances between centres. The carrying capacity is calculated on the assumption that all part of the joint have the same stiffness. The bolter rafter-column connection forces are established as follows (Lokaj, 2008):

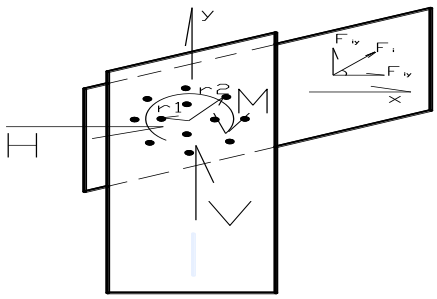



Fig. 3.6 Forces in the Frame Corner Moment resisting

Radius of the bolt circles of the connection:

$$r_1 \leq 0.5h - 4d \tag{3.45}$$

$$r_2 \leq r_1 - 5d \tag{3.46}$$

Number of bolts in a circle:

$$n_1 \leq \frac{2\pi r_1}{6d} \tag{3.47}$$

$$n_2 \leq \frac{2\pi r_2}{6d} \tag{3.48}$$

Load on bolt in column of the frame due to shear and normal forces:

$$F_{V,C} = \frac{(V_{d,c})}{(n_1 + n_2)} \tag{3.49}$$

$$F_{N,C} = \frac{(N_{d,c})}{(n_1 + n_2)} \tag{3.50}$$

Load on bolts in column and rafter of the franme due to bending moment:

$$F_M = M_d \frac{(r_1)}{(n_1 r_1^2 + n_2 r_2^2)} \tag{3.51}$$

Load on bolts in rafter of the frame due to shear and normal forces:

$$F_{V,R} = \frac{(V_{d,r})}{(n_1 + n_2)} \tag{3.52}$$

$$F_{N,R} = \frac{(N_{d,r})}{(n_1 + n_2)} \quad (3.53)$$

Total load of bolts in the axis of the rafter and column of the frame:

$$F_{d,c} \sqrt{[(F_M + F_{V,C})^2 + F_{N,C}]} \quad (3.54)$$

$$F_{d,\backslash r} \sqrt{[(F_M + F_{V,R})^2 + F_{N,R}]} \quad (3.55)$$

Shear force in column and rafter in connection:

$$V_M = \frac{M_d}{\pi} \frac{(n_1 r_1 + n_2 r_2)}{(n_1 r_1^2 + n_2 r_2^2)} \quad (3.56)$$

$$F_{V,d,C} = V_M - \frac{V_{dc}}{2} \quad (3.57)$$

$$F_{V,d,R} = V_M - \frac{V_{dr}}{2} \quad (3.58)$$

It has been established in tests (Racher, 1995) that shear stresses around the fastener are often critical to the load carrying capacity. The shear stresses in frame column and rafter axis are:

$$\tau_{V,C} = \frac{3F_{VdC}}{2bh} \quad (3.59)$$

$$\tau_{V,R} = \frac{3F_{VdR}}{2bh} \quad (3.60)$$

Load carrying capacity for brittle failure of the connection between load and timber fibre in column:

Column axis:

$$F_{V,Rk,CC} = f_{h,c,k}\, t_c\, d \quad (3.61)$$

Rafter axis:

$$F_{V,Rk,RC} = f_{h,r,k}\, t_r\, d \quad (3.62)$$

Where, the characteristic embedding strength of bolt in column are:

$$f_{h,c,k} = \frac{(f_{h,0,k})}{(k_{90}\sin^2\alpha_1 + \cos^2\alpha_1)} \quad (3.63)$$

$$f_{h,r,k} = \frac{(f_{h,0,k})}{(k_{90}\sin^2\alpha_2 + \cos^2\alpha_2)} \quad (3.64)$$

$$\alpha_1 = \arctan\frac{(F_M + F_{VC})}{F_{NC}} \quad (3.65)$$

$$\alpha_2 = \alpha_1 - \left(\frac{\pi}{2} - \alpha_1\right) \quad (3.66)$$

$$f_{h,0,k} = 0.082(1-0.01d)\rho_k \quad (3.67)$$

$$k_{90} = 1.35 + 0.015d \quad (3.68)$$

Load carrying capacity for brittle failure of the connection between load and timber fibre in rafter:

Column axis:

$$F_{V,Rk,CR} = f_{h,c,k}\, t_c\, d \quad (3.69)$$

Rafter axis:

$$F_{V,Rk,RR} = f_{h,r,k}\, t_r\, d \quad (3.70)$$

Where, the characteristic embedding strength of bolt in column are:

$$f_{h,c,k} = \frac{(f_{h,0,k})}{(k_{90}\sin^2\alpha_1 + \cos^2\alpha_1)} \quad (3.71)$$

$$f_{h,r,k} = \frac{(f_{h,0,k})}{(k_{90}\sin^2\alpha_2 + \cos^2\alpha_2)} \quad (3.72)$$

$$\alpha_1 = \text{arctg}\frac{(F_M + F_{VR})}{F_{NR}} \tag{3.73}$$

$$\alpha_2 = \frac{\pi}{2} + \alpha - \alpha_1 \tag{3.74}$$

Characteristic shear strength of the frame members is given by:

$$f_{vd} = \frac{k_{mod} f_{vk}}{\gamma_m} \tag{3.75}$$

The limit state functions for the rafter-column connections failure modes are as follows:

1. Brittle embedment failure in column member along its axis:

$$G(x) = F_{V,d,C} - n\frac{(k_{mod} F_{v,Rk,CC})}{\gamma_m} \tag{3.76}$$

2. Brittle embedment failure in column member along rafter axis:

$$G(x) = F_{V,d,C} - n\frac{(k_{mod} F_{v,Rk,CR})}{\gamma_m} \tag{3.77}$$

3. Brittle embedment failure in rafter member along column axis:

$$G(x) = F_{V,d,R} - n\frac{(k_{mod} F_{v,Rk,RC})}{\gamma_m} \tag{3.78}$$

4. Brittle embedment failure in rafter member along its axis:

$$G(x) = F_{V,d,R} - n\frac{(k_{mod} F_{v,Rk,RR})}{\gamma_m} \tag{3.79}$$

5. Shear failure in column member;

$$G(x) = F_{V,d,C} - \frac{(k_{mod} f_{vk})}{\gamma_m} \quad (3.80)$$

6. Shear failure in rafter member:

$$G(x) = F_{V,d,R} - n\frac{(k_{mod} F_{vk})}{\gamma_m} \quad (3.81)$$

Where, $n = n_1 + n_2$

At the base of the portal frame, the connection between column and foundation are assumed pinned connected using steel angle and bolts. Connections of this type near the end of the timber member with load parallel to grain are at risk of a brittle failure type due to block and plug shear. This type of failure is referred to in Eurocode 5 8.2.3 (5) and in Annex A (informative).

The Block shear capacity (Fig. 3.7) of the joint is given by: (plug shear failure), is given by:

$$F_{bsRkt} = 1.5A_{net,t} f_{t,0,k} \quad (3.82)$$

The plug shear capacity (Fig. 4.6) is given by:

$$F_{bsRktv} = 0.7A_{net,v} f_{v,k} \quad (3.83)$$

With, $A_{net,t}\, t = L_{net,t} t_1$ (3.84)

$$A_{net,t}\, v = L_{net,v} t_1 \quad (3.85)$$

and $L_{net,t} = \sum l_{v,i}$ (3.86)

$$L_{net,v} = \sum l_{v,I} \quad (3.87)$$

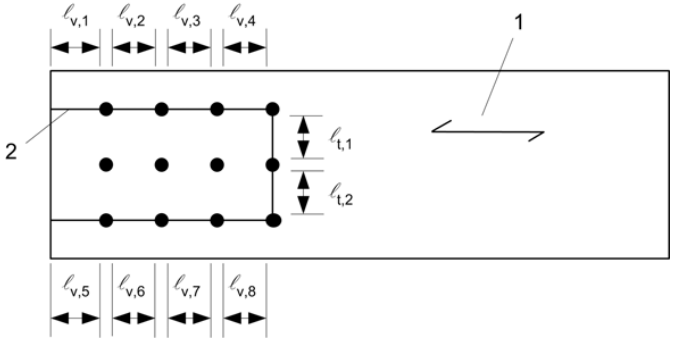



Fig. 3.7: Block shear failure (Eurocode 5, 2004)

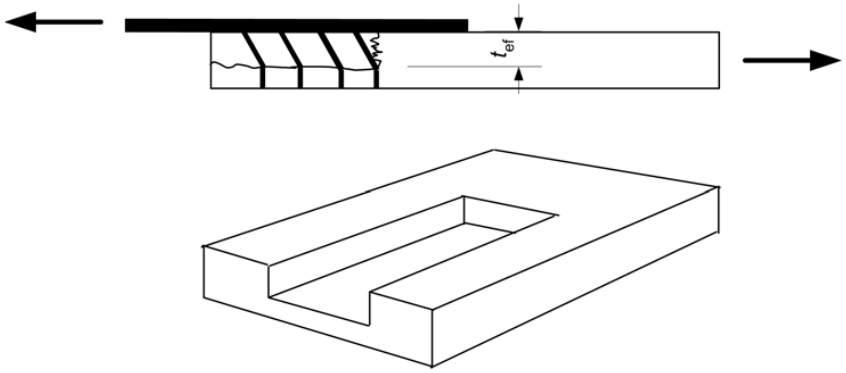



Fig. 3.8 Plug shear failure (Eurocode 5, 2004)

Where, $F_{bs,Rk}$ is the characteristic block shear or plug shear capacity. $A_{net,t}$ is the net cross-sectional area perpendicular to the grain. $A_{met,v}$ is the net shear area in the parallel to grain direction. $L_{net,t}$ is the net width of the cross-section perpendicular to the grain. $L_{net,v}$ is the total net length of the shear fracture area. $L_{t,I}$ and $l_{v,i}$ are defined in figure A.1. $t_1$ is the timber member thickness or penetration depth of the fastener.

The limit state functions for block and plug shear are as follows:

$$G(X) = R_v - \frac{(1.5k_{mod}A_{net}f_{t,0,k})}{\gamma_m} \qquad (3.88)$$

$$G(X) = R_v - \frac{(1.05k_{mod}A_{net}f_{v,k})}{\gamma_m} \quad (3.89)$$

Where $R_v$ is the thrust in column at column base.

The limit state function for frame apex connection is given by equation 3.90.

$$G(x) = R_C - n\frac{(K_{mod}F_{Rk},t_r)}{\gamma_m} \quad (3.90)$$

Where, $F_{Rk}$ is the load carrying capacity of a sigle bolt at the frame apec joint, connecting sloping rafters. $R_c$ is the axial force in rafter at the frame apec. $T_r$ is the rafter thickness, n is the number of bolts at the frame apex.

The reliability of the frame members can be updated by using additional information. One such information is on the member stiffness, E, as it can be the result of mid-span deflection of the frame members, U, subjected to given uniformly distributed loading. According to Eurocode 5, the total member deflection can be written as (Eurocode 5, 2004; Toratti, 2004):

$$U = \frac{(5G_kL^4)}{384E_{mean}I}(1+k_{def})+\frac{(5Q_kL^4)}{384E_{mean}I}(1+\psi_2k_{def})+\frac{(5W_kL^4)}{384E_{mean}I}(1+\psi_2k_{def}) \quad (3.91)$$

Where, $G_k$ is the characteristic dead load. $Q_k$ is the characteristic imposed load. $W_k$ is the characteristic imposed load. $E_{mean}$ is the mean elastic modulus corresponding to the given strength class. I is the member second moment of area (moment of inertia). L is the member span. $k_{def}$ is the creep factor (Eorocode 5) corresponding to the given material type and service class. $\psi_2$ is the modification factor for creep due to variable actions.

The limit state function for column and rafter deflections are given by equations (3.92) and (3.93) respectively, as follows

$$G(x) = \delta_L - \frac{(5W_kL^4)}{384E_{mean}I}(1 + \psi_2 k_{def}) \quad (3.92)$$

$$G(x) = \delta_L - \frac{(5G_kL^4)}{384E_{mean}I}(1+k_{def}) + \frac{(5Q_kL^4)}{384E_{mean}I}(1 + \psi_2 k_{def}) \quad (3.93)$$

Where, $\delta_L$ is the deflection limit (Eurocode 5, 2004).

One of the main objectives of wood frame structures, is its continuous existence after an event, such as wind storm. Verification that the structure performance are met requires a mapping between the objective stated qualitatively and a response quantity and limit state (force or deformation) that can be checked using the principles of structural analysis and mechanics. The mapping invariably requires that the behavior of the structural system be considered as a whole, and present one of the major challenges for performance-based engineering strategies (Rosowsky and Ellingwood, 2002). The percentage drift of wood frame structures under lateral load is often the design criteria, and range from 1.0 to 2% (ven de Lindt and Rosowsky, 2005). In this study a limiting drift of 1% was adopted. This requires that, the lateral displacement at column height must not exceed 0.01 times the column height. Under wind lateral, portal frame is highly dependent on the following (Pedro and Sameh, 2008):

1. Rafter to column stiffness ratio.
2. Rafter span-to-column height ratio
3. Column end support

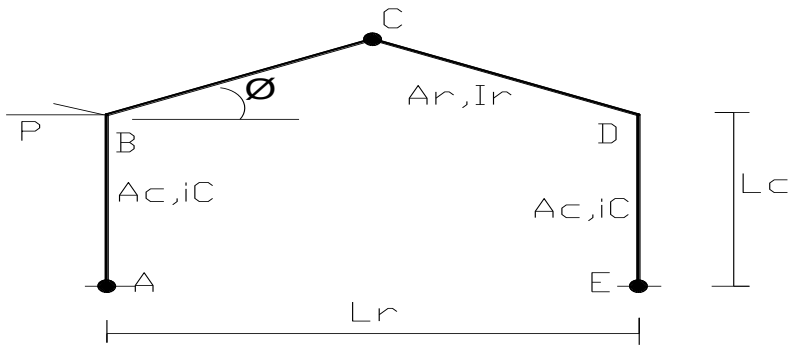



Figure 3.9 : Three hinge portal frame under lateral load P.

For a portal frame with pinned support (Fig. 3.9), the lateral deflection is given by:

$$\Delta_f = (4 + \frac{2k}{\alpha})\frac{L_c^3}{(24ET_c)} \leq \delta L_c \qquad (3.94)$$

Where, $\alpha = \frac{I_{rafter}}{I_c}$ = rafter to column moment of inertia ratio. $\delta = \frac{\Delta_f}{L_c}$ = drift ratio (ratio of lateral deflection to column deflection. $k = \frac{L_{rafter}}{L_c}$ = rafter span to column height ratio; The limit state function for the sway mode is given by equation 6.69.

$$G(x) = \delta L_c - (4 + \frac{2k}{\alpha})\frac{L_c^3}{(24ET_c)} \qquad (3.95)$$

### 3.4.9 Evaluation of the Limit State Functions

During the last two decades much efforts has been directed towards reliable and efficient methods for reliability analysis of structures. Among these methods, first order and second order reliability (FORM and SORM) methods are the earliest and most widely used. FORM and SORM are based on first-order or second-order approximations of the

limit-state at the design point. Extensive reviews of these methods are found in (Rackwitz, 2000). These methods require the evaluation of the derivatives of limit state functions with respect to the random variables. When these functions are explicit functions of the random variables, it is easy to compute the derivatives of these functions.

In many cases of practical importance, particularly for complicated structures, the limit-state functions are usually implicit in terms of the random variables. Therefore, derivatives of the limit-state functions are not easy to derive. Moreover, a significant disadvantage of these methods is that they may lead to erroneous results when the limit-state function has multiple local minimal distance points (Cheng, 2007).

In this study, a different route for structural reliability employing genetic algorithm (GA) is adopted. The genetic algorithms calculate the probability of failure by using the genetic search technique based on a natural selection process and following a search path until failure is reached (Cheng, 2007; Rackwitz, 2000; Wang and Ghosn, 2005). Compared with FORM/SORM, the genetic algorithm has advantage that it does not involve the difficulties of computing the derivatives of limit state functions with respect to the random variables and has the capability of identifying global optimum values of the limit state functions. A genetic algorithm based reliability problem can be formulated in the following form:

Minimise $\beta = \|\mu\|^2 = \mu^T.\mu$ (3.96)

Subject to: $g(\mu) = 0$

Where μ is the vector of standard normal variates; g(μ) is the limit state function; β is the reliability index. The problem in Eqn.(3.96) is a constrained nonlinear optimization problem. The genetic operation is continous until the following stopping criteria is achieved.

1. The average reliability index of the current generation does not show significant improvement over the former generation.

   $\gamma\beta^{(k+1)generation} > \gamma\beta^{kgeneration}$

   γ can be set to 0.95 (Wang and Ghosn, 2005)

2. The first three different minimum reliability indices of the current generation remain the same as those of the previous generation.

### 3.4.10 Setup of Reliability Analysis by Genetic Algorithm

Considering the independent standard normal U-space where failure is defined by a number of failure modes, $g_1$,.$g_2$, , , , ,$g_k$. These modes are reached by following the failure paths in direction $\boldsymbol{I}_i$. In the genetic algorithm (GA) according to Shao and Murotsu (1999), $3^n - 1$ discrete search directions are used in n-dimensional space. The search direction vectors are expressed by a three-digit system where each of the digits belong to the binary set (-1,0,1) as illustrated in Fig 3.10.

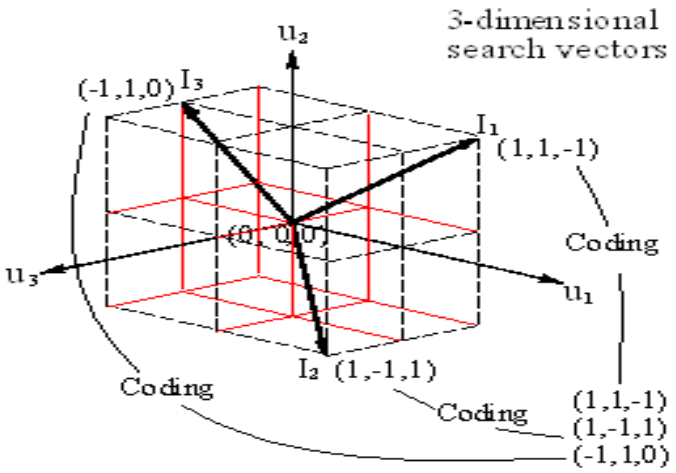



Fig. 3.10: Coding of search direction vectors

The rectangular coordinates of a direction vector constitute a string of digits known as chromosomes in GA parlance. GA simulates the natual selection process based on the survival of the fittest (in this case, which direction should survive and which direction should be killed off). In the GA-based reliability analysis, a fitness is defined as the inverse of the reliability index, β, where β is the distance from the orogin of the U-space to the failure surface as shown in Fig. 3.11.

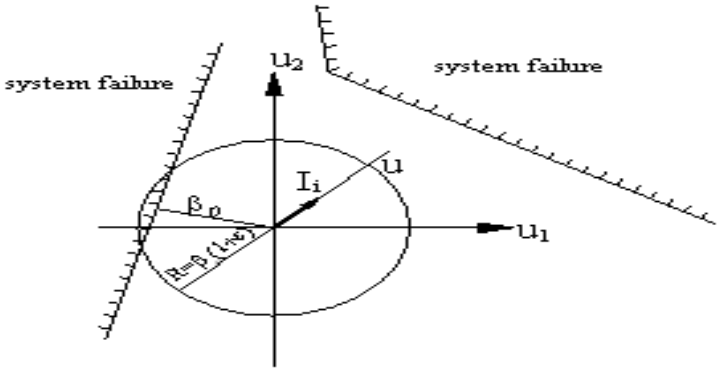



Fig. 3.11 Illustration of GA search strategy

To initiate the search process, GA randomly generates a set of chromosomes to form a first generation of directions. The search is refined by creating a "new generation´of chromosomes using three operators: Selection, Crossover and Mutation. The objective is to improve the fitness of the population of chromosomes with each generation.

Selection simulates the evolution process that allows the chromosomes with best fitness´to survive and mate. The chromosomes in a new generation are the offsprings (or children) of selected chromosomes of the previous generation. The selection is done randomely such that chromosomes with high fitness have the higher probabilities to contribute offsprings in a new generation.

Crossover simulates the natural mating phenomenom whereby offsprings in a new generation are obtained by crossing the chromosomes of two "parents" directions. Each offspring will keep some of the genes from each of its two parents as shown in Fig. 3.12

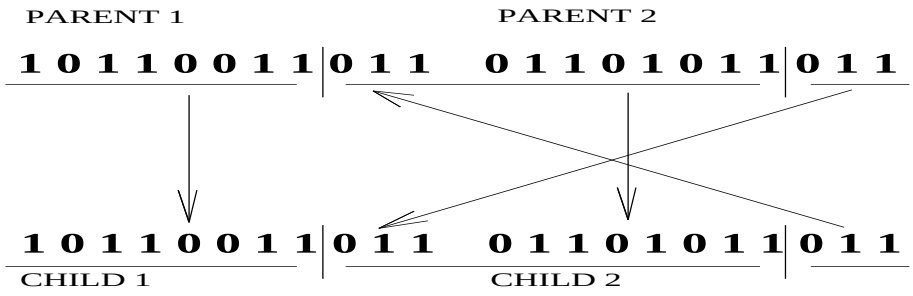



Fig. 3.12 Illustration of crossover operation

Mutation is secondary operator that simulate an inheritance that goes astray. It is an explorative operator that allows the search to wander into regions of the solution space which may never have been reached from the crossover operation. The purpose of the mutation operator is to reduce "in-breeding" that may misdirect the search process. This, new "genetic material" is occationally introduced into the new generation by randomly changing digits in each chromosome. For example , a 1 is randomly changed to 0, or vice versa. This is done only with a very low probability to generate new genetic material without loosing track of the important directions that the search has produced so far. Considering a typical chromosome that is formed by a total of n genes (or variables). For example (1 0 1 1 0 0 1 1 1 0 …….) is one representation of such a chromosome. The m genes associated with 1 in the sequence are called active variables. In this example, the active variables are located in positions 1,3,4,7,8,9,……. The other genes are not active.

The reliabity index associated with the failure modes described by the chromosome is β. An approximation to the probability of failure $P_f$ can be obtained by assuming that the safety marging M (or failure function) is linear in the space defined by the n variables ($X_1$, $X_2$, . . .,$X_n$). This linear equation can be expressed as:

$$M = \frac{1}{\sqrt{m}} (X_1 + X_3 + X_4 + X_7 + X_8 + X_9 + \ldots) + \beta \qquad (3.97)$$

Only the active variables appear in eqn. (3.97), because the non-active variables are known constants (deterministic) parameters that do not affect the safety index.

For a given search direction, on the U-space, the failure point (design point) is designated by **A** and its coordinates are expressed as :

$$\mathbf{OA} = \beta(k_1\mathbf{x}_1, k_2\mathbf{x}_2, \ldots\ldots\ldots, k_n\mathbf{x}_n) \tag{3.98}$$

Where $k_1, k_2, \ldots\ldots, k_n$ are **OA**'s unit vector coordinates in direction $x_1, x_2 \ldots x_n$ suck that:

$$(k_1^2 + k_2^2 + \ldots\ldots + k_n^2)^{1/2} = 1 \tag{3.99}$$

For the coding adopted in this work, as outlined in Fig. 4.8, the k's can be represented as:

$$(k_1, k_2, \ldots\ldots k_n) = (\boldsymbol{I}_1\boldsymbol{I}_2 \ldots\ldots \boldsymbol{I}_n)\frac{1}{\sqrt{m}} \tag{3.100}$$

Where $\boldsymbol{I}_i = 1$ or 0. A value of 1 indicates an increase in the direction of variable $x_i$. 0 indicate that $x_i$ remains at its mean value (origin). The index i = 1,2, . . . . .n, identifies the variable. m gives the total number of non-zero terms in the set $(k_1, k_2, \ldots, k_n)$. This would lead to the alternate representation of OA as:

$$\mathrm{OA} = \frac{\beta}{\sqrt{m}} = (\boldsymbol{I}_1 x_1\ \boldsymbol{I}_2 x_2 \ldots\ldots \boldsymbol{I}_n x_n) \tag{3.101}$$

Each set of $\boldsymbol{I}_i$ values defines a chromosome that represents the direction along the n-dimensional space $x_1, x_2, \ldots\ldots x_n$. The n variables $X_n$ in turn represents the sequence of random variables $X_1, X_2, \ldots . X_n$ in tha order in the normalized U-space. These random variables are denoted $l_1, l_2, \ldots\ldots, l_n$. An approximate number of chromosomes define the

population of each generation. If too few chromosomes are generated, GA has only few possibility to execute the crossover operation. On the other hand, if there are too many chromosomes, GA slows down. Research has shown that increasing the population size beyond a certain limit will not help improve convergence (Deng 2005). Following the approach of Cornell (1996), it is suggested that an optimum population size would be on the order of:

$$N_{popsize} = orde(r\frac{L}{2k}\,3^{k}) \tag{3.102}$$

Where $n_{popsize}$ is the optimum population size, L is the length of the chromosome, and k is the average number of bits per parameter. The value that each active variable takes when failure occur is given by:

$$L_{active} = \beta\frac{i}{\sqrt{m}} = \frac{(x_{active} - \mu_{active})}{\sigma_{active}} \tag{3.103}$$

Where m is the number of non-zero (active) genes, $\mu_{active}$ is the mean value of the acive variable, $\sigma_{active}$ is the standard deviation of the active variable, $\beta$ is the safety index.

### 3.4.11 System Reliability Analysis

Reliability models applied to timber were always related to individual components but not the system. As any real structure is a complex system, system behaviour must be of particular interest. All components in structural systems, must satisfy the basic code requirement, that is. the load effect cannot exceed the resistance. The component-based design is highly conservative because of redundancy. When the load in a critical

component approaches the ultimate value, other components can take additional loads and prevent a failure. However, the quantification of this load sharing requires a special approach using the system reliability models. A considerable effort was directed on the development of the reliability analysis procedures for structural components. The formulation of the limit state function for a structural system is much more difficult than for a component

In the study, thirty two failure modes were considered. Consequently, the thirty one value of Hasofer-Lind reliability index corresponding to thirty one modes of failure were calculated. The system probability of failure of structural systems is calculated from (Melchers, 1999; Afolayan, 2005; Detlevsen and Madsen, 2005):

$$P_{sys} = P(E_i U ... U E_n)$$

$$= \sum_{i=1}^{n} E_i - \sum_{j>i=1}^{n} (E_i \cap E_j) + \sum_{k>j}^{n} (E_i \cap E_j \cap E_k) + (-1)^{n-1} P(E_i \cap \ldots \cap E_n) \quad (3.104)$$

Where $E_i$, I = 1,2,…,n represent the violation of the established failure modes for the component under consideration.

### 3.4.12 Statistical Models of Material Properties of Timber

The strength of structural members depends not only on the strength of the material but also on the stiffness, in the case of timber members (e.g. under bending stress), mainly on bending stiffness (Blass et al, 1995). The Eurocode 5 in common with other Eurocodes provides no data on strength and stiffness properties for structural materials. It merely

states the rules appropriate to the determination of these values to achieve compatibility with the safety format and the design rules of code (Glos, 1995).

Experimental data shows that all important characteristic strength and stiffness properties can be calculated from either bending strength, modulus of elasticity (E) or density (Glos, 1995). These are known as the reference material properties. Other properties, such as tensile and compressive strength parallel and perpendicular to grain, shear strength, and shear modulus are derived from the reference material properties (EN 384, 2003; JCSS, 2006).

In Europe a strength class system that groups all timber species with similar properties together are used. The strength classes for solid timber are given in EN 338 (2008). There are six strength classes for hardwood (D30, D35, D40, D50, D60 and D70). D stands for deciduous. The figure in front of the letter D represents the characteristic bending strength of the material in that class. In this research work, the statistics of the material properties of timber were established based on the standard test according to EN 408 (2002), on five common timber species (*Alstonia boonei, Triplochiton schleroxylon, Terminalia ivorensis Terminalia superb and Lophira alata*) from Nigerian markets. The data generated were analysed in accordance with EN 384 (2003).

A rubost statistical analysis package (Easyfit, 2010) was used to establish the appropriate statistical distribution model. The strength test data were fitted well with both Normal, Lognormal, and Gumbel. However, it was suggested that log-normal distribution be used

for all timber materials in structural reliability analysis, because it is widely used for other materials and because it seems to be the best for timber materials used for long span structures as well (Ranta-Maunus, and Torati, 2002). The probabilistic models of the material properties are presented in Table 3.14.

Table 3.14: Statistics of Reference Material Properties of Timber

| Variable | Mean | Coefficient of Variation | Distribution Model |
|---|---|---|---|
| Bending strength ($N/mm^2$) | $f_m$ | 0.12 to 0.19 | Lognormal |
| Bending Modulus of Elasticity ($N/mm^2$) | $E_m$ | 0.06 to 0.27 | Lognormal |
| Density ($kg/m^3$) | $\rho_{den}$ | 0.04 to 0.19 | Lognormal |

The statistics of the derived material properties were generated based on the recommendation of JCSS probabilistic model code (Kohler and Faber, 2004; JCSS, 2006). This recommendation is presented in Table 3.15.

### 3.4.13 Statistics of Loading Parameters

Actual data on dead and live load uncertainties is not available for the Nigerian reality. In this investigation it is assumed that no significant difference exist in dead and live load between Nigeria and what is reported in international references. Dead and imposed load statistics used in this research are based on data reported by Ellingwood et al (1980) Bartlett et al, (2003); Ranta-Maunus (2004) Allen, (1975) and McGregor (1976). The wind load was generated in accordance with the Eurocode 1-4 (2004).

Table 3.15 Statistical Derived Material Properties of Timber

| **Variable** | **Mean** | **Coefficient of Variation** | **Distribution Model** |
|---|---|---|---|
| Tensile strength parallel to grain | $f_{t,0} = 0.6f_m$ | $1.2COVf_m$ | Lognormal |
| Tensile strength perpendicular to grain | $f_{t,90} = 0.015\rho_{den}$ | $2.5COV\rho_{den}$ | Lognormal |
| Compressive strength parallel to grain | $f_{c,0} = 5f_m^{0.45}$ | $0.8COVf_m$ | Lognormal |
| Compressive strength perpendicular to grain | $f_{c,90} = 0.008\rho_{den}$ | $COV\rho_{den}$ | Lognormal |
| Shear strength | $f_v = 0.2f_m^{0.8}$ | $COVf_m$ | Lognormal |

(Source: JCSS, 2006)

### 3.4.14 Statistics of other Paramerts

The model uncertainties cover deviations and simplifications related to the probabilistic modeling and the limit state equations (JCSS, 2006). The reference properties are determined by standardized tests. Geometrical deviations from specified dimensions, duration of load and moisture effects (damage accumulation) also contribute to model uncertainties if not explicitly accounted for in the probabilistic modeling. Furthermore, the idealized and simplified limit state equations introduce model uncertainties. Geometrical data were generally random variables. In comparison with actions and material properties their variability can in most cases be considered small or negligible (Gulvanessian et al, 2002). In this study, geometrical parameters of the frame

components as well as frame dimensions were considered as deterministic. Any error due to this presumption is assumed to have been taken into account by the model uncertainties. Partial safety factors for both load and column resistance were also considered deterministic. Load duration factor that account for the variability in member properties due to load duration and variation in moisture contents as well as the model uncertainties for load and resistance models are random variables.

Table 3.16 Statistics of Load Variables

| Variable | Mean | Coefficient of Variation | Distribution Model |
|---|---|---|---|
| Dead load, $G_k$ | $1.05G_k$ | 0.10 | Normal |
| Imposed load, $Q_k$ | $1.00Q_k$ | 0.25 | Gumbel |
| Wind load, $W_k$ | $0.90W_k$ | 0.34 | Gumbel |
| Load duration factor, $k_{mod}$ | $1.0k_{mod}$ | 0.15 | Lognormal |
| Model uncertainty (load), $\Theta_S$ | $1.0\Theta_S$ | 0.1 | Lognormal |
| Model uncertainty (strength), $\Theta_R$ | $1.0\Theta_R$ | 0.1 | Lognormal |

(Source: Ellingwood *et al*, 1980; Bartlett *et al*, 2003; Ranta-Maunus, 2004; Andre and Antonio, 2010)

### 3.4.15 Reliability-based Calibration

The Eurocodes and most other traditional codes are based on the deterministic methods of the partial safety factor design. Researchers and designers have been doubting the adequacy of some backgrounds, design assumptions and methods presented in such deterministic codes (Ditlevsen, 2005). Sometimes it is difficult to perceive how it is possible to assess the reliability parameters of structures subjected to complicated gravity and lateral actions by using some ordinary partial actors of fixed values. Practically, the

reliability degree of structures designed by deterministic and probabilistic methods can be markedly different. However, real degree of structural reliability may be defined only by full probability-based concepts.

One of the most important applications of the structural reliability methods is the development of the new generation design codes. The partial safety factors are calculated so that the reliability of structures is at the predetermined target level (Nowak and Szerszen, 2000). Several attempts were made to calibrate the partial safety factors in the new generation codes (Andre and Antonio, 2010). In the present research work, partial safety factors for material properties were calibrated based on the Eurocode 5 resistance models relevant to three hinge timber portal frames. The entire process was implemented using Genetic Algorithms (GA) based First Order Second Moment (FOSM) formulation.

### 3.4.16 Methods of Code Calibration

There are two ways of calibrating new generation code (Tord, 2001; Faber and Sorensen, 2002) as shown in Fig. 3.12. The method of standardized FORM sensitivity coefficients and the minimum error method. The overall calibration process compromises out of two steps: one step to find the target reliability and a second step to find the set of safety elements that more or less guarantee that a preecribed reliability level is attained. According to the two step procedure presented in Fig 3.13 should first establish a target reliability. The point is that, in an economic optimization, the target depends on the consequences of failure as well as on the relative cost to increase safety. Target reliabilities are established by looking at the existing code, in this case, the Eurocodes.

The idea is that the application of a new approach should not lead to large differences from the existing code.

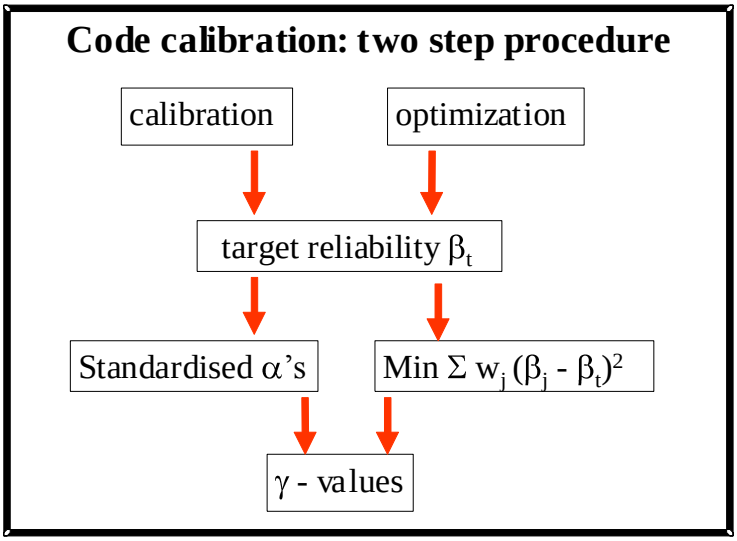



Fig. 3.13 Two step procedure for code calibration

### 3.4.17 Method of Standardised FORM Sensitivity Coefficients

In code calibration based on first order reliability methods (FORM) it is assumed that the limit state function can be written as follows

$$G(\mathbf{x},\mathbf{p},\mathbf{z}) = 0 \qquad (3.105)$$

Where $\mathbf{x} = (x_1, \ldots., x_n)$ is a realization of $\mathbf{X} = (X_1, \ldots.,X_n)$ modeling n stochastic variables describing the uncertain quantities. External loads, strength parameters and model uncertainty variables are example of uncertain quantities $\mathbf{p} = (\mathbf{p_1}.. \ldots. \mathbf{p}_n)$ are M deterministic parameters, for example well defined geometrical quantites, $\mathbf{z} = (z_1 \ldots., z_n)$ are N design variables which are used to design the actual structure. Realisation $\mathbf{x}$ of $\mathbf{X}$ where $g(\mathbf{x},\mathbf{p},\mathbf{z}) \leq 0$ corresponds to failure states, while $g(\mathbf{x},\mathbf{p},\mathbf{z}) > 0$ corresponds to safe states.

Using FORM (First Order Reliability Method), the reliability index β is determined. The corresponding estimate of the probability of failure is

$$P_f = \Phi(-\beta) \quad (3.106)$$

where $\Phi(-\beta)$ is the standard normal distribution function.

If the partial safety factors and if the number of design variables is N = 1, then the design (modeled by z) can be determined from the design equation

$$G(\mathbf{x_c},\mathbf{p},\mathbf{z},\gamma) \geq 0 \quad (3.107)$$

$\mathbf{x_c} = (x_{c1}, \ldots .,x_{cn})$ are characteristic values corresponding to the stochastic variable X. $\gamma = (\gamma_1 \ldots .. ,\gamma_n)$ are m partial safety factors. The partial safety factors γ are usually defined such that $\gamma_i \geq 1$, l = l, . . . .,m. The design equation is closely connected to the limit state function ( ). In most cases the only difference is that the state variables **x** are exchanged by design value $\mathbf{x}^d$ obtained from the characteristic value $\mathbf{x_c}$ and the partial safety factor**, γ.**

A reliability analysis by FORM with the limit state function( ) gives the reliability index β and the β-point $\mathbf{x}^*$. Partial safety factors can then be obtainedfrom

$$\gamma_i = \frac{x_{ci}}{x_i^*} \text{ for strength variables} \quad (3.108)$$

$$\gamma_i = \frac{x_i^*}{x_{ci}} \text{ for load variables} \quad (3.109)$$

In accordance with the method of standardized FORM sensitivity coefficients, the design and characteristic values in equations (3.110) and (3.111) are given by

$$x_i^* = \mu(1-\alpha\beta V)$$

$$x_{ci} = \mu(1-kV)$$

where α is the FORM sensitivity coefficient (negative for loading variable and positive for resistance variable). μ is the mean, V is the coefficient of variation. B is the safety index, k is a value defining the characteristic value. For load, k is usually equal to 0 or another small value (Faber and Sorensen, 2002)' In the case of arbitrary distribution F(…), the value of $x_i^*$ follow from $F(x_i^*) = \Phi(-\alpha\beta)$. The standardized values of α are given by α = +0.80 for strength variable; α = -0.70 for dominant load; α = -0.28 for non-dominant load

### 3.4.18 Code Calibration Using Minimum Error

The second approach of code calibration is shown on the right hand branche in Figure (3.12). The safety elements are determined in such a way that the target reliability or probability is best achieved. The general procedure of code calibration using minmum error is presented as follows.

Step 1: Domain of duality: Definition of the class of structures (the mechanical element, the failure model(s) and the working area of the code) for which the code is aimed. This step includes also the definition of mechanical and probabilistic models.

Step 2: Code objective: Definition of the code objective by an expected target reliability level within the structure class defined in the first step. The code target can be defined as a target probability of failure $p_f$ or a target reliability index $\beta = -\Phi^{-1}(p_f)$. Due to several difficulties to assess this reliability level and according to most authors, the reliability index is assess using classical FORM/SORM approximations. The target reliability is noted $\beta_T$.

Step 3: Selection of design situation: Selection of L specific design situations in the structure class and evaluation of their relative frequencies $\omega_j > 0$ ($\sum_{j=1}^{L} \omega_j = 1$).

Step 4: Measure of target fitting: Choice of a penalty function $W(\omega_j,\beta_j(\gamma_i),\beta_t)$ punishing the deviation of the code from the objective, where $\beta_j$ is the reliability index of the jth specific design situation depending on the partial safety factor $\gamma_i$ used for the design.

Step 5: Optimisation of the partial factors: Determination of the "best" code format, that is the closest format to the code objective. It is performed by solving the following optimization variable.

$$\text{Min } f(\gamma_i) = \sum_{j=1}^{L} W(\omega_j,\beta_j(\gamma_i),\beta_t) \qquad (3.111)$$

### 3.5 Setup of the Numerical Experiments

The First order reliability analysis and safety fctors calibration were implemented using genetic algorithms. For implantation, rubost computer programs were developed using MATLAB 2011. The program flow chart for the genetic algorithms is in Fig. 3.14.

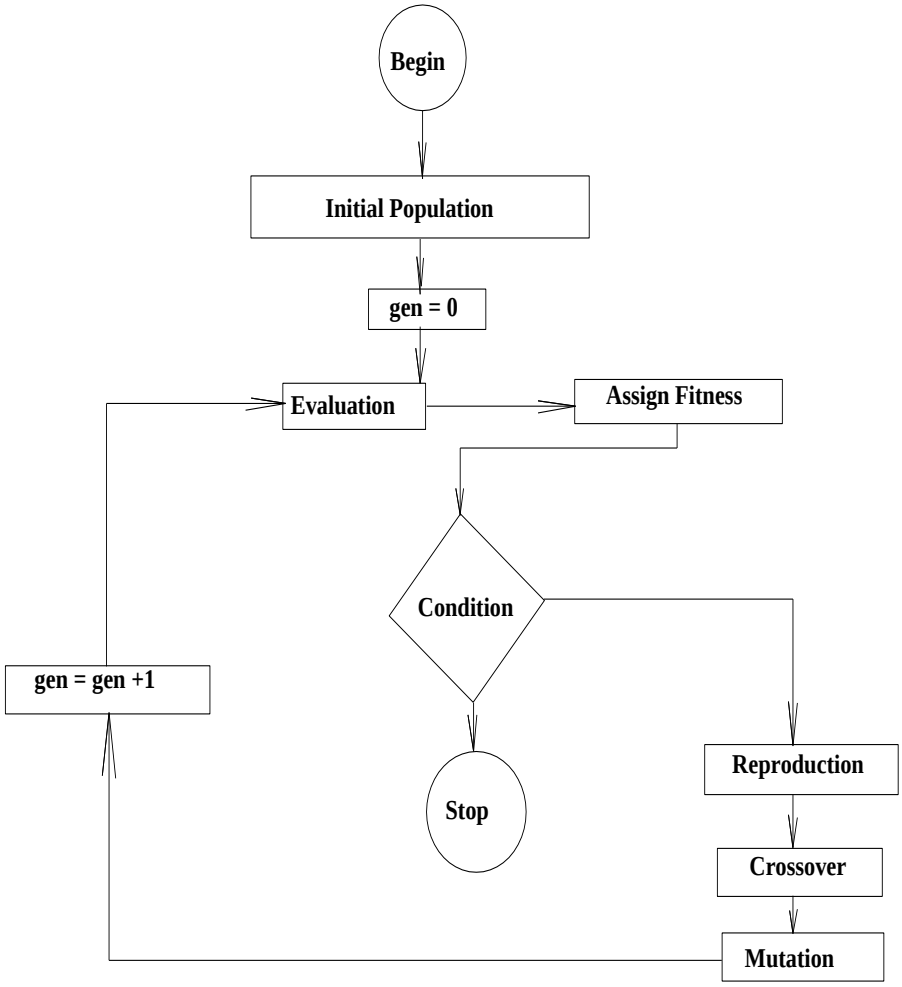



Fig. 3.14 Flowchart of Working Principle of a Genetic Algorithm (GA)

The programs were setup in eleven directories (folders) as follows:

1. Main directory
2. Form directory
3. Distribution model setup directory
4. Coefficient of variation directory

5. Distribution transformation directory
6. Probability of failure directory
7. System reliability directory
8. Wind load directory
9. Material properties directory
10. Genetic algorithm operations directory
11. Input Definition directory

### 3.5.1 Main Directory

The main directory contain the main program 'genehunter.m'. This program was developed to perform the reliability analysis and calibration required in this study. While running this program, the user would be required to select among thirteen options, each option having the capability of execution of a specific function. The detail of this section program is given in Appendix D.

### 3.5.2 Form directory

The form directory contains the subroutines that calculate the safety index for each failure mode. Thirty two subroutines were duly created, thirty for the ultimate and stability limit states criteria, two for the limit state of deflection. The conditional limit state of fire is a nested limit state designed to affect all the thirty two explicit modes of failure.

### 3.5.3 Distribution Model Setup Directory

The ‘distribution model setup directory’ contains the MATLAB function files that assign appropriate distribution model for each random variable. Only five distribution models were considered in this study. These are, the normal distribution, lognormal distribution, gumbel distribution, weibull distribution and frechet distribution

### 3.5.4 Distribution transformation directory

The distribution model transformation directory contains the function files that transform gumbel and lognormal variables to equivalent normally distributed random variables.

### 3.5.5 Coefficient of variation directory

The coefficient of variation directory containing the function file that store the values of the coefficients of variation of each random varible  based of test results and data from literature.

### 3.5.6 Probability of failure directory

Probability of failure direcrory contain the MATLAB function that compute probability of failure with safety index as an input.

### 3.5.7 System reliability directory

The **'system reliability analysis routine'** house the function that perform the system reliability analys, with the safety indices all the phenomenal mode are considered as inputs. This program is reproduced in appendix D.

### 3.5.8 Wind load directory

The Wind load directory containg the MATLAB function 'winload.'. This sub-program perform wind load Generation on the frame considering a given wind load sceneria. The generation of the wind load is in accordance with the Eurocod 1, part 1-4.

### 3.5.9 Material properties directory

The design of timber structures using Eurocode 5 requires information on the material properties of solid timber from EN 338 (2008). A subroutine was created in this study to store and provide the relevant property when required. The subroutine in denoted 'function material property.

### 3.5.10 Genetic algorithm operations directory

The Genetic Algorithm operation requires some dedicated processes. These processes include:

1. Initialisation
2. Crossover
3. Mutation and tournament selection.
4. Tournament Selection

# CHAPTER FOUR

# RESULTS AND DISCUSSION

## 4.1 Experimental Results and Discussion

### 4.1.1 Moisture Content Test Results and Discussion

The moisture content test results are presented in Table 4.1. The mean values of the moisture contents for the *Alstonia boonei* (Ahun), *Triplochiton Scleroxylon* (Obeche), *Terminalia Ivorensis* (Black Afara), *Terminalia superba* (White Afara) *and Lophira Alata* (Ekki) are 20.59%, 16.83%, 19.26% 20.96% 15.3% respectively, with the corresponding coefficients of variation of 34%, 29%, 24% 23% and 22%.

Table 4.1 Moisture content test results

| Timber Specie | Moisture Content (%) | | |
|---|---|---|---|
| | Mean | Standard Deviation | Coefficient of Variation |
| *Alstonia Boonei* (Ahun) | 20.59 | 7.04 | 0.34 |
| *Triplochiton Acleroxylon* (Obeche) | 16.83 | 4.85 | 0.29 |
| *Terminalia Ivorensis* (Black Afara) | 19.26 | 4.65 | 0.24 |
| *Terminalia Superba* (White Afara) | 20.96 | 4.81 | 0.23 |
| *Lophira Alata* (Ekki) | 15.30 | 1.30 | 0.22 |

### 4.1.2 Density Test Results and Discusion

The density test results are presented in Tables 4.2. The mean values of the density of the *Alstonia boonei* (Ahun), *Triplochiton Scleroxylon* (Obeche), *Terminalia Ivorensis* (Black

Afara), *Terminalia superba* (White Afara) *and Lophira Alata* (Ekki) are 360.76 kg/m$^3$, 380.25 kg/m$^3$, 472.10 kg/m$^3$, 533.89 kg/m$^3$ and 955.93 kg/m$^3$ respectively. Also, the corresponding coefficiens of variation are 4%, 16%, 18%, 19% and 4% respectively. This result implied little variability in the density of *Alstonia boonei* (Ahun) and *Lophira Alata* (Ekki) compared with that of the other three specie.

Table 4.2. Density test results

| Timber Specie | Sample Size | Mean value (kg/m$^3$) | Standard Deviation | Coefficient of Variation |
|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | 40 | 360.76 | 13.38 | 0.04 |
| *Triplochiton Scleroxylon* (Obeche) | 40 | 380.25 | 61.36 | 0.16 |
| *Terminalia Ivorensis* (Black Afara) | 40 | 472.10 | 86.66 | 0.18 |
| *Terminalia Superba* (White Afara) | 40 | 533.89 | 103.73 | 0.19 |
| *Lophira Alata* (Ekki) | 40 | 955.93 | 39.21 | 0.04 |

### 4.1.3 Three-point bending Test Results and Discussion

Fourty beams of each specie were subjected to three-point bending test. The length of each piece is 750 mm. The cross section of each piece is 50mm x 75mm. Each specimen was tested on the Avery Denison Universal Testiong Machine at the Heavy Weight Laboratory, of the Department of Civil Engineering, Ahmadu Bello University, Zaria. The specimens were tested in accordance with ASTM D 143 (2006). The central

deflection of each test piece was measured relative to the end supports by means of a dial gauge graduated at 0.01 of a millimeter. The measurements were made at interval of 1000N up to the failure load. Load-deflection scatter plot were made for each test piece for loads from 1000N to a value falling around the proportional limits, in order to keep the plot within the elastic range of deformation. The moduli of elasticity were obtained from the least square lines of the load-deflection plots developed for each specimen. The corresponding bending strengths were obtained using the maximum load at failure. The statistic of the Modulus of Elasticity and the Bending Strength are presented in Table 4.3 and 4.4 respectively. The Moduli of Elasticity (MOE) values of 7309.60N/mm$^2$, 6928.90N/mm$^2$, 11740.00N/mm$^2$, 13243.00N/mm$^2$ and 18004.00 N/mm$^2$ were obtained for the *Alstonia boonei* (Ahun), *Triplochiton Scleroxylon* (Obeche), *Terminalia Ivorensis* (Black Afara), *Terminalia superba* (White Afara) *and Lophira Alata* (Ekki) respectively. The corresponding coefficients of variation are 9%, 19%, 12%, 19% and 13% respectively.

Table 4.3. Modulus of Elasticity Test Results (Three-point bending)

| Timber Specie | Sample Size | Mean value (N/mm$^2$) | Standard Deviation | Coefficient of Variation |
|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | 40 | 7309.60 | 628.47 | 0.09 |
| *Triplochiton Scleroxylon* (Obeche) | 40 | 6928.90 | 1290.30 | 0.19 |
| *Terminalia Ivorensis* (Black Afara) | 40 | 11740.00 | 1393.60 | 0.12 |
| *Terminalia Superba* (White Afara) | 40 | 13243.00 | 2558.40 | 0.19 |
| *Lophira Alata* (Ekki) | 40 | 18004.00 | 2396.80 | 0.13 |

The respective values of bending strength fot the *Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia superba and Lophira alata* are 28.63N/mm$^2$, 38.72N/mm$^2$, 49.04N/mm$^2$, 60.71N/mm$^2$, 69.73N/mm$^2$. The corresponding coefficients of variation are 18%, 9%, 21%, 16% and 18% respectively

Table 4.4. Bending Strength Test Results (Three-point bending test)

| Timber Specie | Sample Size | Mean value (N/mm$^2$) | Standard Deviation | Coefficient of Variation |
|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | 40 | 28.63 | 5.10 | 0.18 |
| *Triplochiton Scleroxylon* (Obeche) | 40 | 38.72 | 3.58 | 0.09 |
| *Terminalia Ivorensis* (Black Afara) | 40 | 49.04 | 10.30 | 0.21 |
| *Terminalia Superba* (White Afara) | 40 | 60.71 | 9.68 | 0.16 |
| *Lophira Alata* (Ekki) | 40 | 69.73 | 12.50 | 0.18 |

### 4.1.4 Modulus of Elasticity and Bending Strength from the Four-point bending Test

Fouty beams of each specie were also subjected to Four Point bending test method according to the provision of EN 408 (2003). The test was performed on the Avery Denison Universal Testing Machine, with a specicially fabricated two-point central load transfer accessory. The MOE was calculated from the load-deflection plots. The computed values of the Moduli of Elasticity are 8192.00N/mm$^2$, 6137.80N/mm$^2$, 12161.00N/mm$^2$, 11614.00N/mm$^2$ and 22750.00N/mm$^2$ obtained for the *Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia Superba and Lophirav Alata*

respectively. The corresponding coefficients of variation are 6%, 27%, 22%, 21% and 12% respectively.

4.5 Modulus of Elasticity Test Results (Four-point bending Test)

| Timber Specie | Sample Size | Mean value ($N/mm^2$) | Standard Deviation | Coefficient of variation |
|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | 40 | 8192.00 | 461.09 | 0.06 |
| *Triplochiton Scleroxylon* (Obeche) | 40 | 6137.80 | 1648.10 | 0.27 |
| *Terminalia Ivorensis* (Black Afara) | 40 | 12161.00 | 2645.00 | 0.22 |
| *Terminalia Superba* (White Afara) | 40 | 11614.00 | 2449.20 | 0.21 |
| *Lophira Alata* (Ekki) | 40 | 22750.00 | 2744.90 | 0.12 |

The bending strength value fot the *Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia superba and Lophira alata* are $43.09N/mm^2$, $54.07N/mm^2$, $70.49N/mm^2$, $83.63N/mm^2$, and $97.79N/mm^2$ respectively. The corresponding coefficients of variation are 12%, 16%, 19%, 19% and 16% respectively.

**Elasticity**

Based on the results obtained from the three-point and the four-point bending tests, mathematical transfer models were developed using linear regression analysis on Microsoft excel (2007). The developed transfer models are meant to facilitate convertion of three-point bending strength (BS) and modulus of elasticity (MOE) to four point bending strength (BS) and modulus of elasticity (MOE). The Least square regression

plots for the bending strengths are presented in Figs 4.7 to 4.8. Also, the least square regression plots for the Modulus of Elasticity are presented in Figs 4.6 to 4.10.

Table 4.6. Bending Strength Test Results (Four-point Bending Test)

| Timber Specie | Sample Size | Mean value ($N/mm^2$) | Standard Deviation | Coefficient of Variation |
|---|---|---|---|---|
| *Alstonia Boonei* | 40 | 43.09 | 5.03 | 0.12 |
| *Triplochiton Scleroxylon* | 40 | 54.07 | 8.65 | 0.16 |
| *Terminalia Ivorensis* | 40 | 70.49 | 13.72 | 0.19 |
| *Terminalia Superba* | 40 | 83.63 | 15.73 | 0.19 |
| *Lophira Alata* | 40 | 97.79 | 15.45 | 0.16 |

**4.1.5 Mathematical Transfer Models for Bending Strength and Modulus of**

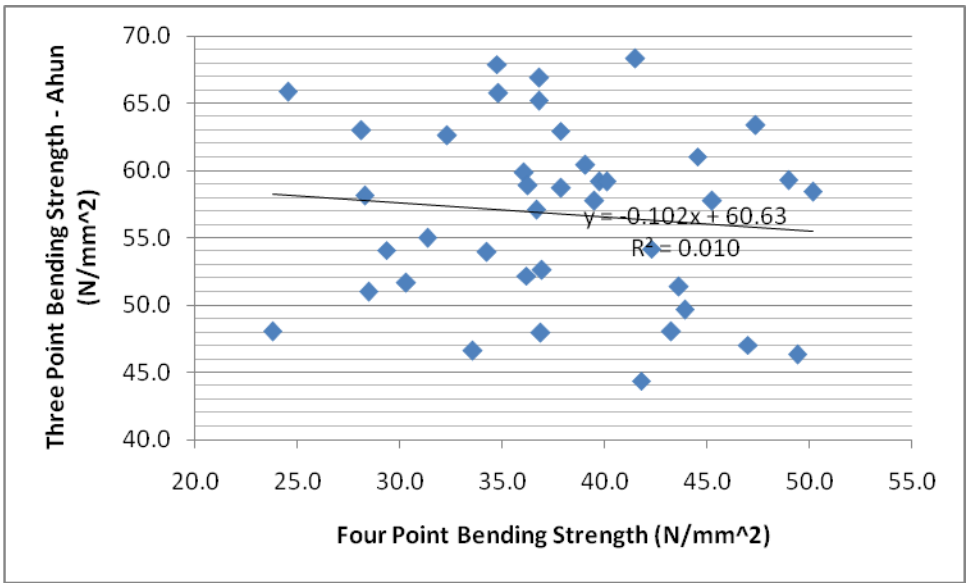



Fig. 4.1 Least Square Regression Plots for Bending Strength of *Alstonia boonei*

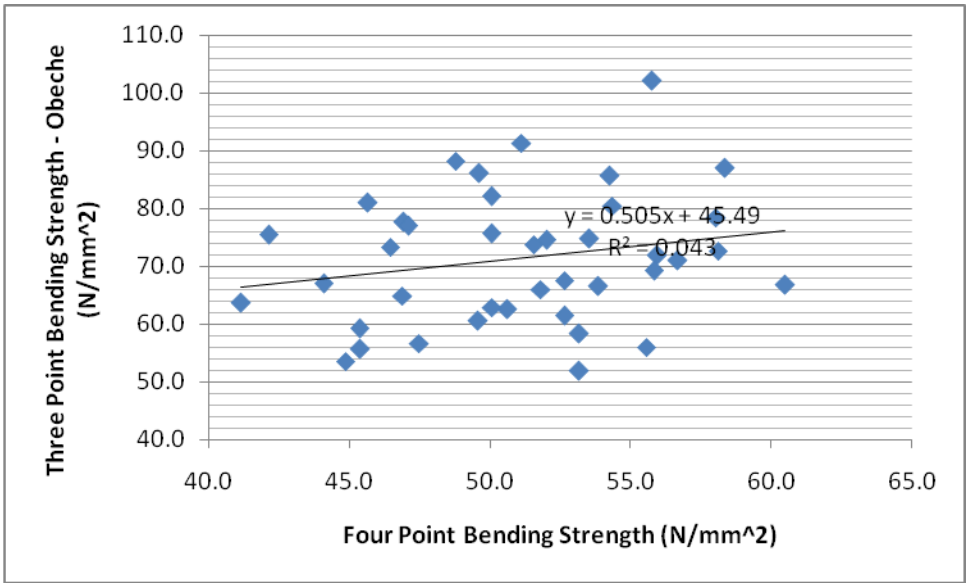



Fig. 4.2 Least Square Regression Plots for Bending Strength of *Triplochiton scleroxylon*

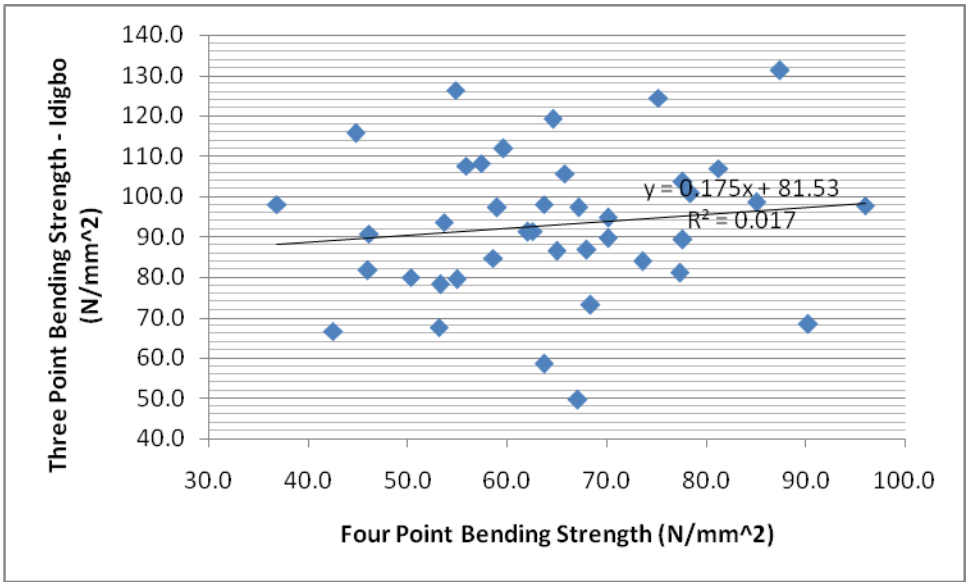



Fig. 4.3 Least Square Regression Plots for Bending Strength of *Terminalia ivorenis*

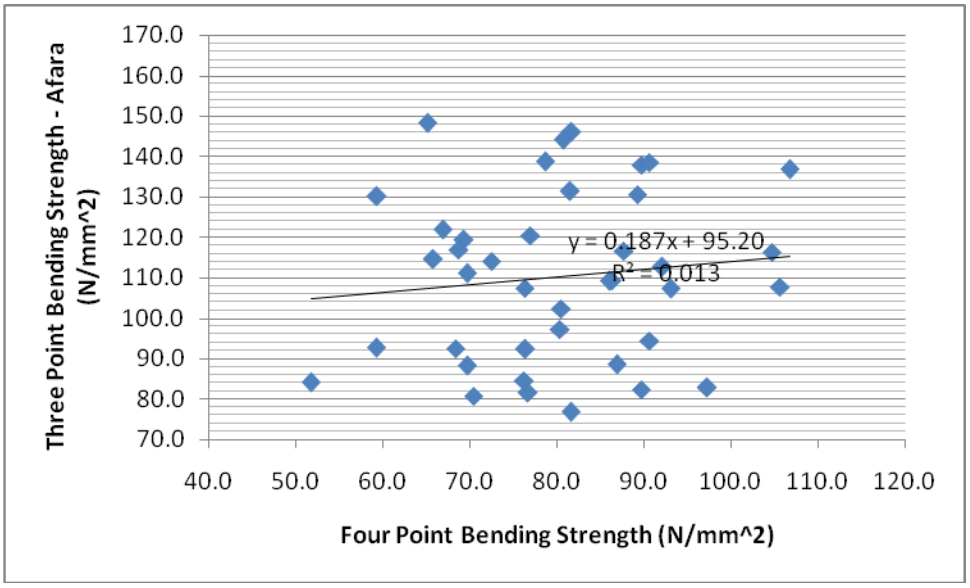



Fig. 4.4 Least Square Regression Plots for Bending Strength of *Terminalia superba*

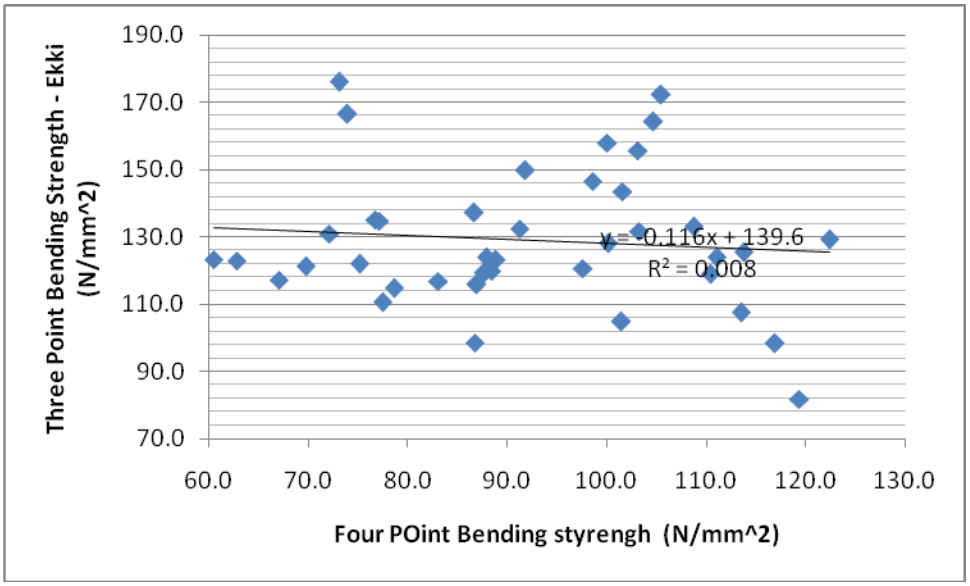



Fig. 4.5 Least Square Regression Plots for Bending Strength of *Lophira alata*

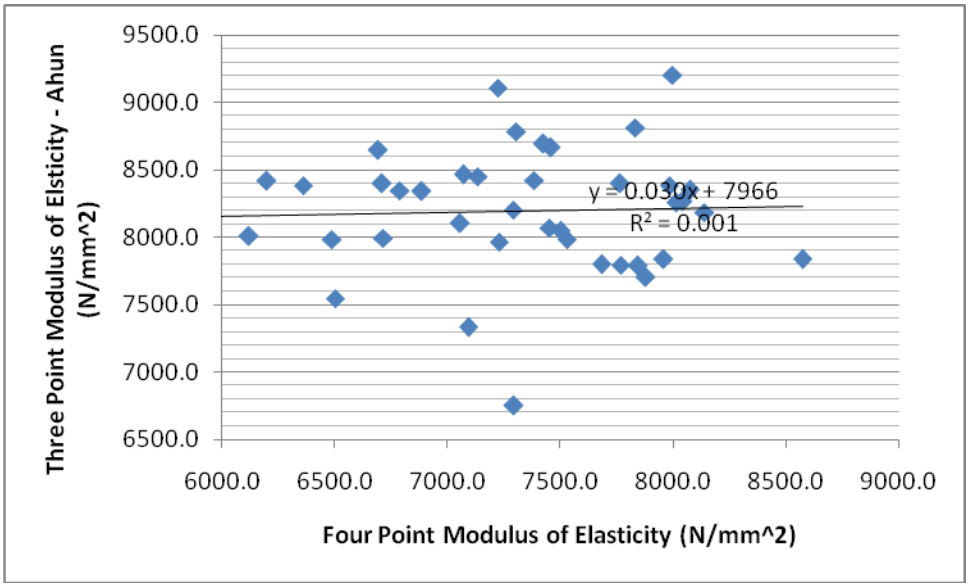



Fig. 4.6 Least Square Regression Plots for Modulus of Elasticity of *Alstonia boonei*

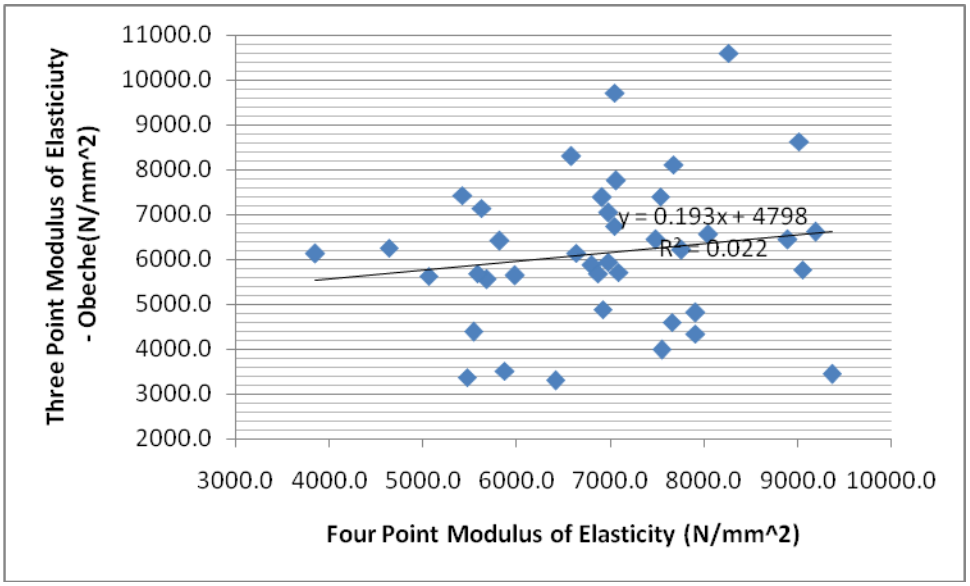



Fig. 4.7 Least Square Regression Plots for Modulus of Elasticity of *Triplochiton scleroxylon*

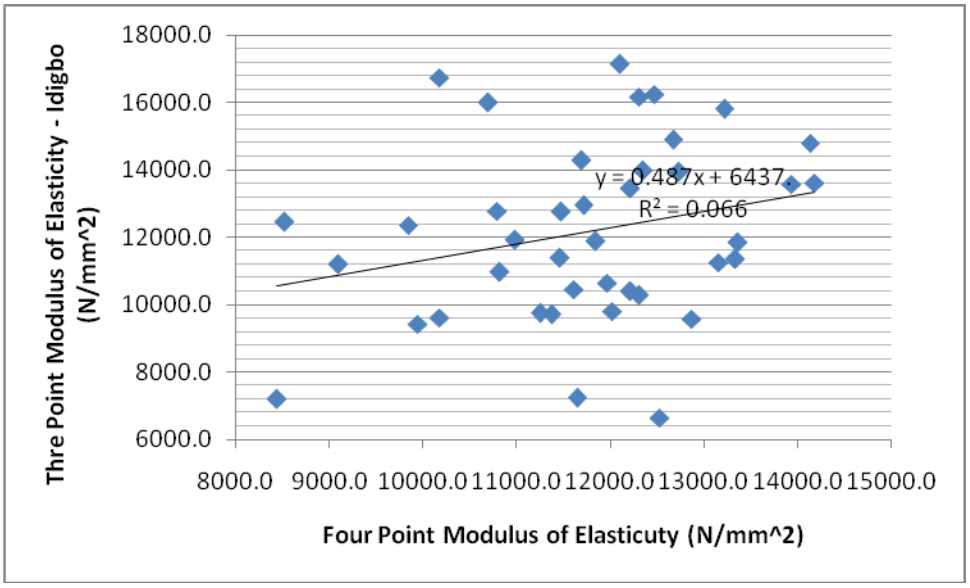



Fig. 4.8 Least Square Regression Plots for Modulus of Elasticity of *Terminalia ivorensis*

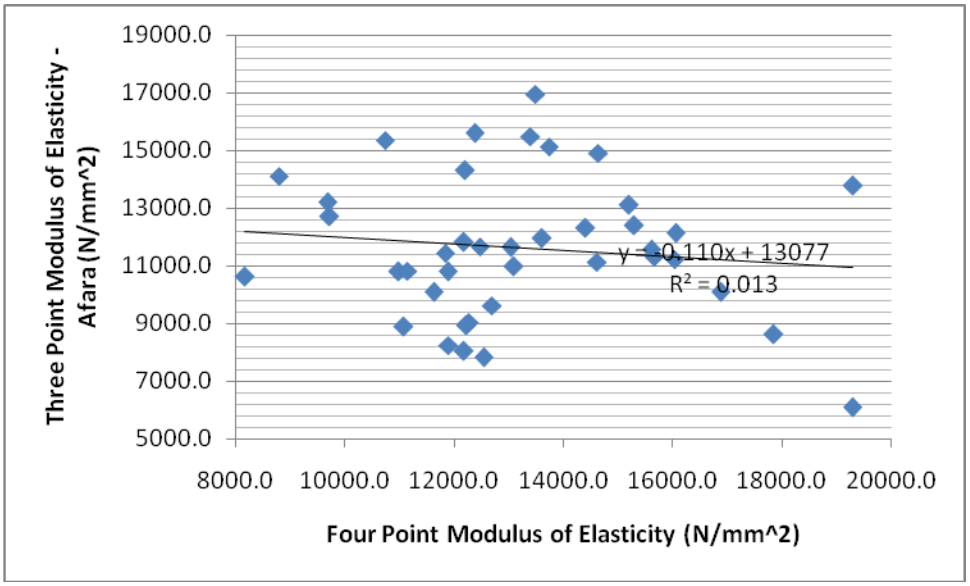



Fig. 4.9 Least Square Regression Plots for Modulus of Elasticity of *Terminalia superba*

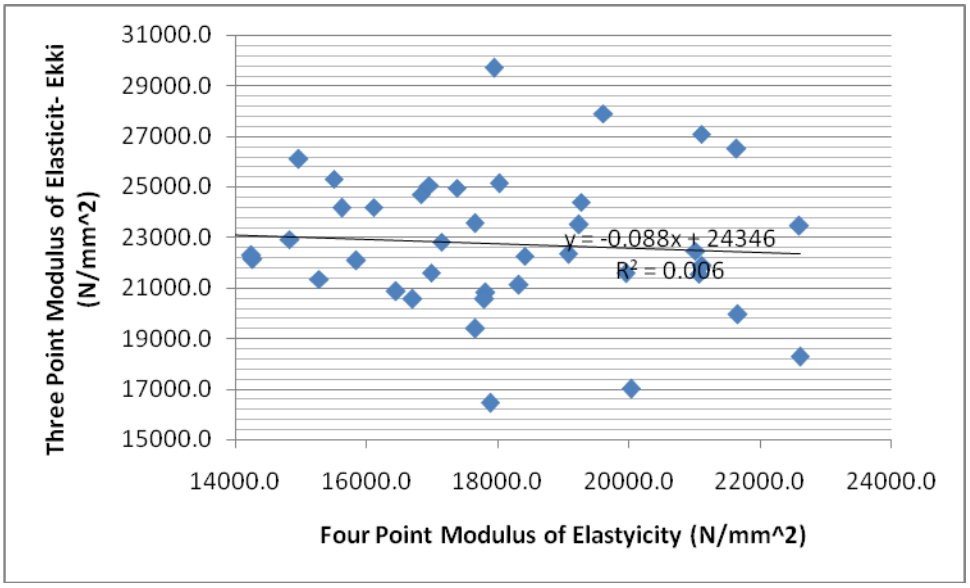



Fig. 4.10 Least Square Regression Plots for Modulus of Elasticity of *Lophira alata*

The mathematical transfer models are presented in Tables 4.7 and 4.8, for the bending strengths and modulus of elasticity respectively.

For instance, from the three-point bending test results for Alstonia Boonei for test piece number 1, the bending strength is 36.80N/mm$^2$ and the modulus of elasticity is 6488.9N/mm$^2$. The transfer model for Alstonia Boonei are $f_{b\text{-}4pt} = 0.102f_{b\text{-}3pt} + 60.63$ and $MOE_{b\text{-}4pt} = 0.030MOE_{b\text{-}3pt} + 7966$, for bending strength and modulus of elasticity respectively. Using these models, the four point bending strength and four point modulus of elasticity of 64.38N/mm$^2$ and 8160.7N/mm$^2$ respectively were obtaiunes. The actual values of the four point bending strength and four point modulus of Elasticity for the first test piece are 66.9N/mm$^2$ and 7886.6N/mm$^2$. The predicted four point bending strength is

less than the actual value by 3.8% (that is 97% accurate). Also, the predicted four point modulus of elasticity exceeded the actual value by only 2.2% (that is 98% accurate).

Table 4.7 Mathematical Transfer Models for Bending Strength

| Timber Specie | Transfer Model from Three-Point to Four-point Bending Strength |
|---|---|
| Alstonia Boonei | $f_{b\text{-}4pt} = 0.102f_{b\text{-}3pt} + 60.63$ |
| Triplochiton Scleroxylon | $f_{b\text{-}4pt} = 0.505f_{b\text{-}3pt} + 46.49$ |
| Terminalia Ivorensis | $f_{b\text{-}4pt} = 0.175f_{b\text{-}3pt} + 81.53$ |
| Terminalia Superba | $f_{b\text{-}4pt} = 0.187f_{b\text{-}3pt} + 95.20$ |
| Lophira Alata | $f_{b\text{-}4pt} = 0.116f_{b\text{-}3pt} + 139.9$ |

Table 4.8 Mathematical TransferModels for Modulus of Elasticity

| Timber Specie | Transfer Model from Three-Point to Four-point Modulus of Elasticity |
|---|---|
| Alstonia Boonei | $MOE_{b\text{-}4pt} = 0.030MOE_{b\text{-}3pt} + 7966$ |
| Triplochiton Scleroxylon | $MOE_{b\text{-}4pt} = 0.193MOE_{b\text{-}3pt} + 4798$ |
| Terminalia Ivorensis | $MOE_{b\text{-}4pt} = 0.487MOE_{b\text{-}3pt} + 6437$ |
| Terminalia Superba | $MOE_{b\text{-}4pt} = 13077 - 0.110MOE_{b\text{-}3pt}$ |
| Lophira Alata | $MOE_{b\text{-}4pt} = 24346 - 0.088MOE_{b\text{-}3pt}$ |

### 4.1.6 Moisture Adjusted Density

The moisture adjusted density values for the five species are presented in Table 4.9. By comparison the density of Alstonia Boonei during test that corresponds to moisture content of 20.59% is 360.76 kg/$m^3$. The density corresponding to moisture contents of 12% and 18% are 345.27kg/$m^3$ and 357kg/$m^3$ respectively. These represent the density lost of 4.29% and 1.04% respectively. This implied that for every 1% drop in moisture content of timber, there is the corresponding drop of 0.1% drop of its density.

Table 4.9: Moisture Adjusted Density

| Timber Specie | Moisture content = 12% | | Moisture content = 18% | |
|---|---|---|---|---|
| | Mean value (kg/$m^3$) | Standard Deviation | Mean value (kg/$m^3$) | Standard Deviation |
| *Alstonia Boonei* | 345.27 | 13.81 | 357.00 | 14.28 |
| *Triplochiton Scleroxylon* | 371.07 | 59.37 | 383.67 | 61.39 |
| *Terminalia Ivorensis* | 454.96 | 81.89 | 470.41 | 84.67 |
| *Terminalia Superba* | 509.97 | 96.89 | 527.29 | 100.19 |
| *Lophira Alata* | 940.16 | 37.61 | 972.09 | 37.08 |

### 4.1.7 Moisture Adjusted Modulus of Elasticity

The moisture adjusted Modulus of Elasticity (MOE) values for the five species are presented in Table 4.10. Considering the value of elastic modulus for Alstonia Boonei at moisture content of 20.59%, the adjusted modulus of elasticity (four-point) is 43.09N/$mm^2$. The value of modulus of elasticity corresponding to moisture contents of

12% and 18% are 9339.20N/mm$^2$ and 8431.77N/mm$^2$ respectively. These represent the respective increment of modulus of elasticity of 14.0% and 2.92% respectively.

Table 4.10: Moisture Adjusted Modulus of Elasticity (MOE)

| Timber Specie | Moisture content = 12% | | Moisture content = 18% | |
|---|---|---|---|---|
| | Mean value (N/mm$^2$) | Standard Deviation | Mean value (N/mm$^2$) | Standard Deviation |
| *Alstonia Boonei* | 9339.20 | 560.35 | 8431.77 | 505.91 |
| *Triplochiton Scleroxylon* | 6593.18 | 1780.16 | 5952.56 | 1607.19 |
| *Terminalia Ivorensis* | 13569.79 | 2985.35 | 12251.29 | 2695.28 |
| *Terminalia Superba* | 13320.76 | 2797.36 | 12026.46 | 2525.56 |
| *Lophira Alata* | 23076.74 | 2767.21 | 20834.51 | 2500.14 |

### 4.1.8 Moisture Adjusted Bending Strength

The moisture adjusted Bending Strength values for the five species are presented in Table 4.11. Once again considering the value of bending strength for *Alstonia boonei* at moisture content of 20.59% is 43.09 N/mm$^2$. The value of Bending Strength corresponding to moisture contents of 12% and 18% are 57.72N/mm$^2$ and 46.15N/mm$^2$ respectively. These represent the respective increment of modulus of elasticity of 33.95% and 7.10% respectively. It is clear from this result, that among all the three reference material properties of timber, moisture variation has the highrst effect on bending strength.

Table 4.11: Moisture Adjusted Bending Strength

| Timber Specie | Moisture content = 12% | | Moisture content = 18% | |
|---|---|---|---|---|
| | Mean value ($N/mm^2$) | Standard Deviation | Mean value ($N/mm^2$) | Standard Deviation |
| *Alstonia Boonei* | 57.72 | 6.93 | 46.15 | 5.54 |
| *Triplochiton Scleroxylon* | 63.05 | 10.16 | 50.41 | 8.07 |
| *Terminalia Ivorensis* | 89.70 | 17.04 | 71.72 | 13.63 |
| *Terminalia Superba* | 115.04 | 21.86 | 91.98 | 17.48 |
| *Lophira Alata* | 108.34 | 17.33 | 86.62 | 13.86 |

### 4.1.9 Distributions Fitting and Tests of Goodness of Fit

Kolmogorov Smirnov (K-S) test was adopted in this study. The test was performed on the density, global modulus of elasticity and bending strength, for each of the five species, in order to access if each data set comes from a population with a specific distribution. 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 (Ang and Tang, 1975). The hypothesis is that "The data follow a specified distribution". The hypothesis regarding the distributional form is rejected if the test statistics, is greater than the critical value. The Kolmogorov Smirnov test of goodness of fit were implemented using Easyfit software. The results are presented in Tables 4.12, 4.13 and 4.14 for density, modulus of elasticity and bending steength respectively.

### 4.1.10 Distribution Model for Density

From Table 4.12, the critical value for Kolmogorov Smirnov test at $\alpha = 0.05$, for density for all the five tested species (*Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia superba and lophira Alata)* is 0.2101. This value is greater than the K-S statistics of both the normal, lognormal and gumbel distribution. The hypothesis that the density values belong to each of the three tested distribution models is now accepted. This implied that, all the three distribution models can be used to model the density of each of the specie. However, considering the preferential ranking of the three fitted distribution model, it can be concluded that, the density of each of the species can best be modeled using Normal distribution. Therefore, the most appropriate theoretical distribution model for timber density is Normal distribution.

The P-Value, in the table is calculated based on the test statistics, and denotes the theoretical value of the significance levels in the sense that the null hypothesis (Ho) will be accepted for all values of $\alpha$ less than the P-Value. P-Values can be useful, in particular when the null hypothesis is rejected at all predefined significant levels, and one need to know at which level it could be accepted.

Table 4.12 Kolmogrov Smirnov Goodness of Fit Tests on Density Test Results at 5% Level of Significance ($\alpha_{0.05}$)

| Timber Specie | Distribution | Statistics | P-value | Rank | Reject Hypothesis |
|---|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | Normal | 0.0929 | 0.9963 | 1 | No |
| | Lognormal | 0.0937 | 0.9918 | 2 | No |
| | Gumbel | 0.1251 | 0.5972 | 3 | No |
| *Triplochiton Scleroxylon* (Obeche) | Normal | 0.1073 | 0.9252 | 1 | No |
| | Lognormal | 0.1189 | 0.6184 | 2 | No |
| | Gumbel | 0.1593 | 0.2803 | 3 | No |
| *Terminalia Ivorensis* (Black Afara) | Normal | 0.0704 | 0.9136 | 1 | No |
| | Lognormal | 0.1114 | 0.7996 | 2 | No |
| | Gumbel | 0.1324 | 0.4423 | 3 | No |
| *Terminalia Superba* (White Afara) | Normal | 0.0599 | 0.9948 | 1 | No |
| | Lognormal | 0.0870 | 0.9118 | 2 | No |
| | Gumbel | 0.1178 | 0.7266 | 3 | No |
| *Lophira alata* (Ekki) | Normal | 0.0789 | 0.9442 | 1 | No |
| | Lognormal | 0.0830 | 0.8968 | 2 | No |
| | Gumbel | 0.1103 | 0.5189 | 3 | No |

Table 4.13 Kolmogrov Smirnov Goodness of Fit Tests on Modulus of Elasticity Test Results At 5% LEVEL OF SIGNIFICANCE ($\alpha_{0.05}$)

| Timber Specie | Distribution | Statistics | P-value | Rank | Reject Hypothesis |
|---|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | Normal | 0.0998 | 0.5998 | 2 | No |
| | Lognormal | 0.0948 | 0.3766 | 1 | No |
| | Gumbel | 0.3715 | 0.1059 | 3 | No |
| *Triplochiton Scleroxylon* (Obeche) | Normal | 0.1132 | 0.9095 | 1 | No |
| | Lognormal | 0.1606 | 0.9546 | 2 | No |
| | Gumbel | 0.1650 | 0.6120 | 3 | No |
| *Terminalia Ivorensis* (Black Afara) | Normal | 0.0749 | 0.9678 | 1 | No |
| | Lognormal | 0.0789 | 0.8472 | 2 | No |
| | Gumbel | 0.0899 | 0.6018 | 3 | No |
| *Terminalia Superba* (White Afara) | Normal | 0.0761 | 0.7040 | 1 | No |
| | Lognormal | 0.1076 | 0.7462 | 2 | No |
| | Gumbel | 0.1214 | 0.5150 | 3 | No |
| *Lophira alata* (Ekki) | Normal | 0.0929 | 0.5445 | 2 | No |
| | Lognormal | 0.0922 | 0.7810 | 1 | No |
| | Gumbel | 0.0983 | 0.6605 | 3 | No |

### 4.1.11 Distribution Model for Modulus of Elasticity

Table 4.13 displayed the result of goodness of fit test for modulus of elasticity of structural timber, considering the generated material propetties of the five tier species under investigation in this work. The critical value in 0.2101 which is greater than the corresponding K-S statistics of the considered distribution models (normal, lognormal

and gumbel distribution). This is a clear warrant that each of the three distribution models can be used to model the modulus of elasticity of timber. Considering the ranking of the validity of the three distribution model, it is observed that, the most fitted distribution model for modulus of elasticity of Alstonia Boonei and Lophira Alata is normal distribution, while for Triplochiton Scleroxylon, terminal superbas and terminalia ivorensis, the best distribution model is lognormal distribution; all at 5% significance level.

### 4.1.12 Distribution Model for Bending Strength

The Kolmogorov Smirnov test of goodness of fit test at $\alpha = 0.05$, for bending strengths of all the five species under investigation as presented in Table 4.14. This value is greater than the corresponding K-S statistics of the normal, lognormal, and gumbel distribution. Each of the three distribution models can therefore be used to model the bending strength of timber. Considering the ranking of the validity of the three distribution model, it is observed that, the most fitted distribution model for bending strength of Alstonia Boonei is the normal distribution, while fot the other four species, the most fitted distribution model is Lognormal Distribution.

Table 4.14 Kolmogrov Smirnov Goodness of Fit Test on Bending Strength Test Results at 5% Level of Significance ($\alpha_{0.05}$)

| Timber Specie | Distribution | Statistics | P-value | Rank | Reject Hypothesis |
|---|---|---|---|---|---|
| *Alstonia Boonei* (Ahun) | Normal | 0.1181 | 0.7843 | 1 | No |
| | Lognormal | 0.1402 | 0.8313 | 2 | No |
| | Gumbel | 0.1873 | 0.3715 | 3 | No |
| *Triplochiton Scleroxylon* (Obeche) | Normal | 0.0874 | 0.6434 | 2 | No |
| | Lognormal | 0.0775 | 0.2279 | 1 | No |
| | Gumbel | 0.1161 | 0.2025 | 3 | No |
| *Terminalia Ivorensis* (Black Afara) | Normal | 0.0744 | 0.9661 | 1 | No |
| | Lognormal | 0.0931 | 0.9478 | 2 | No |
| | Gumbel | 0.1171 | 0.8743 | 3 | No |
| *Terminalia Superba* (White Afara) | Normal | 0.1075 | 0.9612 | 2 | No |
| | Lognormal | 0.1035 | 0.7027 | 1 | No |
| | Gumbe | 0.1254 | 0.5562 | 3 | No |
| *Lophira alata* (Ekki) | Normal | 0.1011 | 0.8489 | 3 | No |
| | Lognormal | 0.1000 | 0.8551 | 1 | No |
| | Gumbel | 0.1116 | 0.7983 | 2 | No |

### 4.1.13 Skeweness and Excess Kurtosis of Density

Figs 4.11 to 4.15 are the histograms showing the distribution of density data and plots of the probability density functions pdf of the three fitted theoretical distribution models (normal distribution, lognormal distribution and gumbel distribution) for *Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia superba* and *Lophira alata*

respectively. Table 4.15 presents values of the skeweness and excess kurtosis for the density of each of the five timber species. Based on the values presented for, it is clear that the density data are moderately skewed. *Alstonia boonei* and *Terminalia superba* are skewed to the right with skewenes of +0.15 and +0.18 respectively, while, *Triplochiton scleroxylon; Terminalia ivorensis* and *Lophira alata* are negatively skewed with skeweness of -0.16, -0.49 and -0.35 respectively.

The pdf plots of *Triplochiton scleroxylon* and *Lophira alata* have sharp central peak (leptokurtic), with positive excess kurtosis of +0.07 and +0.66 respectively. On the other hand the pdfs of the density of *Alstonia boonei, Terminalia ivorensis* and *Terminalia superba* are lower and broader shape (platykurtic), with excess kurtosis of -1.0, -0.14 and -0.63 respectively.

Table 4.15: Skeweness and Excess Kurtosis for Density

| Timber Specie | Skeweness | Excess Kurtosis |
|---|---|---|
| *Alstonia boonei* (Ahun) | 0.15 | -1.00 |
| *Triplochiton scleroxylon* (Obeche) | -0.16 | 0.07 |
| *Terminalia ivorensis* (Black Afara) | -0.49 | -0.14 |
| *Terminalia superba* (White Afara) | 0.18 | -0.63 |
| *Lophira alata* (Ekki) | -0.35 | 0.66 |

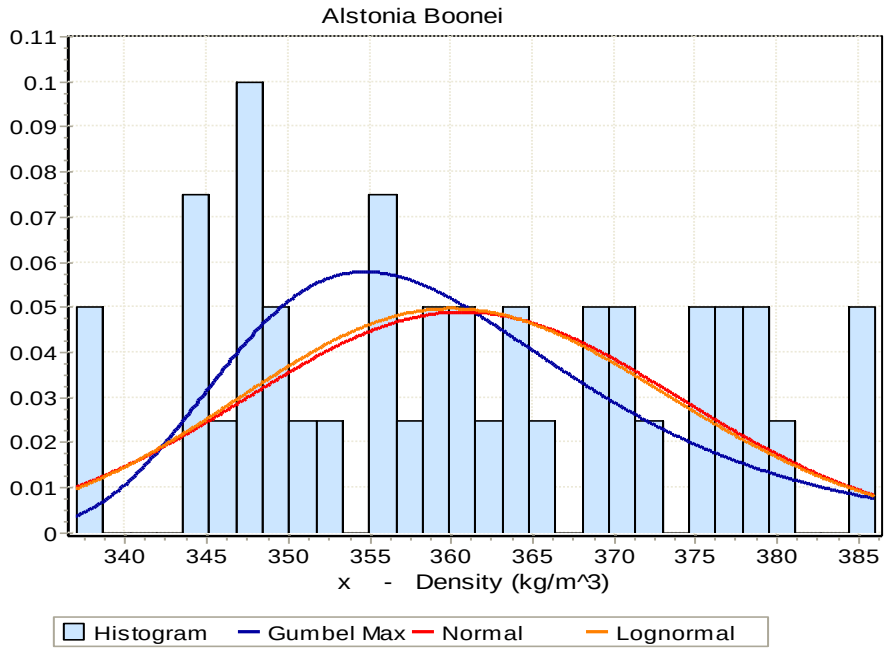



Fig. 4.11 Probability Density Function Plots for Density of *Alstonia boonei*

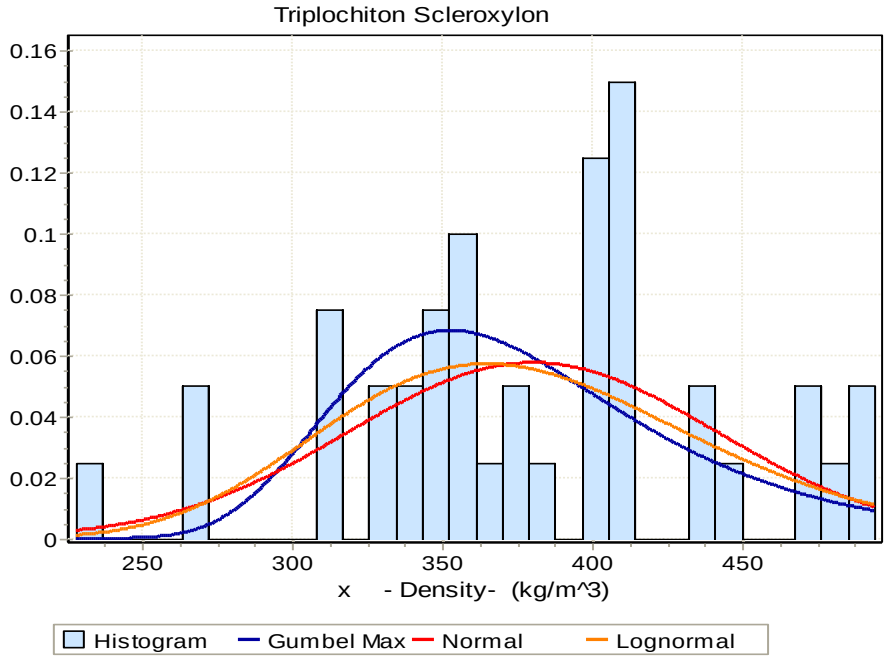



Fig. 4.12 Probability Density Function Plots for Density of *Triplochiton scleroxylon*

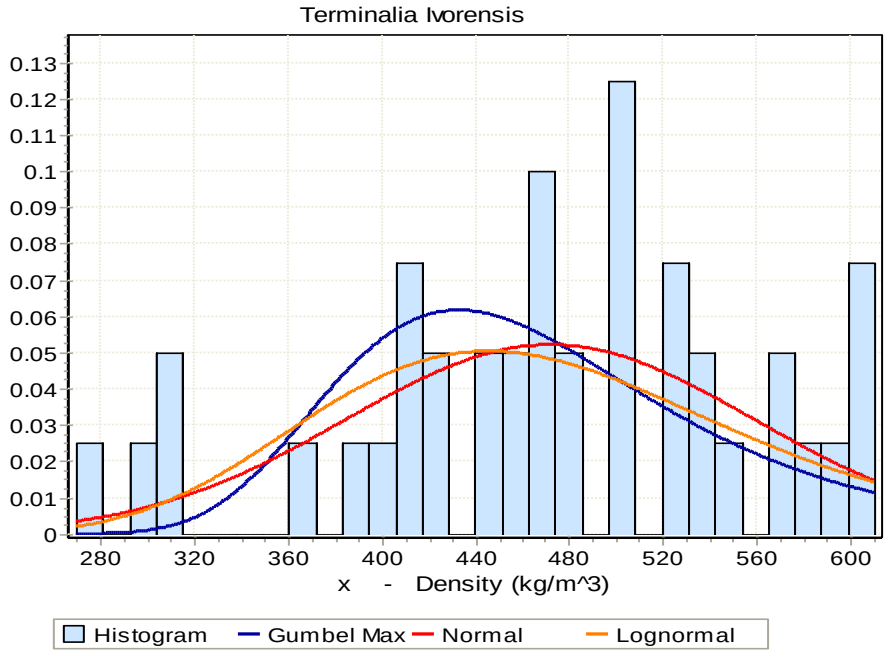



Fig.4.13 Probability Density Function Plots for Density of *Terminalia ivorensis*

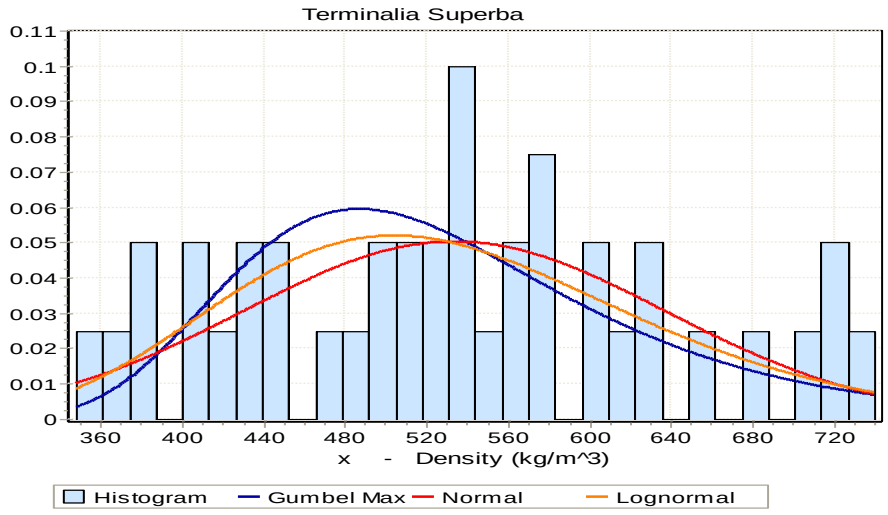



Fig. 4.14 Probability Density Function Plots for Density of *Terminalia superba*

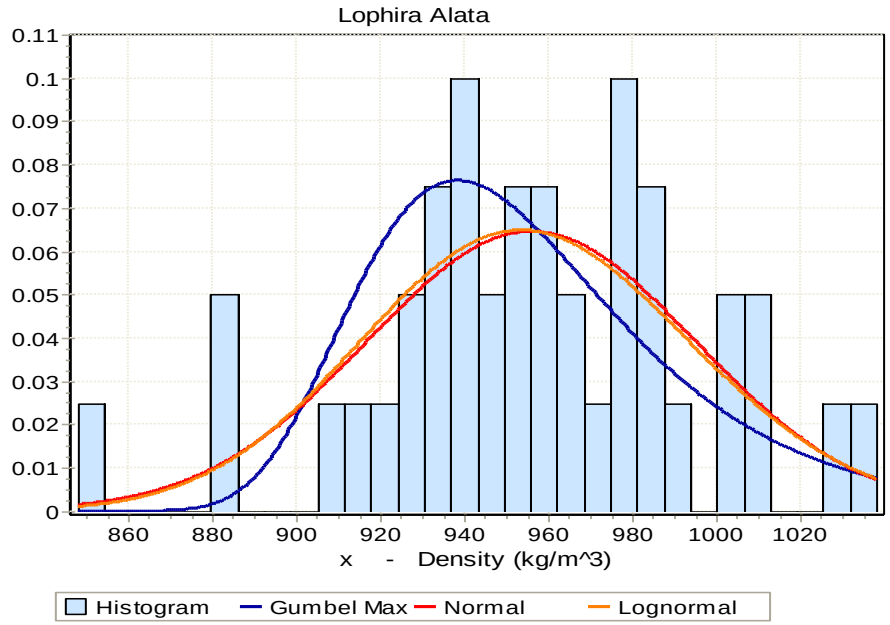



Fig. 4.15 Probability Density Function Plots for Density of *Lophira alata*

### 4.1.14 Skeweness and Excess Kurtosis of Modulus of Elasticity

In Figs 4.16 to 4.10 the histograms of the distribution of modulus of elasticity data with the plots of the fitted probability density functions pdf were displayed. The value of skeweness and kurtosis pf the fitted pdf are presented in Table 4.16. The results also shown that the modulus of elasticity data for all the tested timber species are moderately skewed. MOE of Alstonia boonei, and Terminalia ivorensis skewed to the left, with skeweness values of -0.46 and -0.01 respectively. For Triplochiton scleroxylon, Termiania superba and Lophira alata are positively skewed, with skeweness values of +0.41, +0.09 and +0.10 respectively.

Table 4.16: Skeweness and Excess Kurtosis for Modulus of Elasticity

| Timber Specie | Skeweness | Excess Kurtosis |
|---|---|---|
| *Alstonia boonei* (Ahun) | -0.46 | 1.66 |
| *Triplochiton scleroxylon* (Obeche) | 0.41 | 0.52 |
| *Terminalia ivorensis* (Black Afara) | -0,01 | -0.47 |
| *Terminalia superba* (White Afara) | 0.09 | -0.27 |
| *Lophira alata* (Ekki) | 0.10 | 0.55 |

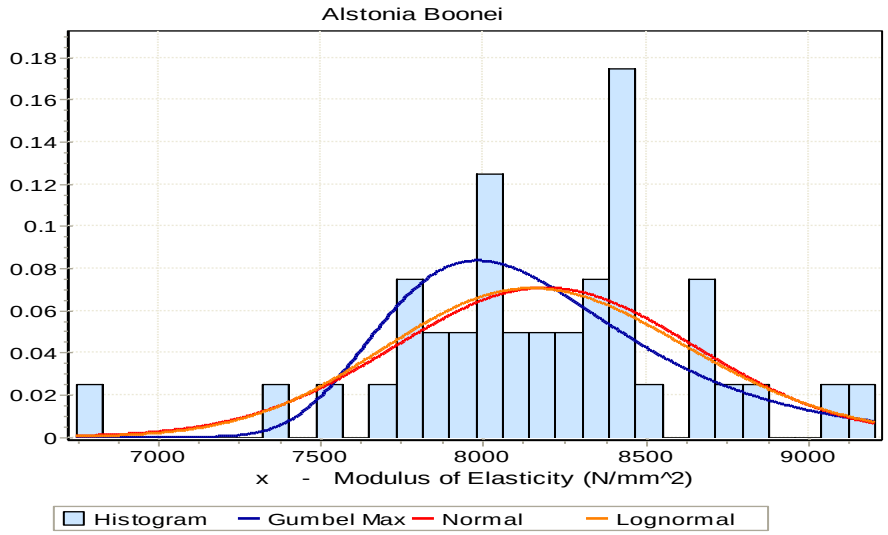



Fig. 4.16 Probability Density Function Plots for Modulus of Elasticity of *Alstonia Boonei*

The pdf plots of modulus of elasticity of *Alstonia boonei*, *Triplochiton scleroxylon* and *Lophira alata*, have sharp central peak (leptokurtic), with positive excess kurtosis of +1.66 +0.52 and +0.55 respectively. On the other hand the pdfs plots of modulus of elasticity of *Terminalia ivorensis* and *Terminalia superba* are lower and broader shape (platykurtic), with excess kurtosis of -0.47, and -0.27 respectively.

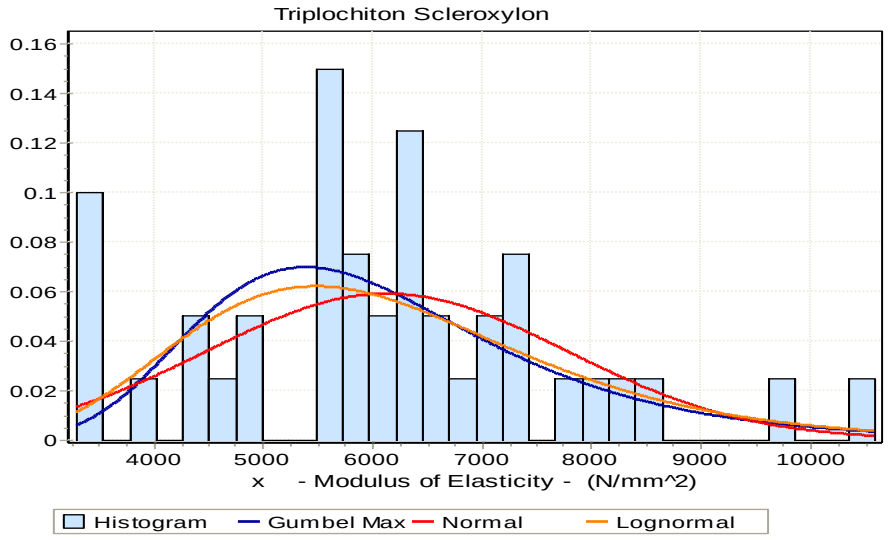



Fig. 4.17 Probability Density Function Plots for Modulus of Elasticity of *Triplochiton Scleroxylon*

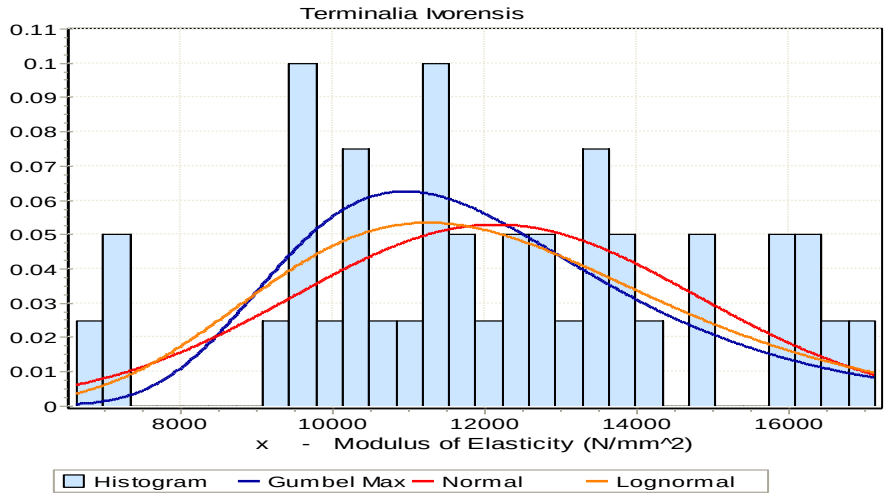



Fig.4.18 Probability Density Function Plots for Modulus of Elasticity of *Terminalia Ivorensis*

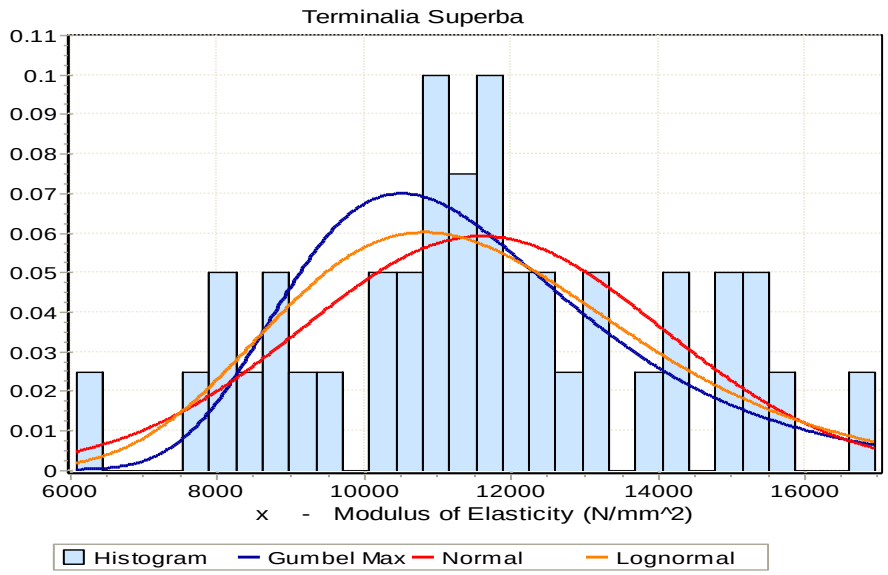



Fig. 4.19 Probability Density Function Plots for Modulus of Elasticity of *Terminalia Superba*

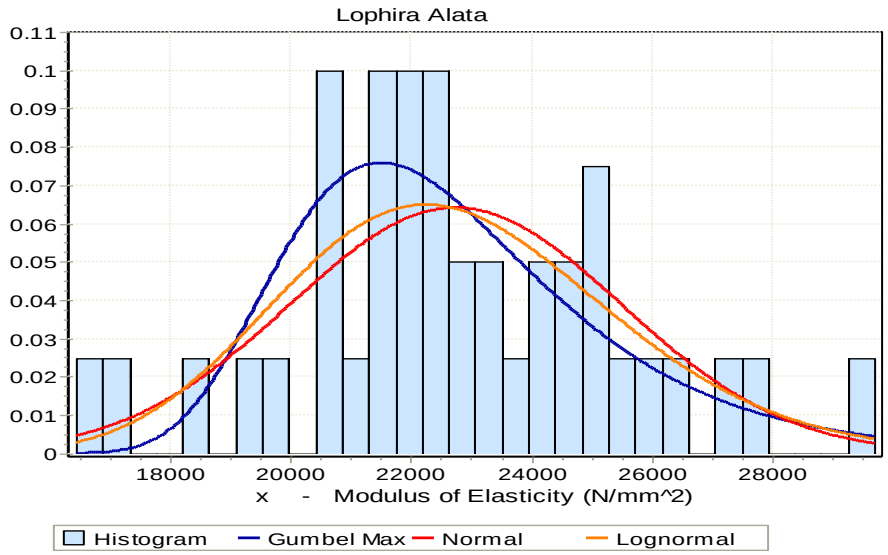



Fig. 4.20 Probability Density Function Plots for Modulus of Elasticity of *Lophira Alata*

### 4.1.15 Skeweness and Excess Kurtosis of Bending Strength

The histograms of the distribution of bending strength data with the plots of the fitted probability density functions pdf were displayed in Figs 4.21 to 4.25. The value of skeweness and kurtosis of the fitted pdf are presented in Table 4.17. As observed from the presented results the bending strength data for all the five candidate timber species are moderately skewed. Bending strength of *Alstonia boonei*, and *Terminalia ivorensis* skewed to the left, with skeweness values of -0.05 and -0.06 respectively. For *Triplochiton scleroxylon, Termialia superba and Lophira alata* are positively skewed, with skeweness values of +0.45, +0.15 and +0.46 respectively.

**Table 4.17: Skeweness and Excess Kurtosis for Bending Strength**

| Timber Specie | Skeweness | Excess Kurtosis |
|---|---|---|
| *Alstonia boonei* (Ahun) | -0.09 | -0.96 |
| *Triplochiton scleroxylon* (Obeche) | 0.45 | -0.002 |
| *Terminalia ivorensis* (Black Afara) | -0.06 | 0.08 |
| *Terminalia superba* (White Afara) | 0.15 | -1.05 |
| *Lophira alata* (Ekki) | 0.46 | 0.42 |

The pdf plots of bending strength of *Terminalia ivorensis* and *Lophira alata*, have sharp central peak (leptokurtic), with positive excess kurtosis of +0.08 and +0.42 respectively. On the other hand the pdfs plots of modulus of elasticity of *Alstonia boonei, Triplochiton*

*scleroxylon* and *Terminalia superba* are lower and broader shape (platykurtic), with excess kurtosis of -0.96, -0.002 and -1.05 respectively

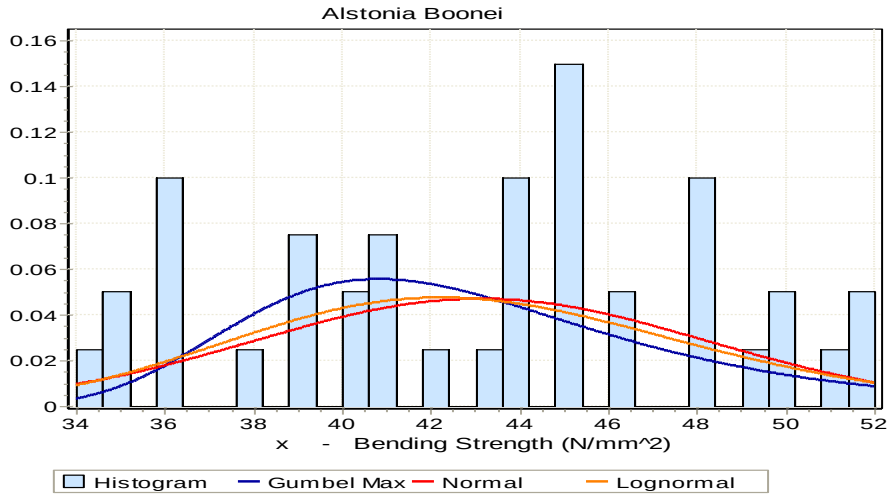



Fig. 4.21 Probability Density Function Plots for Bending Strength of *Alstonia Boonei*

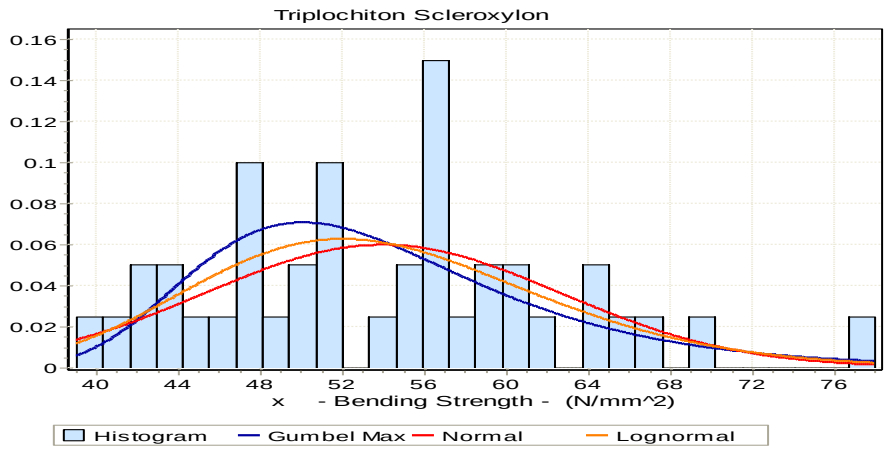



Fig. 4.22 Probability Density Function Plots for Bending Strength of *Triplochiton Scleroxylon*

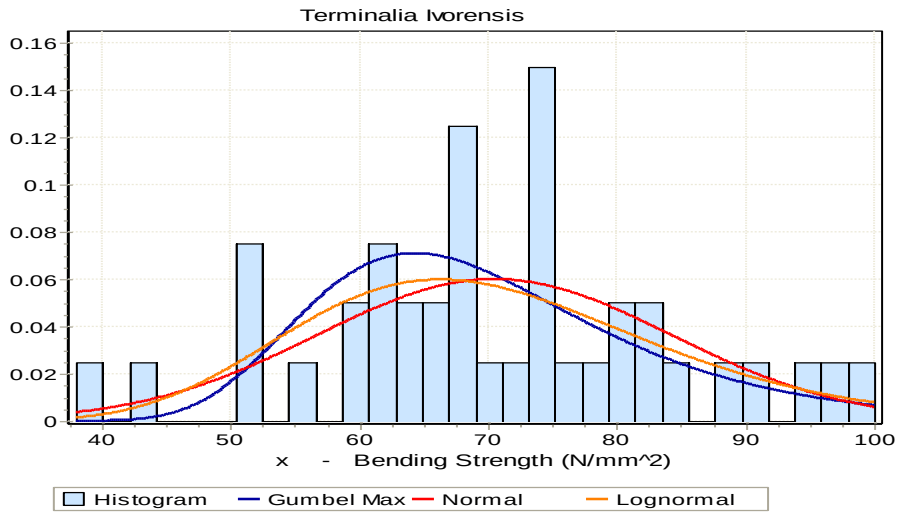



Fig. 4.23 Probability Density Function Plots for Bending Strength of *Terminalia Ivorensis*

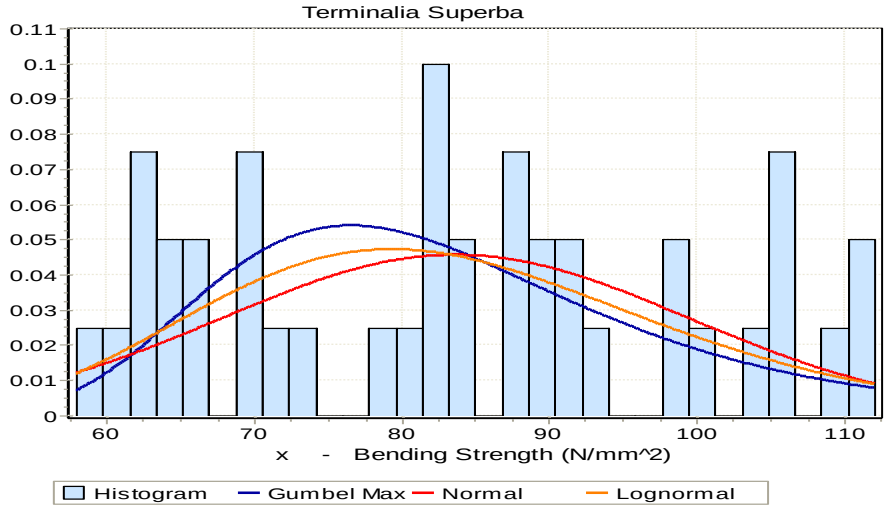



Fig. 4.24 Probability Density Function Plots for Bending Strength of *Terminalia Superba*

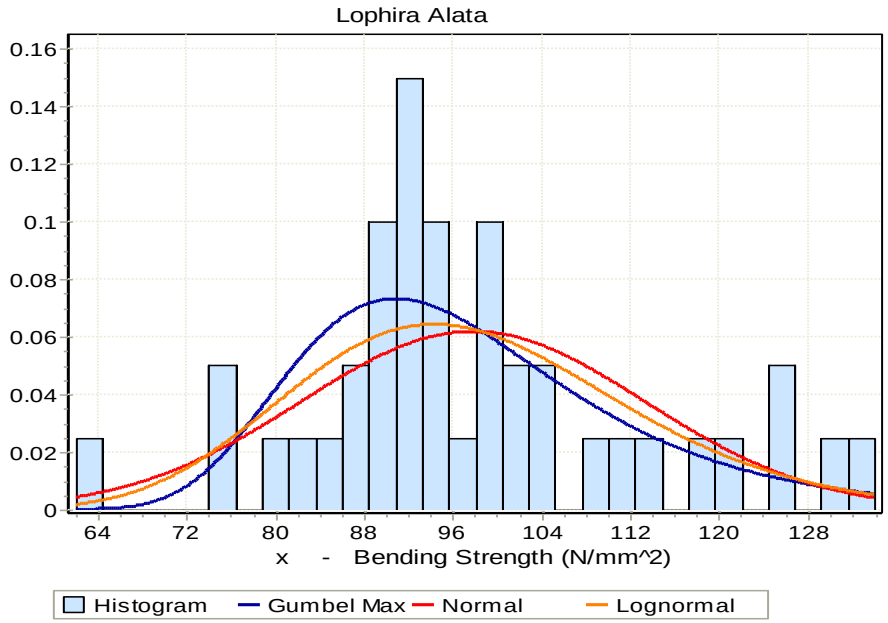



Fig. 4.25 Probability Density Function Plots for Bending Strength of *Lophira Alata*

### 4.1.16 Percentile Values

Given a data set that has been ordered in increasing magnitude, the median, first quartile and third quartile can be used to split the data into four pieces. The first quartile is the point at which one fourth of the the data lies below it. The medin is located exactly in the middle of the data set, with half of all of the data below it. The third quartile is the place where three fouth of the data lies below it. To split the data to more than just two pieces, the idea of the quartile is generalized to that of percentile. The nth percentile of a set of data is the point where n% of the data is below it. The characteristic values of material properties reported in EN 338 (2009) are base on the 5 percentile values (EN 384, 2003). The percentile values of Density, Modulus of elasticity and Bending Strength were

genereated from EASYFIT statistical package. The results presented in Tables. 4.18 to 4.20 respectivey.

**Table 4.18 Percentile Values of Density**

| Percentile ($kg/m^3$) | Timber Specie | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
| Minimum | 336.94 | 227.57 | 270.49 | 348.13 | 847.84 |
| Maximum | 385.61 | 494.16 | 609.53 | 739.70 | 1038.00 |
| 5% | 338.05 | 269.45 | 294.70 | 371.40 | 882.47 |
| 10% | 345.21 | 309.29 | 320.26 | 381.25 | 907.85 |
| 25% (Lower Quatile) | 348.52 | 342.30 | 414.98 | 450.02 | 934.49 |
| 50% (Median) | 359.71 | 375.25 | 477.11 | 532.58 | 955.93 |
| 75% (Upper Quatile) | 371.34 | 410.15 | 530.87 | 606.08 | 981.12 |
| 90% | 379.66 | 474.84 | 588.59 | 704.22 | 1010.30 |
| 95% | 385.28 | 491.52 | 603.43 | 722.84 | 1025.70 |

**Table 4.19. Percentile Values of Modulus of Elasticity (MOE)**

| Percentile ($N/mm^2$) | Timber Specie | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
| Minimum | 6751.50 | 3291.00 | 6639.20 | 6978.10 | 16454.00 |
| Maximum | 9202.50 | 10594.00 | 17141.00 | 16958.00 | 29686.00 |
| 5% | 7342.50 | 3372.40 | 7201.00 | 7834.00 | 17062.00 |
| 10% | 7712.80 | 3557.20 | 9435.00 | 8271.50 | 19442.00 |
| 25% (Lower Quatile) | 7988.00 | 5052.20 | 10311.00 | 10079.00 | 21178.00 |
| 50% (Median) | 8229.20 | 6138.50 | 11897.00 | 11495.00 | 22812.00 |
| 75% (Upper Quatile) | 8422.70 | 7114.80 | 13967.00 | 13179.00 | 24625.00 |
| 90% | 8777.60 | 8272.40 | 16148.00 | 15593.00 | 26449.00 |
| 95% | 9093.40 | 9661.40 | 17141.00 | 16958.00 | 27852.00 |

**Table 3.20. Percentile Values of Bending Strength (BS)**

| Percentile ($N/mm^2$) | Timber Specie | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
| Minimum | 33.66 | 39.38 | 37.71 | 58.28 | 62.09 |
| Maximum | 51.82 | 77.50 | 99.59 | 112.41 | 133.57 |
| 5% | 35.14 | 40.74 | 22.76 | 61.31 | 74.59 |
| 10% | 35.77 | 42.52 | 51.35 | 62.54 | 79.92 |
| 25% (Lower Quatile) | 39.02 | 47.53 | 61.65 | 70.17 | 89.30 |
| 50% (Median) | 43.96 | 54.22 | 70.18 | 83.66 | 93.92 |
| 75% (Upper Quatile) | 47.22 | 59.35 | 79.76 | 97.21 | 103.65 |
| 90% | 49.95 | 65.92 | 90.14 | 105.20 | 124.12 |
| 95% | 51.49 | 69.17 | 95.81 | 110.70 | 133.57 |

### 4.1.17 Characteristic Values of the Reference Material Properties

The sawn timber tested are represented by 200 specimen (40 for each specie) tested using four point bending test on the Avery Denison Universal Testing Machine. The characteristic values determined in accordance with EN 384 are given in Table 4.21, Table 4.22 and 4.23 for bending strength modulus of elasticity and density respectively.

Table 4.21: 5% Fractile and Characteristic Values of Bending Strength

| *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia Ivorensis* | *Terminalia Superba* | *Lophira Alata* |
|---|---|---|---|---|
| $f_k$ ($N/mm^2$) | $f_k$ ($N/mm^2$) | $f_k$ ($N/mm^2$) | $f_k$ ($N/mm^2$) | $f_k$ ($N/mm^2$) |
| 39.36 | 45.63 | 50.13 | 68.67 | 83.54 |

Table 4.22: 5% Fractile and Characteristic Values of Modulus of Elasticity

| *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia Ivorensis* | *Terminalia Superba* | *Lophira Alata* |
|---|---|---|---|---|
| $E_k$ ($N/mm^2$) | $E_k$ ($N/mm^2$) | $E_k$ ($N/mm^2$) | $E_k$ ($N/mm^2$) | $E_k$ ($N/mm^2$) |
| 7597.02 | 3300.70 | 7804.08 | 7859.29 | 16709.28 |

Table 4.23: 5% Fractile and Characteristic Values of Density

| *Alstonia Boonei* | *Triplochiton Scleroxylon* | *Terminalia Ivorensis* | *Terminalia Superba* | *Lophira Alata* |
|---|---|---|---|---|
| $\rho_k$ ($kg/m^3$) | $\rho_k$ ($kg/m^3$) | $\rho_k$ ($kg/m^3$) | $\rho_k$ ($kg/m^3$) | $\rho_k$ ($kg/m^3$) |
| 333.44 | 282.38 | 330.70 | 361.98 | 910.91 |

### 4.1.18 Characteristic Value of the Derived Material Properties

The reference material properties were generated in this study from series of laboratory experiments, as presented in the previous sections. Other properties of timber such as tension and compression strength parallel and perpendicular to grain, shear strength, and hear modulus, are the derived material properties.

Table 4.24: Derived Material Properties of *Alstonia boonei*

| Material Property | Unit | Mean | Coefficient of Variation | Characteristic Value |
|---|---|---|---|---|
| Tension Strength Parallel to Grain | $N/mm^2$ | 27.69 | 0.14 | 21.29 |
| Tension Strength Perpendicular to Grain | $N/mm^2$ | 0.61 | 0.04 | 0.57 |
| Compression Strength Parallel to Grain | $N/mm^2$ | 33.65 | 0.10 | 28.10 |
| Compression Strength Perpendicular to Grain | $N/mm^2$ | 2.86 | 0.04 | 2.67 |
| Shear Strength | $N/mm^2$ | 4.50 | 0.12 | 3.61 |
| Shear Modulus | $N/mm^2$ | 526.99 | 0.06 | 474.82 |

Table 4.25: Derived Material Properties of *Triplochiton scleroxylon*

| Material Property | Unit | Mean | Coefficient of Variation | Characteristic Values |
|---|---|---|---|---|
| Tension Strength Parallel to Grain | $N/mm^2$ | 30.25 | 0.19 | 20.77 |
| Tension Strength Perpendicular to Grain | $N/mm^2$ | 0.65 | 0.16 | 0.48 |
| Compression Strength Parallel to Grain | $N/mm^2$ | 35.02 | 0.13 | 27.51 |
| Compression Strength Perpendicular to Grain | $N/mm^2$ | 3.07 | 0.16 | 2.26 |
| Shear Strength | $N/mm^2$ | 4.83 | 0.16 | 3.55 |
| Shear Modulus | $N/mm^2$ | 372.04 | 0.27 | 206.30 |

Table 4.26: Derived Material Properties of *Terminalia ivorensis*

| Material Property | Unit | Mean | Coefficient of Variation | Characteristic Values |
|---|---|---|---|---|
| Tension Strength Parallel to Grain | $N/mm^2$ | 43.03 | 0.23 | 26.70 |
| Tension Strength Perpendicular to Grain | $N/mm^2$ | 0.8 | 0.18 | 0.56 |
| Compression Strength Parallel to Grain | $N/mm^2$ | 41.04 | 0.15 | 30.88 |
| Compression Strength Perpendicular to Grain | $N/mm^2$ | 3.76 | 0.18 | 2.64 |
| Shear Strength | $N/mm^2$ | 6.41 | 0.19 | 4.40 |
| Shear Modulus | $N/mm^2$ | 765.71 | 0.22 | 487.76 |

Table 4.27: Derived Material Properties of *Terminalia superba*

| Material Property | Unit | Mean | Coefficient of Variation | Characteristic Values |
|---|---|---|---|---|
| Tension Strength Parallel to Grain | $N/mm^2$ | 55.19 | 0.23 | 34.25 |
| Tension Strength Perpendicular to Grain | $N/mm^2$ | 0.90 | 0.19 | 0.62 |
| Compression Strength Parallel to Grain | $N/mm^2$ | 45.90 | 0.15 | 34.54 |
| Compression Strength Perpendicular to Grain | $N/mm^2$ | 4.22 | 0.19 | 2.90 |
| Shear Strength | $N/mm^2$ | 7.82 | 0.19 | 5.37 |
| Shear Modulus | $N/mm^2$ | 751.65 | 0.21 | 491.20 |

Table 4.28: Derived Material Properties of *Lophira alata*

| Material Property | Unit | Mean | Coefficient of Variation | Characteristic Values |
|---|---|---|---|---|
| Tension Strength Parallel to Grain | $N/mm^2$ | 51.97 | 0.19 | 35.68 |
| Tension Strength Perpendicular to Grain | $N/mm^2$ | 1.65 | 0.04 | 1.54 |
| Compression Strength Parallel to Grain | $N/mm^2$ | 44.68 | 0.13 | 35.10 |
| Compression Strength Perpendicular to Grain | $N/mm^2$ | 7.78 | 0.04 | 7.27 |
| Shear Strength | $N/mm^2$ | 7.45 | 0.16 | 5.48 |
| Shear Modulus | $N/mm^2$ | 1302.16 | 0.12 | 1044.33 |

The characteristic strengths of these properties were generated for each specie based on the transformation functions presented in EN 384 (2003) and JCSS, (2006). The results are presented in Tables 4.24 to 4.28

### 4.1.19 Bartlet's Test

Bartlets tests were executed in this study, using 'Analyse it' statistical package, the results are presented in Figs 4.26, 4.2.7 and 4.2.8 for density,modulus of elasticity and bending strength respectively. The hypothesis was accepted for all cases, except in Table 3.30, where, the p-value for modulus of elasticity is less than the significance level, and therefore rejected. The decision therefore warranted the use of the analysis of variance

(ANOVA), in testing null hypothesis on the equality of mean density, modulus of elasticity and bending strength.

Table 4.29 Bartlet's Test for Density

| Timber Specie | P-Value | Decision |
|---|---|---|
| *Alstonia boonei* (Ahun) | 0.135 | Accept |
| *Trioplochiton Scleroxylon* (Obeche) | 0.069 | Accept |
| *Terminalia Ivorensis* (Black Afara) | 0.781 | Accept |
| *Terminalia superba* (White Afara) | 0.604 | Accept |
| *Lophira Alata* (Ekki) | 0.223 | Accept |

**Table 4.30 Bartlet's Test for Modulus of Elasticity**

| Timber Specie | P-Value | Decision |
|---|---|---|
| *Alstonia boonei* (Ahun) | 0.313 | Accept |
| *Trioplochiton Scleroxylon* (Obeche) | 0.235 | Accept |
| *Terminalia Ivorensis* (Black Afara) | 0.788 | Accept |
| *Terminalia superba* (White Afara) | 0.029 | Reject |
| *Lophira Alata* (Ekki) | 0.728 | Accept |

Table 4.31 Bartlet's Test for Bending Strength

| Timber Specie | P-Value | Decision |
|---|---|---|
| *Alstonia boonei* (Ahun) | 0.963 | Accept |
| *Trioplochiton Scleroxylon* (Obeche) | 0.613 | Accept |
| *Terminalia Ivorensis* (Black Afara) | 0.832 | Accept |
| *Terminalia superba* (White Afara) | 0.998 | Accept |
| *Lophira Alata* (Ekki) | 0.275 | Accept |

### 4.1.20 The Two-way Analysis of Variance (Two-way ANOVA)

F-tests are used to compare the between- and within- timber species properties variances to determine if they are different. A hypothesis test is shown which tests the hypothesis that the specie samples have different means. The *p*-value shows the probability of rejecting the null hypothesis, that the samples have the same mean, when it is in fact true. A significant p-value implies that at least two samples have different means. Tables 4.32, 4.33 and 4.34 gave the results of two-way analysis of variance, implemented on Analyse it statistical package. The results shows that, the assumption of equal mean density, modulus of elasticity and bending strength within samples of species can be accepted. However, the null hypothesis for equality of mean value of material properties between timber species is not valid, and therefore rejected. The conclusion is that, the samples of each specie considered for test were not affected by the difference in the sources of the material. The data can therefore be used to classify each of the tested species into it appropriate strength class in accordance with EN 338 (2009).

Table 4.32 Two-way ANOVA (Density)

| Source of variability | F-Statistics | p-value | Critical value | Decision |
|---|---|---|---|---|
| Within timber specie | 1.11 | 0.3244 | 0.05 | Reject |
| Between timber species | 566.03 | 0.0001 | 0.05 | Accept |

Table 4.33 Two-way ANOVA (Modulus of Elasticity**)**

| Source of variability | F-Statistics | p-value | Critical value | Decision |
|---|---|---|---|---|
| Within timber specie | 1.10 | 0.3330 | 0.05 | Reject |
| Between timber species | 357.58 | 0.0001 | 0.05 | Accept |

Table 4.34 Two-way ANOVA (Bending strength)

| Source of variability | F-Statistics | p-value | Critical value | Decision |
|---|---|---|---|---|
| Within timber specie | 1.00 | 0.4768 | 0.05 | Reject |
| Between timber species | 125.03 | 0.0001 | 0.05 | Accept |

### 4.1.21 Allocation of Strength Classes

Strength classes are sets of material properties that can be assigned to specific combinations of timber species and strength graded. The use of strength classes is intended to simplify the specification of timber, since it groups species in pre-defined categories. Characteristic values are taken from bending strength, bending stiffness and density. The allocation to a strength class is governed by the lowest strength class applicable to one of these properties. The classification of the species into various hardwood strength classes was made using the characteristic bending strength, mean

value of density and mean values of modulus of elasticity. The limiting values of the reference material properties from EN 338 (2009) are presented in Table 4.35. Each of the five timber species was then assigned to appropriate timber strength. These were presented in Tables 4.36, 4.37, 4.38, 4.39 and 4.40 for *Alstonia boonei* (Ahun), *Triolochiton scleroxylon* (Obeche), *Terminalia ivorensis* (Black Afara), *Terminalia superb*a (White Afafra) and *Lophira alata* (Ekki) respectivey. In the tables, values of reference material properties based on 12% and 18% moisture content as well as the NCP 2 (1973) Dry-basic stress (at 18% moisute content) were compared. A strength class was assigned to each specie based on the limiting properties orecented in Table 4.35. As observed from the table, the values of the bending strength in the NCP 2 were are very small. The reason is that, the NCP 2 data was based on the old British timber design code (CP 112). The dry stresses were based on the test on a clear timber specimen. On the other hand, the values in EN 338, are based on test on full size structural timber. There is the need to review the NCP 2 to be in line with the current global technology best practice.

Table 4.35 Limiting Values for Classification into Hardwood Classes (EN 338, 2009)

| S/No. | Strength Class | Limits of Characteristic Bending Strenght ($N/mm^2$) | Limits of Characteristic Density ($kg/m^3$) | Limits of Mean Bending Strenght ($kN/mm^2$) |
|---|---|---|---|---|
| 1 | D18 | $f_{m,k} \leq 18$ | $\rho_k \leq 475$ | $E_{mean} \leq 9.5$ |
| 2 | D24 | $18 < f_{m,k} \leq 24$ | $475 < \rho_k \leq 485$ | $9.5 < E_{mean} \leq 10$ |
| 3 | D30 | $24 < f_{m,k} \leq 30$ | $485 < \rho_k \leq 640$ | $9.5 < E_{mean} \leq 10$ |
| 4 | D35 | $30 < f_{m,k} \leq 35$ | $640 < \rho_k \leq 670$ | $9.5 < E_{mean} \leq 10$ |
| 5 | D40 | $35 < f_{m,k} \leq 40$ | $670 < \rho_k \leq 700$ | $10 < E_{mean} \leq 11$ |
| 6 | D50 | $40 < f_{m,k} \leq 50$ | $700 < \rho_k \leq 780$ | $11 < E_{mean} \leq 14$ |
| 7 | D60 | $50 < f_{m,k} \leq 60$ | $780 < \rho_k \leq 840$ | $14 < E_{mean} \leq 17$ |
| 8 | D70 | $70 \leq f_{m,k}$ | $1080 \leq \rho_k$ | $20 \leq E_{mean}$ |

Table 4.36 Proposed Allocations of Strength Class for *Alstonia boonei*

| Reference Material Properties | 12% Adjusted Values | 18% MC djusted Values | NCP 2 Dry Basic Stress (!8% MC) | NCP 2 Strength Class | Allocated Strength Class (EN 338, 2009) |
|---|---|---|---|---|---|
| Bending Strength ($N/mm^2$) | 39.36 (Characteristic) | 46.15 (Mean) | 9.0 (factored) | N7 | D18 |
| Density ($kg/m^3$) | 345.27 (Mean) | 357.0 (mean) | 432.0 (factored) | | |
| Modulus of Elasticity ($kN/mm^2$) | 9.34 (Mean) | 8.43 (mean) | 5.30 (factored) | | |

Table 4.37 Proposed Allocations of Strength Class for *Triplochiton Scleroxylon*

| Reference Material Properties | 12% Adjusted Values | 18% MC Adjusted Values | NCP 2 Dry Basic Stress (!8% MC) | NCP 3 Strength Class | Allocated Strength Class (EN 338, 2009) |
|---|---|---|---|---|---|
| Bending Strength ($N/mm^2$) | 45.63 (Characteristic) | 50.41 (mean) | 11.2 (factored) | N6 | D18 |
| Density ($kg/m^3$) | 371.07 (Mean) | 383.67 (mean) | 432.0 (factored) | | |
| Modulus of Elasticity ($kN/mm^2$) | 6.59 (Mean) | 5.95 (mean) | 5.30 (factores) | | |

Table 4.38 Proposed Allocations of Strength Class for *Terminalia Ivorensis*

| Reference Material Properties | 12% Adjusted Values | 18% MC adjusted Values | NCP 2 Dry Basic Stress (!8% MC) | NCP 2 Strength Class | Allocated Strength Class (EN 338, 2009) |
|---|---|---|---|---|---|
| Bending Strength ($N/mm^2$) | 50.13 (Characteristic) | 71.79 (mean) | 14.0 (factored) | N4 | D18 |
| Density ($kg/m^3$) | 345.27 (Mean) | 470.41 (mean) | 576 (factored) | | |
| Modulus of Elasticity ($kN/mm^2$) | 13.56 (Mean) | 12.25 (mean) | 9.0 (factored) | | |

Table 4.39 Proposed Allocations of Strength Class for *Terminalia Superba*

| Reference Material Properties | 12% Adjusted Values | 18% MC djusted Values | NCP 2 Dry Basic Stress (!8% MC) | NCP 2 Strength Class | Allocated Strength Class (EN 338, 2009) |
|---|---|---|---|---|---|
| Bending Strength ($N/mm^2$) | 68.67 (Characteristic) | 91.98 (mean) | 11.2 (facored) | N6 | D24 |
| Density ($kg/m^3$) | 509.97 (mean) | 527.29 (mean) | 464.0 (factored) | | |
| Modulus of Elasticity ($kN/mm^2$) | 13.32 (Mean) | 12.03 (mean) | 7.10 (factored) | | |

Table 4.40 Proposed Allocations of Strength Class for *Lophira alata*

| Reference Material Properties | 12% Adjusted Values | 18% MC djusted Values | NCP 2 Dry Basic Stress (!8% MC) | NCP 2 Strength Class | Allocated Strength Class (EN 338, 2009) |
|---|---|---|---|---|---|
| Bending Strength ($N/mm^2$) | 83.54 (Characteristic) | 86.62 (mean) | 35.5 (facored) | N1 | D60 |
| Density ($kg/m^3$) | 940.156 (mean) | 527.29 (mean) | 1136 (factored) | | |
| Modulus of Elasticity ($kN/mm^2$) | 23.08 (mean) | 20.08 (mean) | 15.0 (factored) | | |

## 4.2 Reliability Analysis Results and Discussion

### 4.2.1 Component and System Reliability of Frame

The portal frame was analysed using the probabilistic models formulated in chapter three. Reliability analysis was performed for the identification of those parts of the structure essential for the safety, and evaluate the sensitivity of structural safety with respect to the uncertainties included in the probabilistic model. For each of the thirty two failure modes formulated in chapter four, modal reliability index $\beta_i$ as well as system reliability index $\beta^s$ were estimated using genetic algorithms (GA) based first order reliability mrthod (FORM) implemented through the MATLAB-program 'genehunter.m', developed in the study.

The modal reliability indices $\beta_i$ given in Tablee 4.41 and 4.42 indicated that, failure mode 15 is the most significant (predominant) mode of failure. This failure mode correspond to the failure of rafter-column connection.

Comparing the modal reliability indices in Tables 4.41 and 4.42 with the systen reliability indices presented in Table 4.43, it is clear that, the reliability indices of the predominant modes of failure correspond to the system reliability indices.

Three-hinged portal frame is statically determinate (non-redundant). A statically determinate structure is a series system. In the present paper the generalised system reliability is estimated by the Hohenbichler approximation (Madsen *et al,* 1986) using the modal realiability indices. In a series system the failure of one element always lead to

system failure. The occurrence of failure in the predominant mode is therefore expected to resuts to the failure of the portal frame. Since the predominant (critical) safety indices are equivalent to the system safety indices, therefore, it is expected that the safety of a well-designed timber structural system depend on the reliability of its critical mode of failure.

The failure mode corresponding to frame lateral displacemt (sway mode) has also a relatively low reliability index. Also, the failure modes for frame memeber deflection have the highest relibility indices. However, the estimatee for reliability indice for sway nd deflection are strongly related to the choice of criteria for drift limit and limiting deflections respectively. The resulting reliability indiced could therefore be easily regulated.

The reliabilty indices in Tables 4.41 to 4.43, were based on 3.8 target reliability index (Eurocode 0, 2002). The frame was designed for each of the five timber species (*Alstonia boonei, Triplochiton scleroxylon, Terminalia ivorensis, Terminalia superba and Lophira alata*) under investigation. The frame member sections at the eave (location of the most citical mode of failure) was considered as the design criteria. Frame made with *Lophira alata* has the least section at the target reliability of 3.8 (section height = 415mm), this was followed by the frame made with *Terminalia superba* (section height = 500mm), then *Terminalia ivorensis* (section height = 520mm), then *Triplochiton scleroxylon* (section height = 555mm). Frame made with *Alstonia boonei* has the heghest depth requirement (section height = 570mm). This results correspond well with results strength

classification of the timber species presented in chapter three, in which, *Lophira alata* was classifies as D60 based on EN 338 (2009). *Terminalia superba* was classified as D24, *Terminalia ivorensis*, *Triplochiton scleroxylon* and *Alstonia Boonei* were allocated strength D18.

The thirty two failure modes were grouped into eight failure phenomena; compression failure, bending failure, buckling failure, beam-column connection failure, frame base failure, failure due to excessice member deflection, failure due to excessive lateral displacement (sway mode) and failure of the frame apex connection. Consideration of failure phenomena rather than member failure is the practice in Eurocode (Chanakya, 2009). In the old British Standards, structural design are based on structural elements, such as design of beams, design of columns. The Eurocode group all element under prticulur failure phenomena, to avoid un necessary repititions.

Table 4.41 Modal Reliability Indices (Failure mode 1-16)

| Failure Mode. | Depth of Section Corresponding to Target Safety Index (mm) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
| | 570 | 555 | 520 | 500 | 415 |
| 1 | 8.90 | 8.97 | 8.88 | 8.89 | 8.90 |
| 2 | 8.12 | 7.90 | 8.17 | 8.13 | 8.09 |
| 3 | 7.99 | 8.03 | 8.30 | 8.47 | 7.98 |
| 4 | 8.67 | 8.70 | 8.84 | 8.94 | 8.67 |
| 5 | 6.13 | 6.15 | 6.25 | 6.31 | 6.13 |
| 6 | 7.67 | 7.63 | 7.43 | 7.30 | 7.67 |
| 7 | 8.46 | 8.36 | 8.49 | 8.47 | 8.45 |
| 8 | 8.82 | 8.86 | 8.81 | 8.82 | 8.83 |
| 9 | 7.99 | 8.03 | 8.30 | 8.47 | 7.98 |
| 10 | 8.67 | 8.70 | 8.84 | 8.94 | 8.67 |
| 11 | 6.17 | 6.18 | 6.28 | 6.34 | 6.17 |
| 12 | 6.48 | 6.49 | 6.52 | 6.54 | 6.48 |
| 13 | 4.29 | 4.25 | 4.23 | 4.19 | 4.15 |
| 14 | 6.45 | 6.42 | 6.41 | 6.39 | 6.37 |
| 15 | 3.88 | 3.85 | 3.86 | 3.84 | 3.86 |
| 16 | 6.23 | 6.21 | 6.22 | 6.20 | 6.21 |

Table 4.42 Modal Reliability Indices (Failure mode 17 – 32)

| Failure Mode | Depth of Section Corresponding to Target Safety Index (mm) | | | | |
|---|---|---|---|---|---|
| | *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
| | 570 | 555 | 520 | 500 | 415 |
| 17 | 5.73 | 5.71 | 5.72 | 5.70 | 5.73 |
| 18 | 7.41 | 7.39 | 7.40 | 7.38 | 7.39 |
| 19 | 4.82 | 4.83 | 4.90 | 4.91 | 5.05 |
| 20 | 6.79 | 6.80 | 6.84 | 6.85 | 6.94 |
| 21 | 5.73 | 5.73 | 6.02 | 6.25 | 5.00 |
| 22 | 6.57 | 6.54 | 6.70 | 6.85 | 5.61 |
| 23 | 7.75 | 7.72 | 7.79 | 7.87 | 7.11 |
| 24 | 6.24 | 6.22 | 6.36 | 6.51 | 5.33 |
| 25 | 8.84 | 8.84 | 8.82 | 8.80 | 8.88 |
| 26 | 8.29 | 8.30 | 8.38 | 8.43 | 8.19 |
| 27 | 9.21 | 9.21 | 9.16 | 9.12 | 9.47 |
| 28 | 7.16 | 7.16 | 7.30 | 7.42 | 6.48 |
| 29 | 13.89 | 11.03 | 14.50 | 13.65 | 13.59 |
| 30 | 15.41 | 13.73 | 15.75 | 15.28 | 15.25 |
| 31 | 5.73 | 5.25 | 5.82 | 5.70 | 7.78 |
| 32 | 12.39 | 12.35 | 12.47 | 12.59 | 11.37 |

Table 4.43 System Reliability Indices

| *Alstonia Boonei* (Ahun) | *Triplochiton Scleroxylon* (Obeche) | *Terminalia Ivorensis* (Black Afara) | *Terminalia Superba* (White Afara) | *Lophira Alata* (Ekki) |
|---|---|---|---|---|
| 2.93 | 3.86 | 3.83 | 3.84 | 3.86 |

### 4.2.2 Sensitivity Analysis

In order to analyse the sensitivity of the frame reliability with respect to stochastic as well as deterministic parameters, a sensitivity analysis has been performed. Figures 4.26 to 4.30 shows the sensitivity of the reliability indices of each of the phenomenal failure modes to variations of critical section modulus. This sensitivity measure (also known as the reliability elasticity coefficient), gives the change in the reliability indices due changed in critical section modulus.

Fig. 4.26, 4.27, 4.28, 4.29 and 4.30 are for frames made with *Alstonia boonei*, *Triplochiton scleroxylon*, *Terminali ivorensis*, *Terminalia superba* and *Lophira alata* timber species respectively. For all the plot, safety indices for each of the considered failure modes increases with the increase in the section modulus. It is also observed in the results presented in table 4.26, the resuts in the plots, indicated that, failure of rafter-column connection is predominant, with the least safety index at every point.

The failure mode next to the predominant is the sway mode. Its dominance became clearly effective beyond a section modulus of 4500 $cm^3$. The margin between the predominant mode and sway mode reliabilty is enormous. In Fig. 4.26, At the least

section modulus in the plot (3000 $cm^3$), the safety index of the predominant mode of failure is -2.8, while the safety index of the sway mode is 2.4.

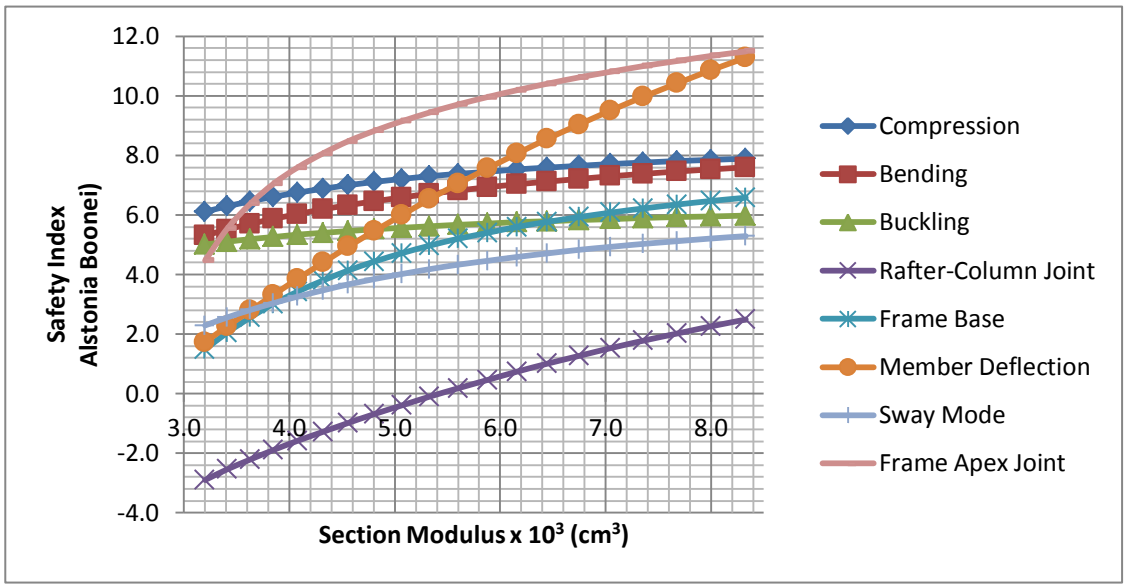



Fig. 4.26 Variation of safety index with section modulus for various failure modes (Alstonia boonei)

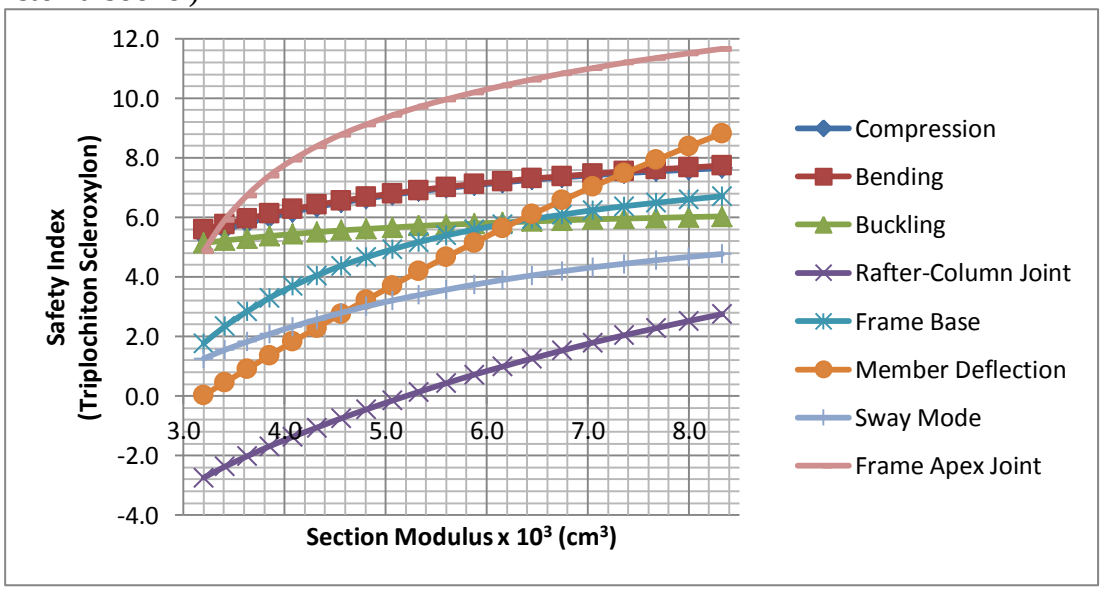



Fig. 4.27 Variation of safety index with section modulus for various failure modes (Triplochiton scleroxylon)

Failure mode dominance change in the plots were also clearly observed. For instance in Fig. 4.26, failure mode frame base, is dominant to all other modes except for the predominant and sway mode between section modulus of 3000cm$^3$ and 6500cm$^3$. Beyond section modulus of 6000cm$^3$, buckling mode subordinated the frame base failure mode. Also deflection mode is next to the predominant failure mode at section modulus between 3000cm$^3$ and 3800cm$^3$. Beyond section modulus of 3800cm$^3$, the reliability for deflection continue to rise rapidly and exceeded the reliability of all the other mode except for the frame apex connection. The frame apex connection failure mode was found to be less significant.

It is clear that a significant variation in reliability levels exists between the eight design tasks considered. Excessive reliability is present for the frame-apex connection, compression and deflection modes. Component based design of the frame members to meet pre-defined target reliability will result to error. Target safety index should therefore be the system safety index. If the Target safety index is achieved, the safety of the frame for all the other failure modes wuold be quaranteed.

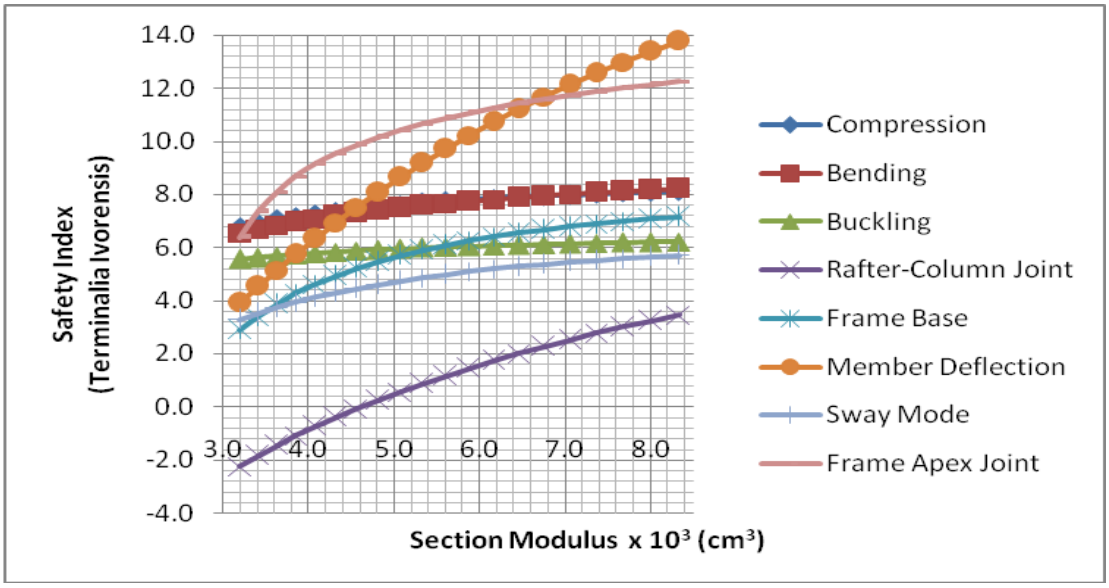



Fig. 4.28 Variation of safety index with section modulus for various failure modes (Terminalia ivorensis)

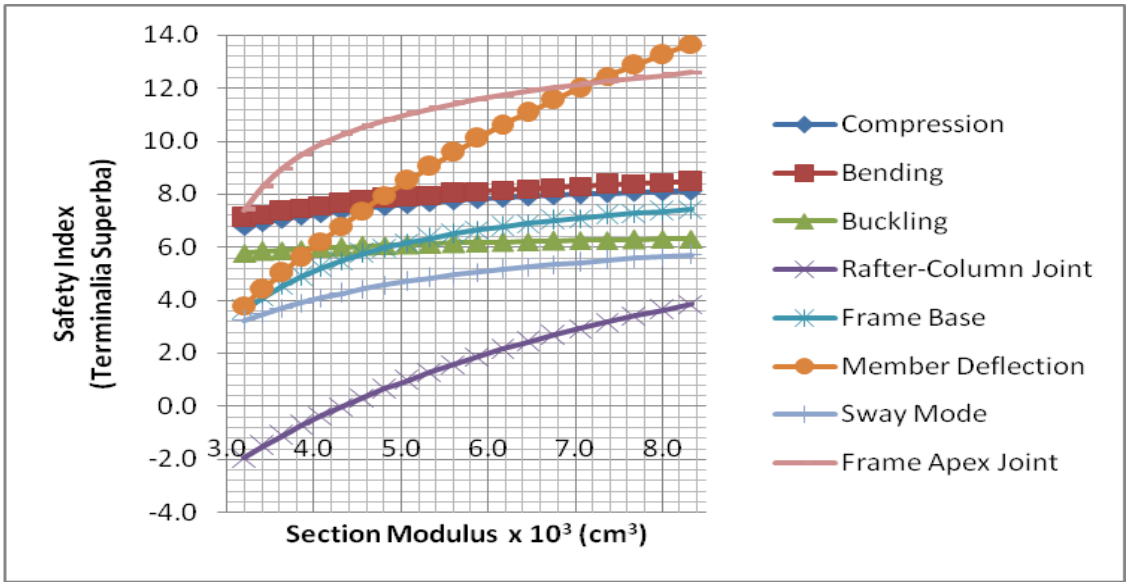



Fig. 4.29 Variation of safety index with section modulus for various failure modes (Terminalia superba)

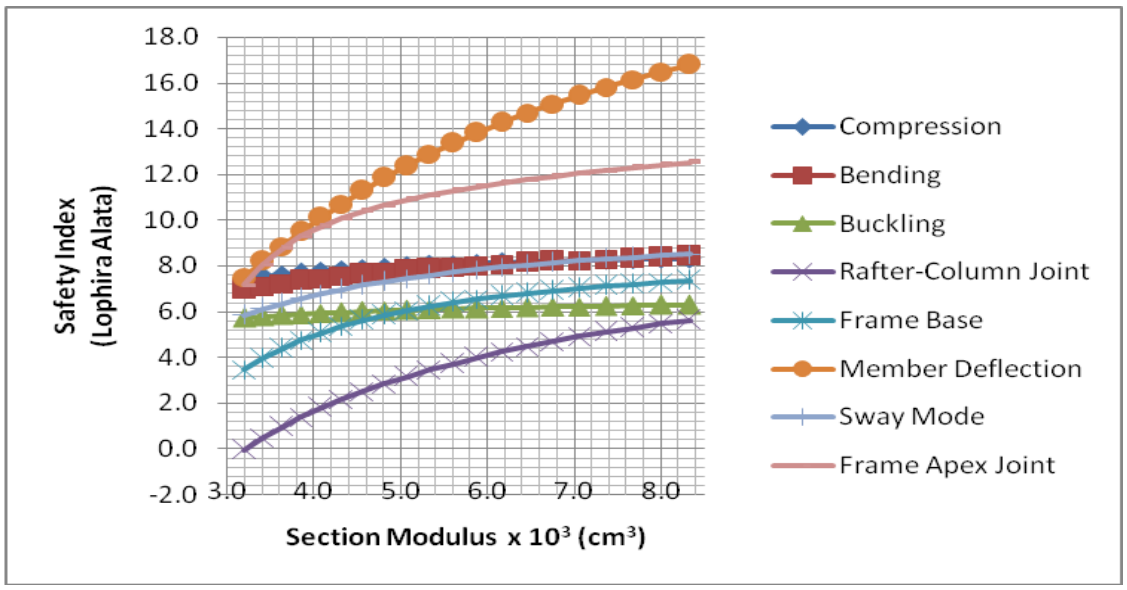



Fig. 4.30 Variation of safety index with section modulus for various failure modes (Lophira alata)

Fig. 4.31 show the variation of the safety index with critical section modulus for frames made with each of the five timber species. Safety indices increase steadily with increasing section modulus. Considering a target safety index of 2.0, the required section modulus for *Lophira alata* frame was 4200cm$^3$. For *Alstonia boonei*, *Triplochiton scleroxylon, Terminalia Ivorensis and Terminalia superba*, the required section modulus are 6000cm$^3$, 6500cm$^3$, 7400cm$^3$, and 8000cm$^3$ respectively. The ratio of the optimal material required for frame made with *Alstonia boonei* frame, *Triplochton scleroxylon* frame, *Terminala ivorensis* frame and *Terminalia superba* frame to the frme made with Lophira alata are 1.90, 1,76, 1.55 and 1.43 respectively.

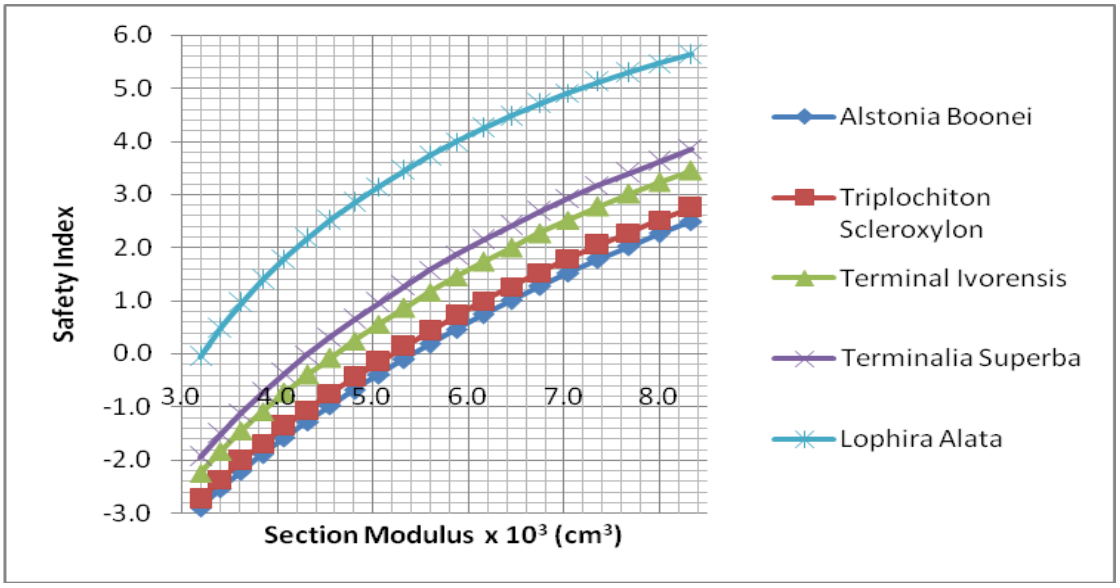



Fig. 4.31 Predominant mode reliability for various tim ber species

### 4.2.3 Reliability of the Portal Frame in Fire

Affordability, construction speed, favourable physical-mechanical, ecological and aesthetical properties of wood are only some of the aspects why wood is more and more frequently used for the construction of residential buildings around the world (Krystyna and Antonin, 2010). First of all, mainly with roof structures in Nigeria, it is necessary that attention is paid to meeting all fire regulations and the correct design of wood elements as regard fire effects.

Wood elements exposed to fire and a temperature of about $300^0$C flame up on their surfaces and in the beginning they burn relatively intensely. Step-by-step, a char layer occurs. The layer show about 6 times better heat-insulation properties that grown timber (Krystyna and Antonin, 2010). Reliability analysis was carried out for the frame under fire consideration. The analysis was done based on the methods given in Eurocode 5 part

1-2 (2004) on loading condition under fire and on the charring rate of the wood section. The charring rate of the most critical section $d_{crit}$ was calculated by the charring rate equation of the Eurocode 5 as:

$$d_{crit} = \beta_n t + k_0 d_0 \quad (4.1)$$

where, $\beta_n$ is the charring rate, $K_0$ and $d_0$ are parameters defined in the code.

The results are presented in Fig. 4.32 to 4.40. Fig. 4.32, shows the variation of safety index with fire exposure time for the failure modes due to axial compression. The frame was subjected to one hour fire exposure time. For all the five timber species whose material properties were considered in the limit state functions, safety index decrease with incresing fire exposure time.

Between 0 and about 30 minutes of exposure to fire, safety indices were slightly decreasing. Likewise, the safety indices for frame made with each of the five timber species are very close. Beyond 30 minutes of fire exposure, the decrease in safety levels were observed to be rapid. For instance, for frame made with *Alstonia boonei* timber, safety index decreased from 7.6 to 7.0, between 0 and 30 minutes. However, between 30 minutes and I hour of exposure to fire, safety index changed from 7.0 to 3.0. The reason behind this trend, is that, the charring rate given in the Eurocode 5 is a function of time. At low fire exposore time, the decrease in the load carrying capacity is low. However, as the fire exposure time became high, the decrease in the load carrying capacity is also high. For the compression failure, the specic frame under investigation is within safe

domain for one hour fire exposure time. Compression is therefore not a possible cause of failure of the frame in fire.

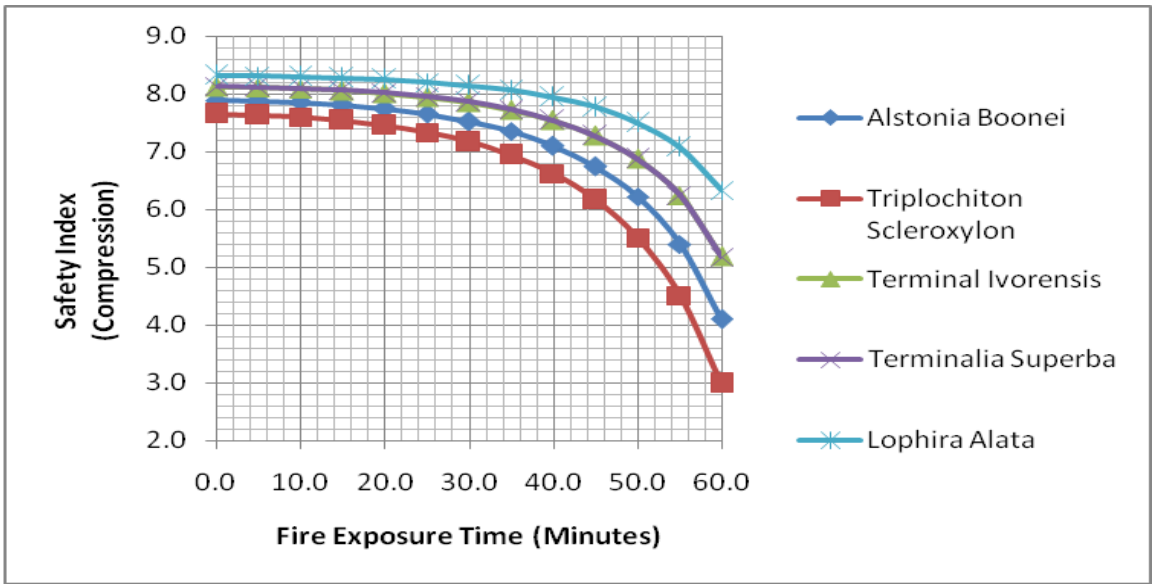



Fig. 4.32 Variation of safety index with fire exposure time for compression failure mode

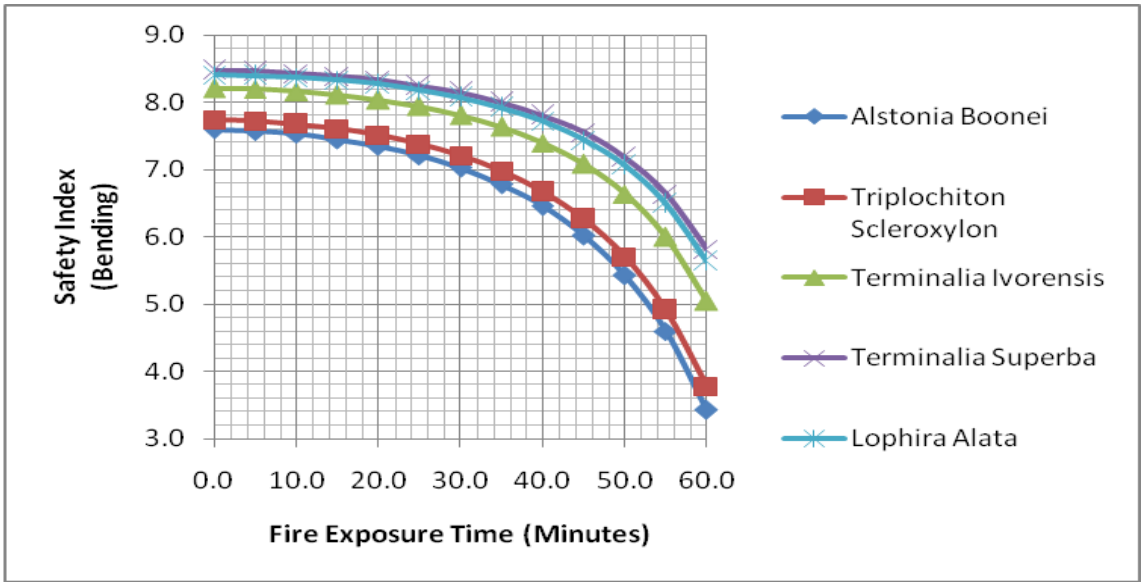



Fig. 4.33 Variation of safety index with fire exposure time for bending failure mode

Fig. 4.33, shows the variation of safety index with fire exposure time for bending modes of failure. The behaviour of the frame in bending is similar to compression. However, Under compression Frame made with *Triplochiton Scleroxylom* perform worse than frame made with the other four timber species, however, for the bending failure mode frame made with Alstonia Boonei specie performed worse.

Fig. 4.34 show the variation of safety index with fire exposure time for buckling modes of failure. Buckling modes of failure is more critical than the compression and bending modes. For the worst performing frame, the safety index at 0 exposure to is 6.0, and after 1 hour of fire exposure the safety index became 4.1. Buckling failure in an instability failure. Its variation is more dependent on member geometery, particulurly member slenderness. Since, increse in fire results to reduced cross section, then increase in fire exposure time leads to decrease in buckling capacity of frame members.

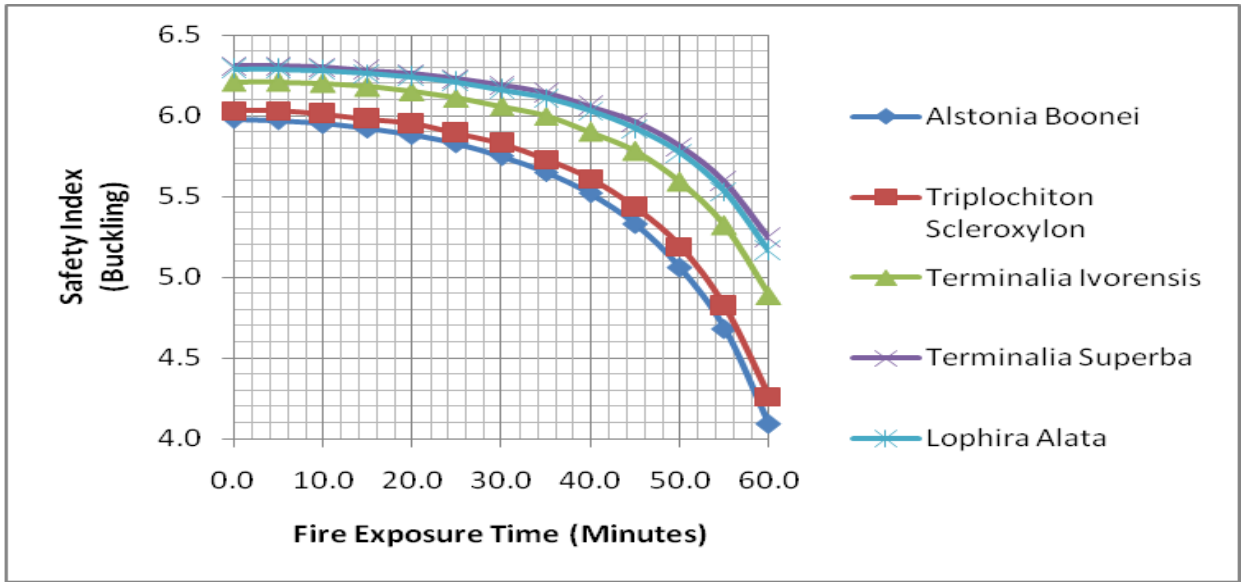



Fig. 4.34 Variation of safety index with fire exposure time for buckling failure mode

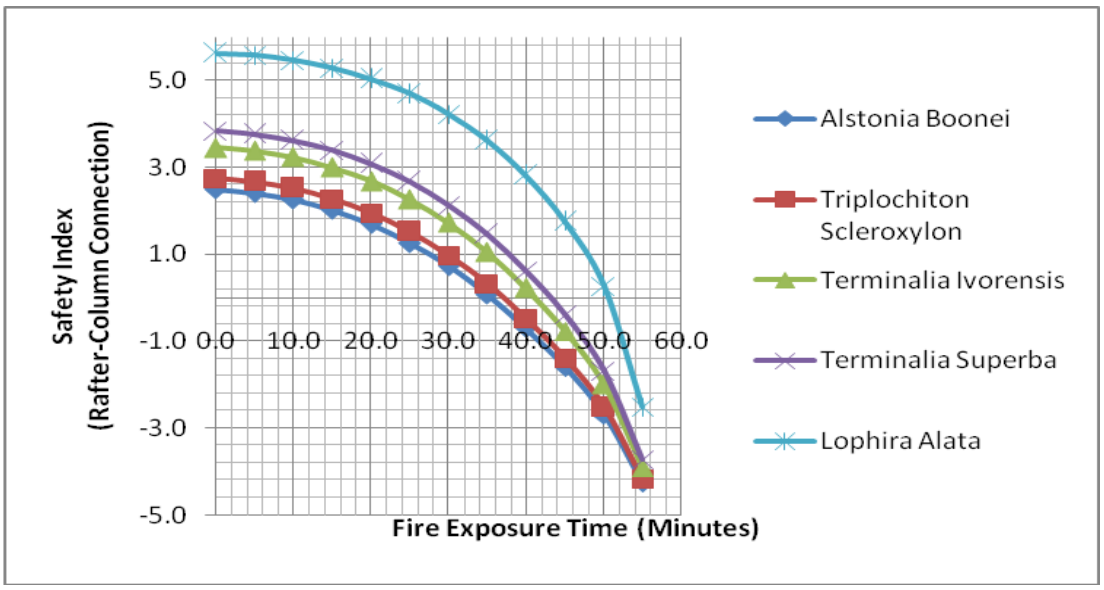



Fig. 4.35 Variation of safety index with fire exposure time for rafter-column connection failure mode

Fig. 4.35 shows the variation of safety index with fire exposure time for the failure modes of frame rafter-column connection. Unlike the compression, bending and buckling failure modes, in which, at lower fire exposure time the safety margin between different timber species were veri small, and decreeased at higher fire expore time, the situation here is otherwise. At low fire exposure time, the safety index is higher for frame made with lophira alata timber specie than for frame made with the other species. As the fire exposure time became high, the margin of safety gradually narrowed. At 0 fire exposure time, safety index for lophira alata frame is 5.5 as against 3.8, 3.6, 2.9 and 2.2 respectively for *Terminalia superba, Terminalia ivorensis, Triplochton scleroxylon and Alstonia boonei frame. Lophira alata* frame failed at 52 minutes after the commencement of fire. However, the failure of Triplochton scleroxylon and Alstonia boonei frames occurred at 40 minutes exposure time, while for Terminalia Ivorensis and

Terminalia superba, failure occurred at 45 minutes and 47 minutes fire expoure time respectively. In coclusion one can say the performance of thet moment resisting connection between rafter and column for frame made with *Lophira alata* is very high.

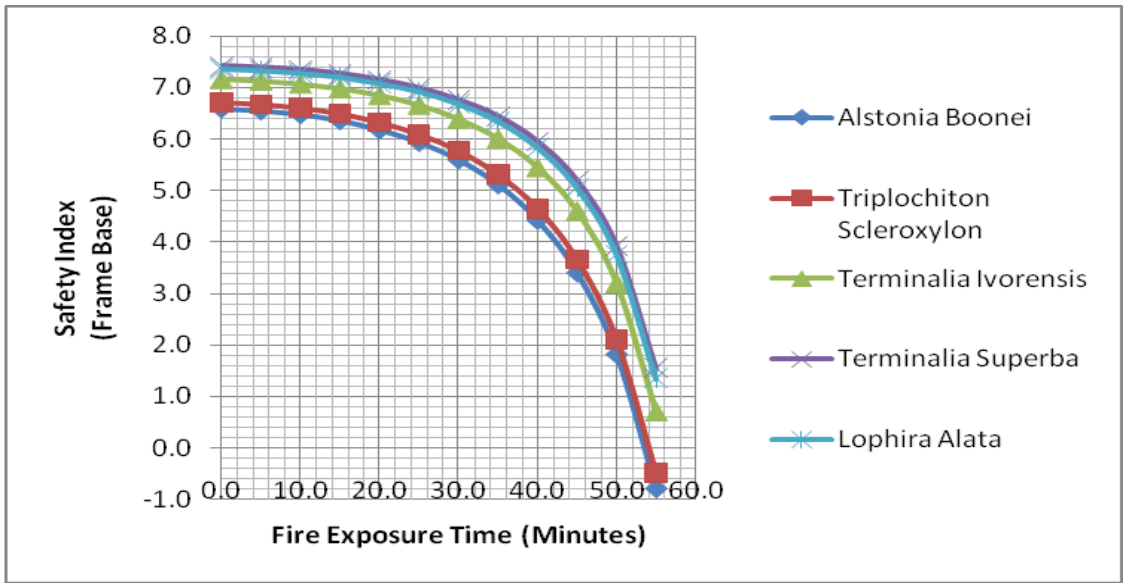



Fig. 4.36 Variation of safety index with fire exposure time for frame base failure mode

Fig. 4.36 shows the variation of safety index with fire exposure time. At low fire exposure time, the safety of the frame at the base is similar to what were observed fo compression, bending and buckling modes of failure. However, after one hoour, while the frame is in safe domain for compression and bending, at frame based the frame approach failure domain, asymptotically. With the trend observed for rafter-column connection and the frme base, it is clear that connection bfailure in highly critical for portal frame

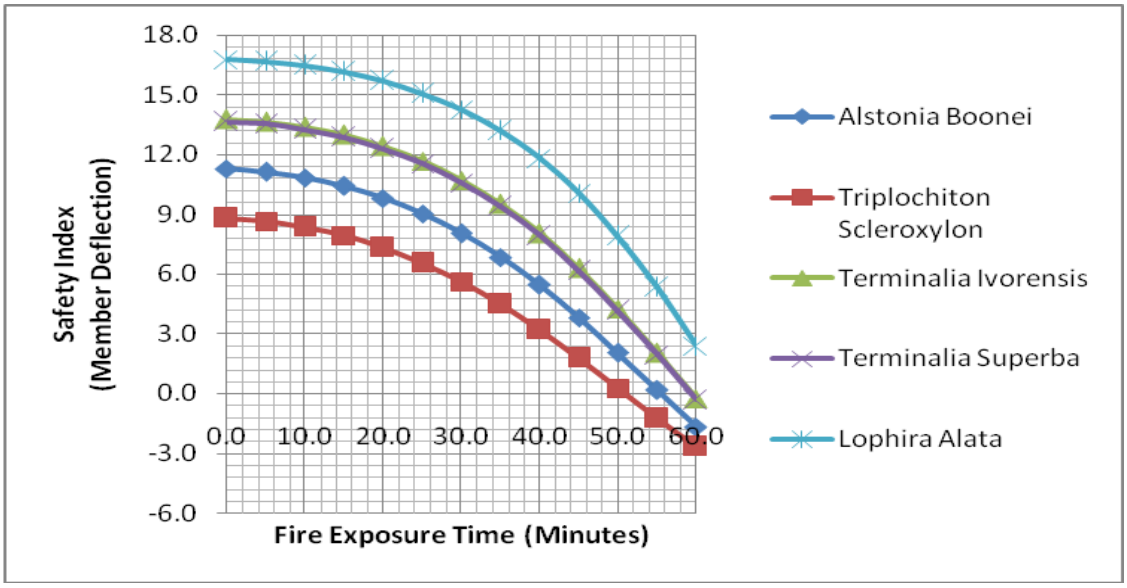



Fig. 4.37 Variation of safety index with fire exposure time for member deflection failure mode

Fig. 4.37 is a plot showing the variation of safety index witrh fire exposure time for deflection failure mode. Deflection capacity of material is a function of its stifness. This is clear, as the Lophira alata specie has the highest elastic modulus, the safety index for lophira alata frame is high. As observed on the plot, the safety index at 0 minutes exposure time is 17.0 for lophira alata frame, 13.0 for terminalia ivorensis and terminalia superba frame, 10..5 for Triplochiton scleroxylon frame and 9.0 for Alstonia boonei frame. As the fire exposure time in creases, a non-linear but continuous decrease in safety index was observed. At 55 minutes fire expusure time, Alstonia boonei frame failed. What is observed on Firg 4.37, indicated that without threat by fire, the least critical mode of failure is conding, considering the very high safety level, with safety indices in the order of 13 to 18. However as the frame is blought under fire, the safety level drastically decreased with increasing time of fire exposure. The safety index approaches

0 whe the fire exposure time approaches 1 hour fire exposure time. This implied that, with this trend, as the fire exposure exposure time is further increased, deflelection is likely to be the governing mode of failure of the portal frame.

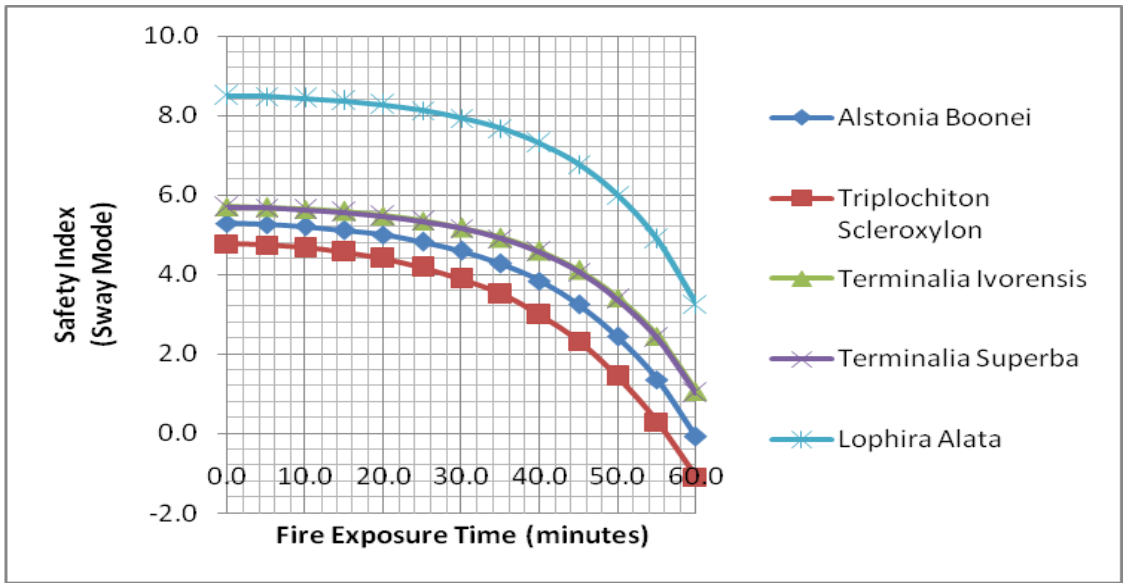



Fig. 4.38 Variation of safety index with fire exposure time for frme sway mode

Fig. 4.38 is the variation of safety index with fire exposure time for sway mode. It is clear from the plots that high strength timber results to high safety level to frame subjected to excessive lateral displacement. For instance, the Lophira alata frame has a safety index of 3.2 after one hour of fire. On the other hand, frames made with Terminalia ivorensis and Terminalia superba have safety index of 1.0, at one hour fire exposure. Alstonia boonei and Triplochiton scleroxylon frames have safety index of 0 and -1.0 respectively, after being exposed to fire for one hour.

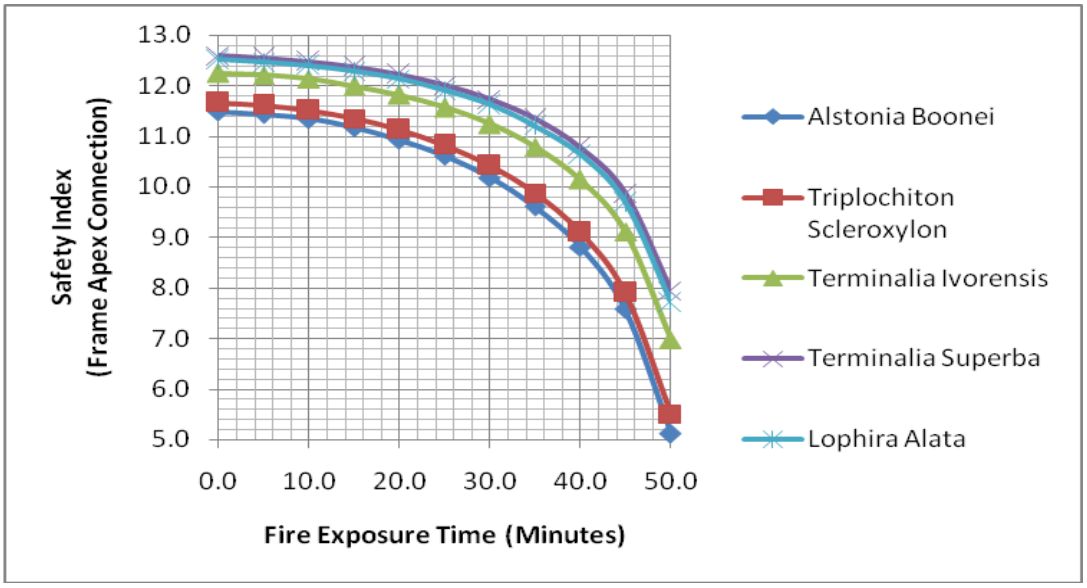



Fig. 4.39 Variation of safety index with fire exposure time for frame apex connection failure mode

Fig. 4.39 shows the variation of safety index with fire exposure time for the failure of frame apex connection. The frame lies on safe domain with one hour fire exposure irrespective of the timber specie. The variation of the safety index with fire exposure time is non linear. The least safety index at the beginning of the fire is 11.5. This decreases continously up to 5.5 after one hour in fire.

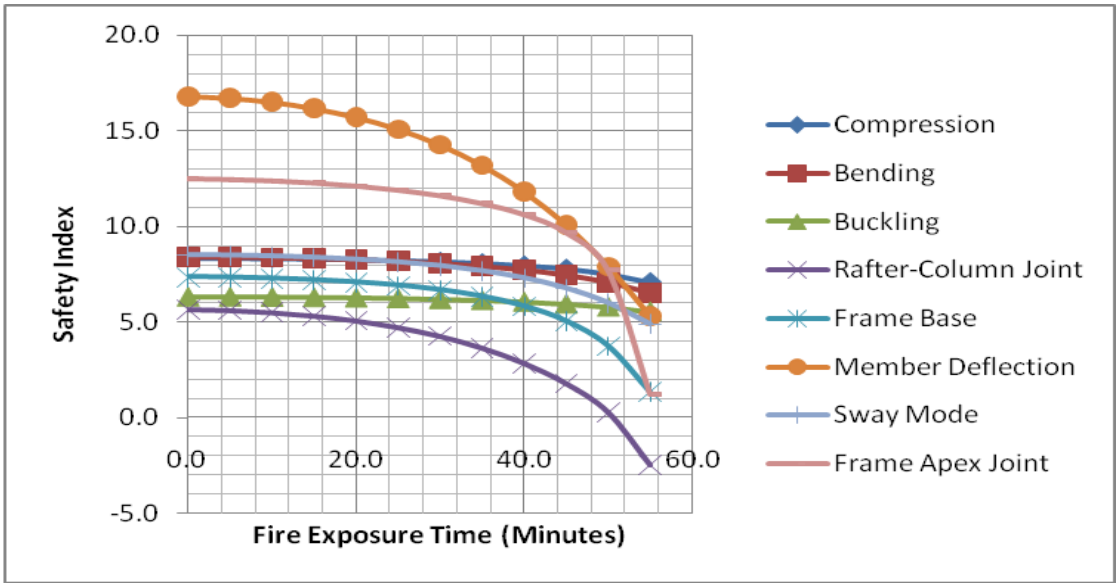



Fig. 4.40 Variation of safety index with fire exposure time for various failure modes

The safey levels achieved by all the considered failure modes in fire are shown on Fig. 4.40. Throughout the fire time, the frame rafter to column connection was critical. Member deflection mode was found to be least critical at fire exposure time less than 50 minutes. After 50 minutes of exposure to fire, compression and bending modes became less critical. Deflection mode became the second most critical mode of failure after 56 minutes of exposure to fire.

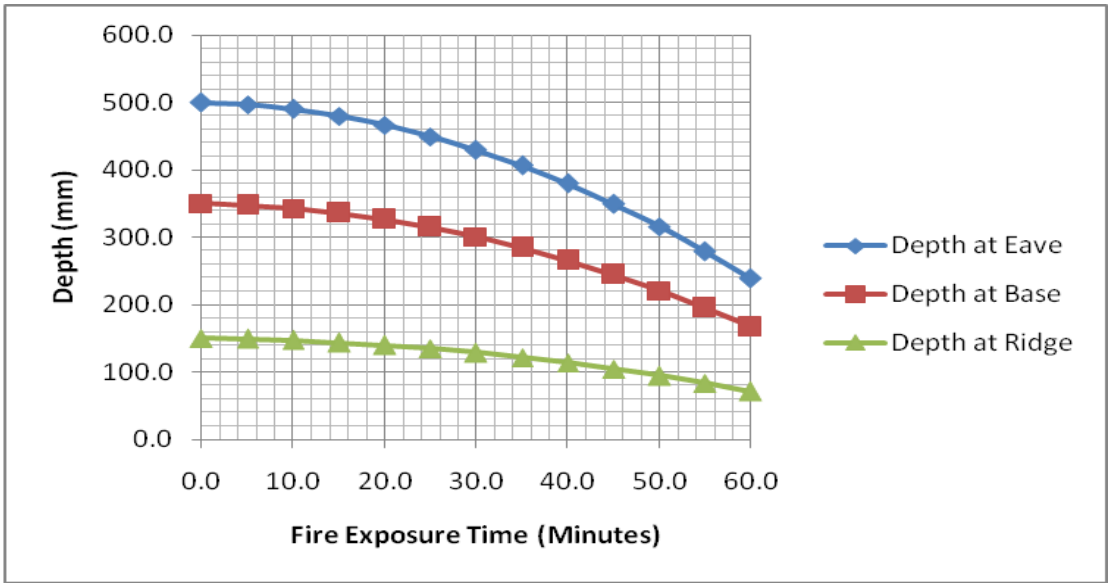



Fig. 4.41 Variation of safety index with fire exposure time for frame at various frame sections

Fig. 4.41 shows the variation of section height with fire exposure time at different sections of the frame. At the beginning of fire, the margin between the frame section heigh at hauch, apex and base are large. The margin continue narrowing during the fire. This trend is clear, as the frame section modulus if a function of the section first moment of area. At higher depth, the first moment of area is high for the large un charred section. As the fire time increses, so does the sections decreses in proportion to the charring rate prescribed in the Eurocode 5. The results obtained in this research agreed with the work presented in Sholadoye (2008).

### 4.2.4 Reliability Analysis of the Frame Subjected to Wind Action

Fig. 4.42 shows the variation of safety index with wind speed for the various modes of failure under investigation. It is clear from the plot thar four modes of failure (rafter-

column joint, sway mode, frame base and frame apex joint) are more sensitive to variation of wind speed in a wind critical frame; that is frame located in a wind prone zones. Deflection mode of failure is not sensitive to variation of wind speed. Buckling mode was observed to be the most significat at wind speed ranging from 5m/s to 20m/s. Beyond a wind speed of 20m/s, the rafter-column connection failure mode assumed predominance.

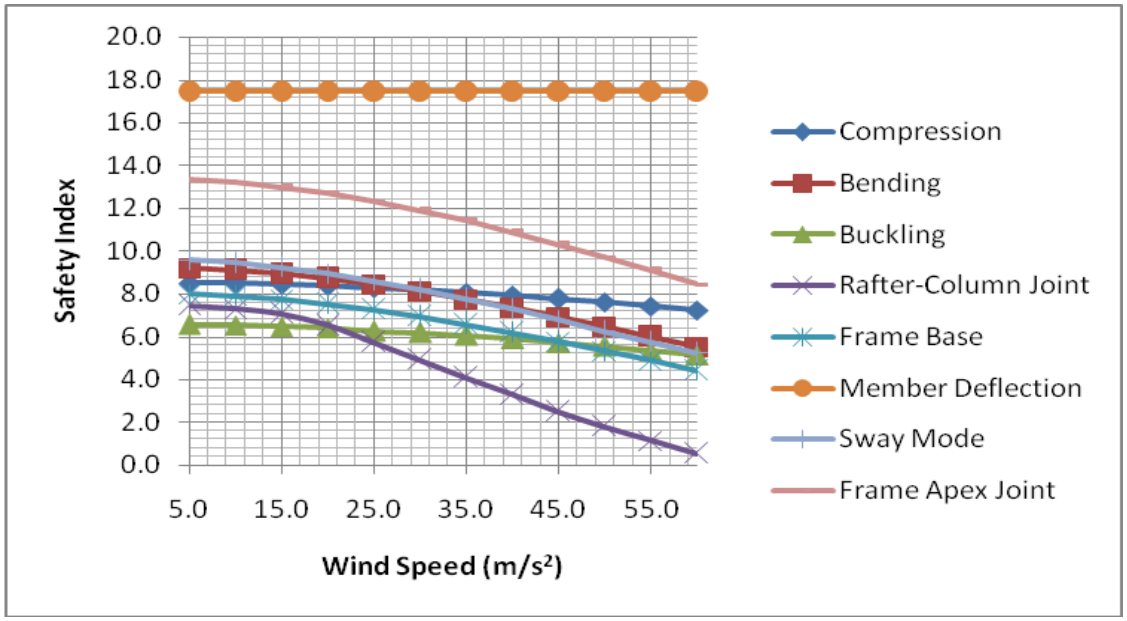



Fig. 4.42 Variation of Safety Index with Wind Speed

### 4.2.5. Effect of Frame Rafter Slope on the Frame Reliability

Variation of safetem reliability with sritical section modulus for various rafter slopes, was presented in Fig. 4.43. The plots showed that slope of rafter in a three-hinge timber portal frame has significant effect on the system reliability of timber portal frame. For instace the variation of system safety index for rafter slope of $5^0$ ranges from -2.0 to 4.0.

However, as the rafter slope is increased to $20^0$, safety index changes from 0 to 5.5. Thhis impled that, for a pitched portal frame, large rafter slope lead to high reliability.

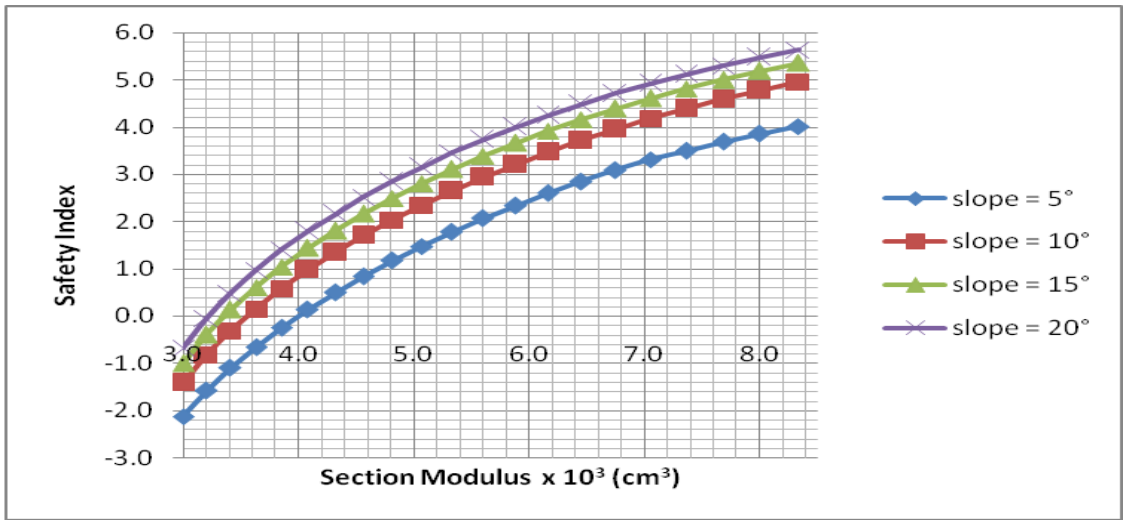



Fig. 4.43 Variation of System Safety Index with Critical Section Modulus

### 4.2.6 Sensitivity Analysis due to Change in Frame Size (Default size factor = 1.0)

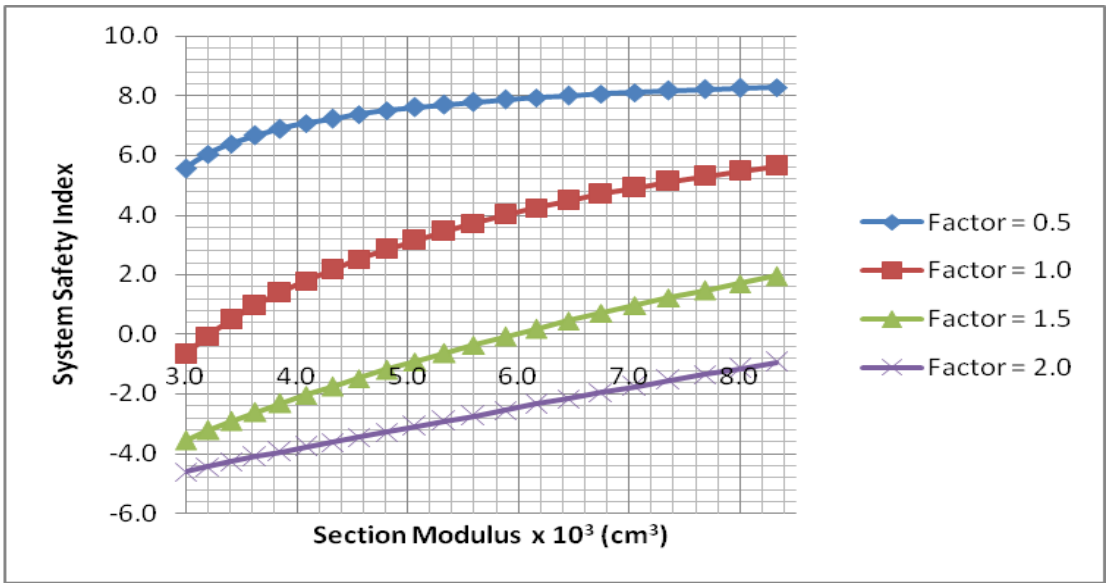



Fig. 4.44 Variation of System Safety Index with Frame Size.

Fig. 4.44 show the plots of the variation of safety index with critical section modulus for various frame size factor. The default size factor was 1.0. A size factor of 2.0 signified that the frame size (height and spa) is double, other thing being equal. From the plot, Changing from default size have a very significant effect on the system reliability of the frame. For instance, at a section modulus of 6000 $cm^3$, the system safety index for frame with the default size, is 4.0. When the deafault size factor is decreased to 0.5, the system reliability increased to 8.0 (100% increase). However, when the default size is incresed to 1.5, the system reliability changed to 0. As the default size if further incresed to 2.0, the system reliability changeg to -2.4 (about 120% decreased in safety level).

### 4.2.7 Effect of Timber Service Class on the Frame Reliability

The effect of service class on the reliability of the portal frame was investigated. The result Is presented in Fig. 4.45. It is clear from the plot that Frame designed with materials of service class 1 and 2 have the same reliability at various design situation. However, frame made with material of service class 3 show drop of system reliability. For instance at section modulus of 6000$cm^3$, the system reliability changed from 4.0 for service classess a nand 2 to 3.2 for service class 3 (about 12% decrease).

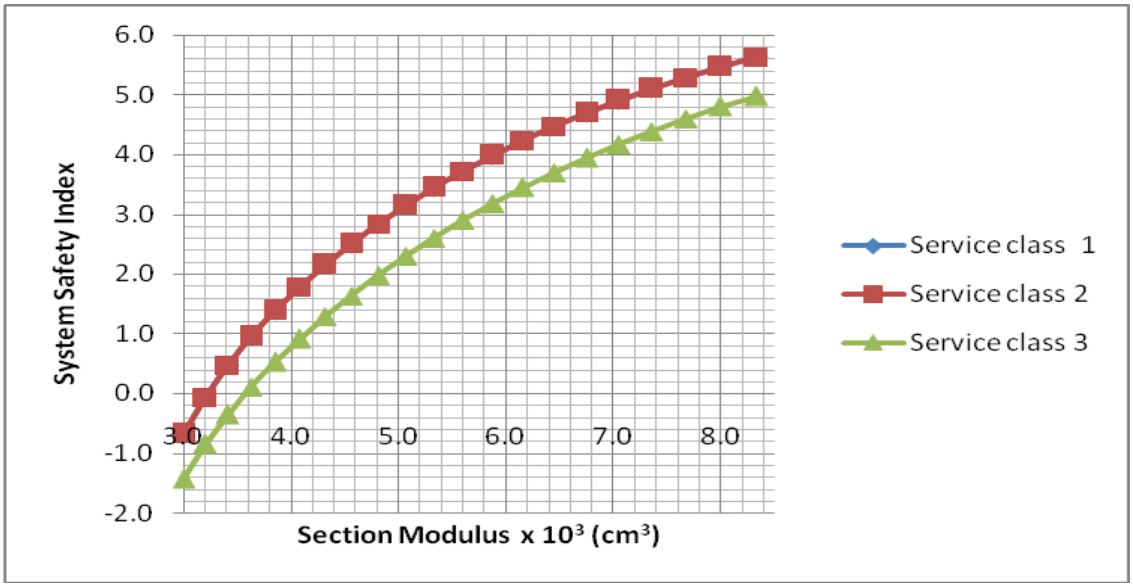



Fig. 4.45 Effect of Service Class on System Reliability

### 4.28 Effect of Frame Member Slenderness

Fig. 4.46 show the plots of the variation of safety index with frame member slenderness ration, for the eight pheni=omenal modes of failure under investigation. For all the failure modes show continous decrease in safety levels with increasing member slenderness. The are failure modes transition as the slenderness ration increases. Buckling and sway mode were predominant at slenderness ratio less than or equal to 16. When the member slenderness exceed 16. The rafter-column mode of failure became predominant. It is also clear from the plot. At the least member slenderbness of 5, the safety ibdex for rafter-column connection is 9, the safety index due to member deflection is as high as 20. However at the highest member slenderness of 70, the safety indices of rafter-column connection, and deflection equalised.

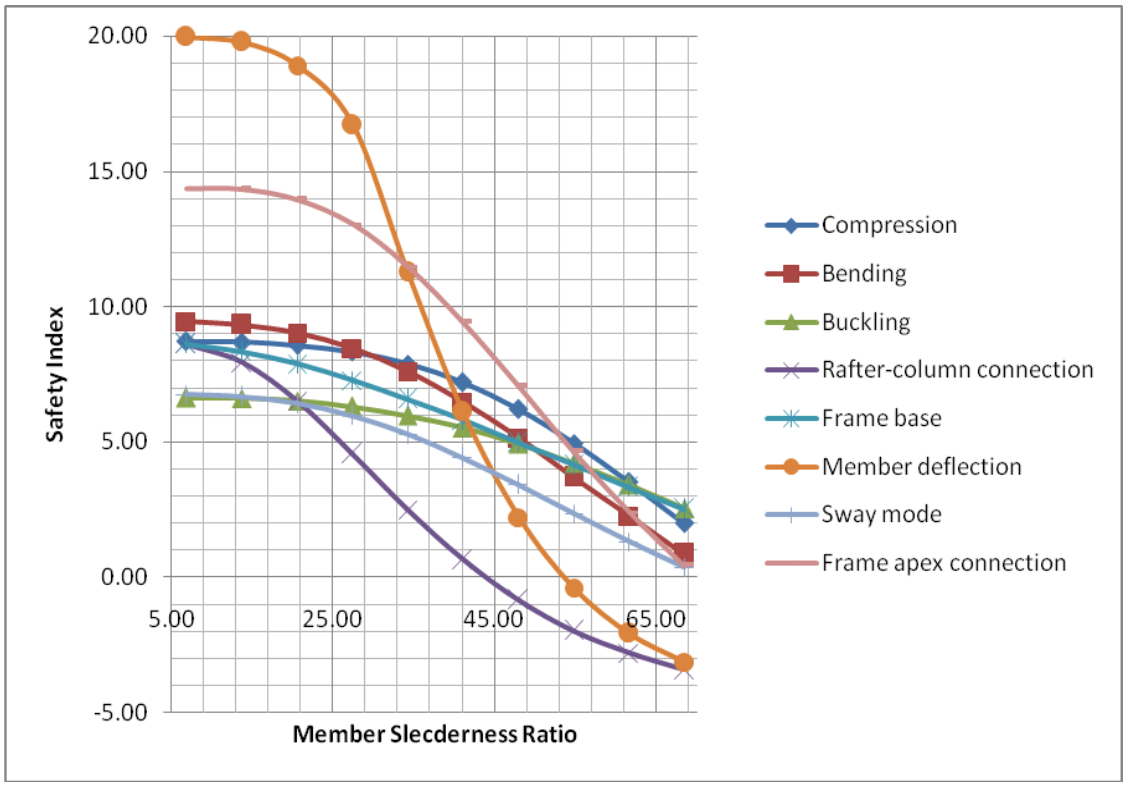



Fig. 4.46 Variation of Safety Index with Member Slenderness Ratio

### 4.2.9 Effect of Drift Limit on Sway Mode Reliability

The most important design parameter for performance base analysis of wood frame structures under lateral loading if the percentage drift. It is therefore the majoir design criteria. Fig 4.47 show the plots of the variation of frame system reliability with critical section height at various dift limitAt drift limit of 0.1%, the frame is on failure domain up to a depth of 460mm. When the drift limit is changed to 0.3, the safety index increased drastically at all depth. For example at section height of 400mm, sysem reliability changed from -2.0 to 2.5. As the drift limit is further increased, safety index changed to 4.0. The change in safety level due to increase in % drift became slight at higher drift

limits, from 1% upward. In conclusion therefore, to achieve high livel of reliability under lateral load, the drift limit must be high.

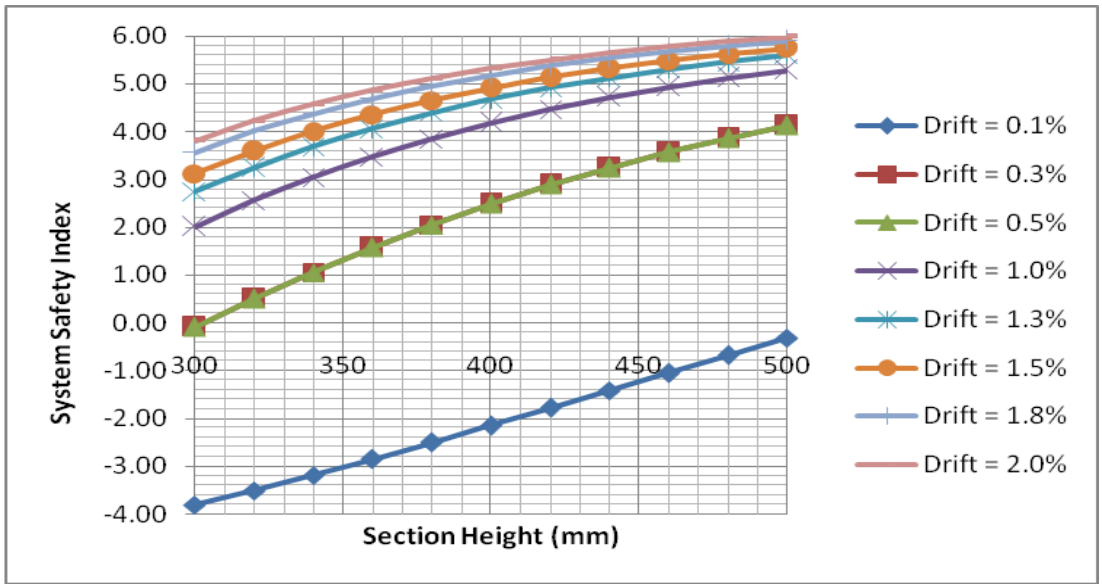



Fig. 4.47 Variation of System Safety Index Frame Critical Depth at Various % Drift

### 4.2.10 Effect of Variable to Total Load Ratio of Frame Safety

For timber structures, the ratio of variable to total load is in the range of 0.2 to 0.8, that is from heavy to light weight (Ranta maunus, 2002). Example, the portal frame analysed in Kitkergaard and Sorensen (2008) has the ratio of variable to total load of 0.56. A light weight timber portal frame was assumed in this study, witrh variable to total load ratio of 0.8. The effect of changing the variable to total load ratio was investigated. The result is presented in Fig. 4.48.

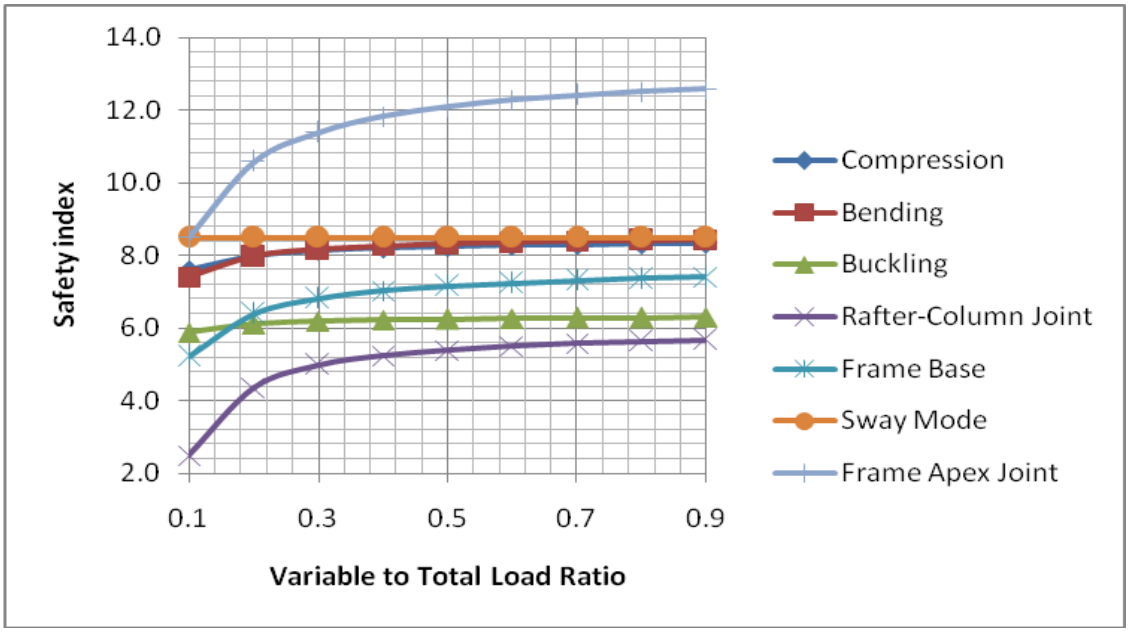



Fig. 4.48. Effect of Change in Load Ratio on the Safety of the Frame.

### 4.2.11 Effect of Load Participation on Frame Safety

The plots of the variation of safety index with permant load contribution factor are shown on Fig. 4.49. Permanent load contribution signifies the contribution of dead load load in the total loading acting on the structure. As against the trends in the previous plot, rafter-column connection mode is only critical ar permannt load contribution between 0.1 and 0.3. However, as the permanent load contribtion exceeds 0.3 (variable load exceeds droped from 0.7), buckling mode of failure became ptredominant. Deflection mode is least critical and less sensitive to the variation of permanent load contribution.

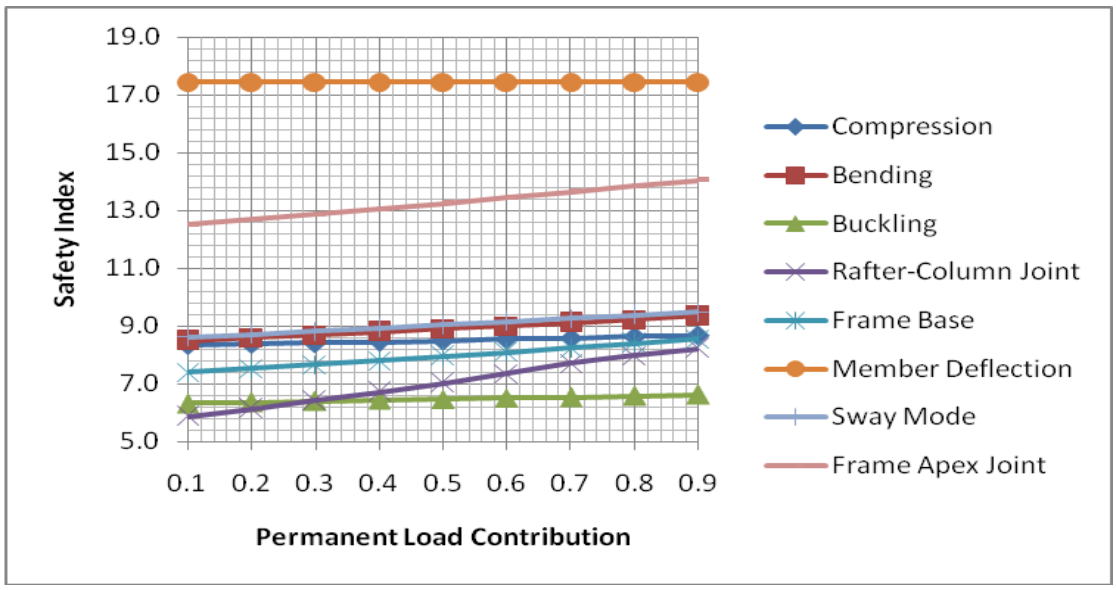



Fig. 4.49 Variation of Safety Index with Permanet Loa Contribution

### 4.2.12 Effect of Change in Distribution Model of Material Property

Fig. 4.50 shows the variation of system safety index with critical section modulus for the five different distribution models. The default distribution model adopted for the strength and stiffness of material froperties is the lognormal distribution. From the plots, safety index increase with increase section modulus. At section modulus of say 3300cm$^3$, changing distribution model from lognormal to gumberl resulted to increase in safety leve from 8.5 to 9.2 (8.5% increase). When the material distribution model is changed to Normal distribution, the safety index dropped from 8.5 to 7.8 (decreased by 8.2%). When the material distribution model is changed to weibull distribution the safety index is further dropped to 7.4. (decreased by 12.9%).

A very large drop in safety level is observed when the material distribution model is changed from lognormal to frechet distribution. The safety index changed from 8.5 to 2.8

(67.1%). The resuts obtained indicate a veryb serious effect of distribution model selection forv the purposed of structural reliability analysis. For instance, the cost of safety increased sis times when the distribution model of material properties is changed from the default lognormal to frechet.

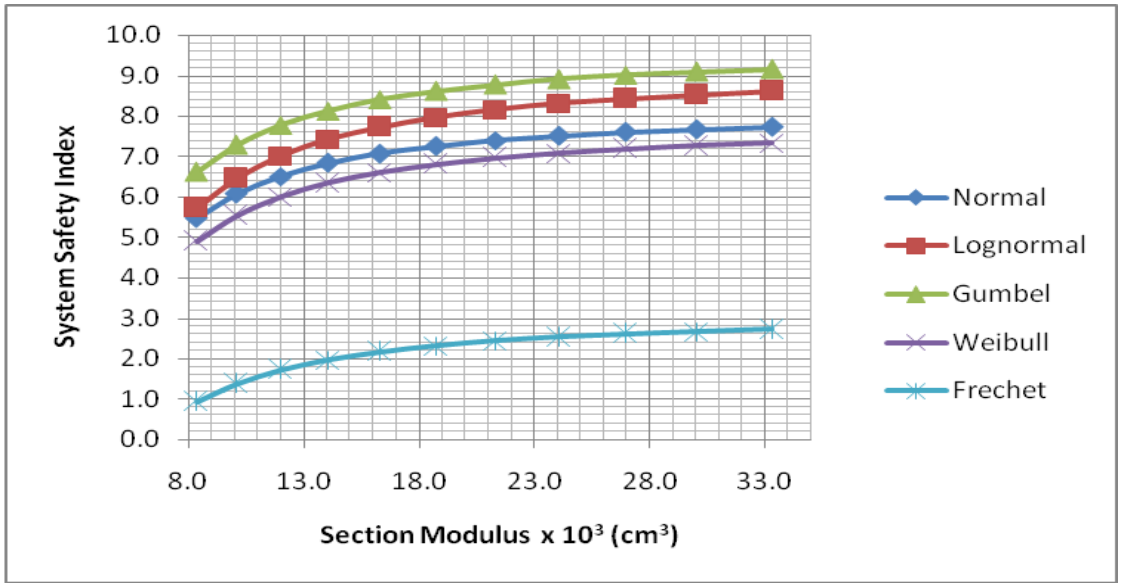



Fig. 4.50 Variation of System Safety Index with Section Modulus For the various Material Distribution Model

### 4.2.13 Effect of Change in Distribution Model of Load Parameters

Fig. 4.51, the effect of changing the distribution model of loading parameters on the system reliability of the portal frame. Unlike for the distribution models of the material properties, change in distribution models of loading parameters from normal to lognormal distribution, system reliability increased slightly. For instance, at section modulus of 3300cm$^3$, system safety index incresed from 8.4 to 8.8, when the load distribution is changed from normal to lognormal. However, a negligible chenge in system safety level

were observed when the load distribution model was chenged from normal to gumbel, weibull or frechet distribution.

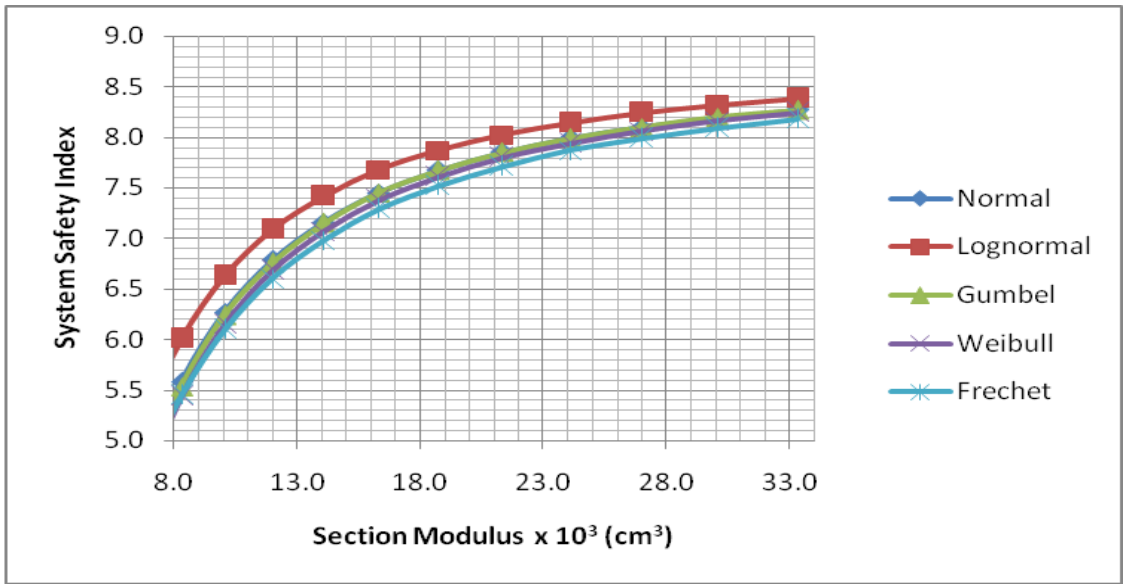



Fig. 4.51 Variation of System Safety Index with Section Modulus For the various Load Distribution Model

### 4.2.14 Effect of Change in Coefficientt of Variation of Material Property and Load

Fig. 4.52 shows the variation of modal safety indices with coefficient of variation of the material properties of the frame. For the eigth modes of failure under investion, safety index decrease with increasing coefficient of variation. The variations are similar except for the buckling mode. At 5% coefficient of variation of material properties, safety index for the frame rafter-column connection is the least with a numerical value of 3.2. In ascending order of magnitude, safety index for sway mode, frame base, bending, buckling compression and frame apex connection are 10.2, 11.2, 12.0, 14.0, 14.0 and 16.4 respectively. As he material covariance increases, the variation of safety indices are simlar order except for the buckling mode. At the smallest value of material covariance,

the safety index is 14., which is the same as that of compression mode, less than for the frame apex connection mode and higher than the other five modes of failure. However as the material covariance in icreased from 5% to 10% safety index of buckling subordinatee those of compression and frame apex coonection and became equal to the safety index for bending mode. As the material covariance decreased to 17.5%, the safety index for buckling further subordinate that of bending and became equal to that of frame base. Also, as the material covariance is futher decreased to 35%, the safety index further subordinated that of frame base and became equal to that of sway mode. However, at all values of material covariance, frame rafter-column connection remained predominant.

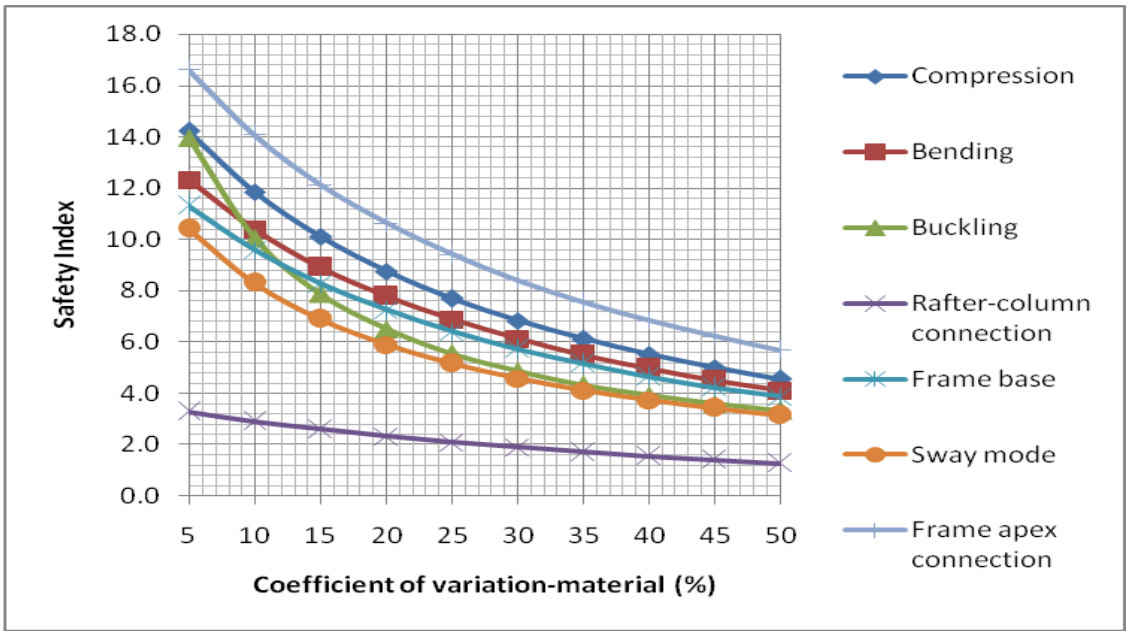



Fig. 4.52: Effect of Variation of Material Covariance on Frame Safety For the Various Modes

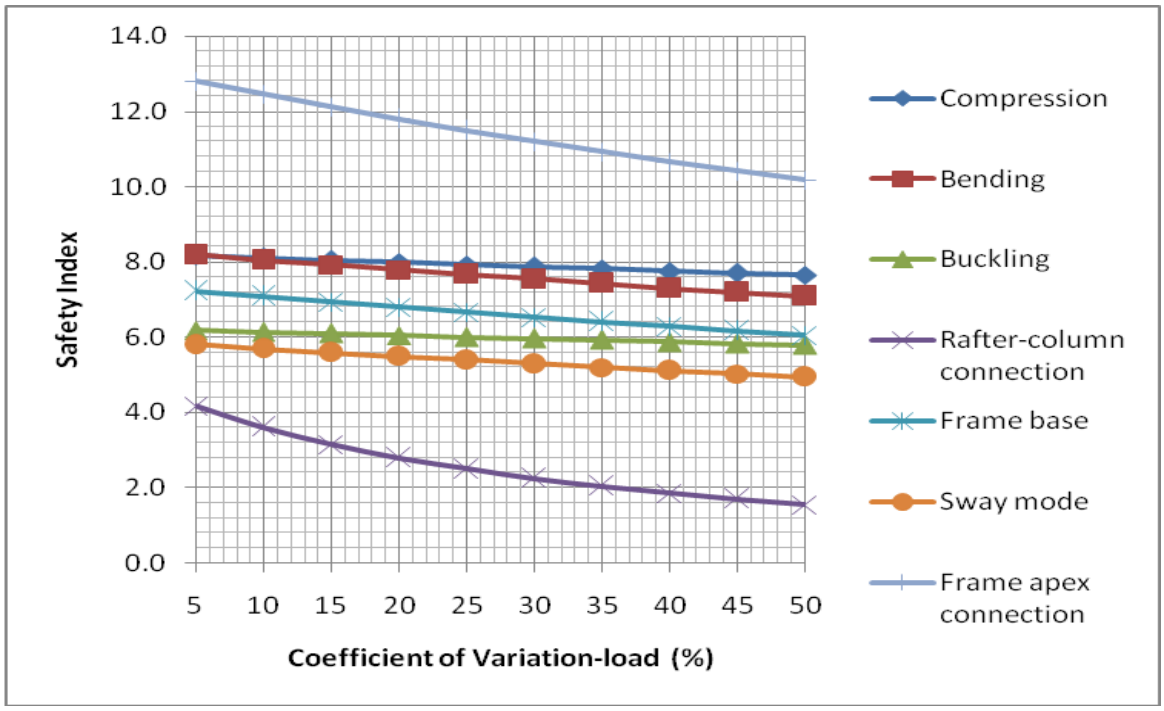



Fig. 4.53: Effect of Variation of Loading Covariance on Frame Safety For the Various Modes

Fig. 4.53 shows the variation of modal safety indices with the coefficient of variation of loading parameters. For all the failure modes under investigation, safety index decreases with increasing coefficient of variation of loading parameter. However, the decrease in safety index with incresing load covariance is more pronounned in the upper (Frme apex connection) and lower (rafter-column connection) failure modes.

### 4.3 Reliability-Based Calibration

#### 4.3.1 Calibration of Material Safety Factors

All British design codes were withdrawn in April, 2010 and engineers should now be using Eurocodes (Alasdair and Thomasons, 2011). This introduce limit state design

approach in the design of timber structures. This has great implication on the Nigerian structural Engineers. Nigeria in common with other common wealth countries depended on the British codes of parcatice. Introduction of Eurocode in Britain meant the introduction of Eurocode in Nigeria.

The Eurocode allowed member countries of the European Union and their allies adopting to Eurocode to calibrate certain parameter to suit them. These include partial safety factors. This is especially the partial safety factors of material properties of timber. This is particular due to certain peculiarities associated to the timber species, the environment and design practice.

The limit state concept has been discussed in detail elsewhere (Beal, 2001; Beal, 2010). In addition, Eurocode 0 Annexes B and C give guidance on design methods based on reliability analysis. However Eurocode 0 section C4 notes that 'Full probabilistic methods (Level III) are seldom used in the calibration of design codes because of the frequent lack of statistical data'. It states that the general basis of partial safety factors in current Eurocodes is method (a): 'calibration to a long experience of building tradition' (Eurocode 0, 2004). In other words, despite frequent references to probabilistic methods, current Eurocodes do not have partial safety factors derived from reliability analysis; they are based on experience and previous codes (Alasdair and Thomasons, 2011).

To take advantage of the reliability-based design approach, to achieve a greater consistency in the safety level for design of engineering structures, and cope with the

degree of uncertainties in material properties, one could calibrate a set of (statistical) information-sensitive partial safety factors for specified target reliability (Gwanga and Hong, 2009). With this in mind, the present study calibrated material safety factors for the Eurocode 5 resistance model, considering a three-hinge timber portal frame for material properties, using data on material properties partly obtained experimentally and partly from the most recent studies such as the JCSS (2006). The calibration was based on the reliability of the predominant mode of failure of the portal frame using the method of standardized form coefficients, implemented through the developed MATLAB program 'genehunter'.

Table 4.44 shows the calibrated material safety factors for various target system safety indices. The first step in the calibration process was the selection of the calibration scope. The calibration scope in this study was limited to a three-hinged timber portal frame. The second step of the calibration process was the selection of the calibration point. This is the design space, consisting of all basic variables. In this work, the calibration was made at system rather than component level.

Thirty two member failure modes were identified and grouped into eight phenomenal failure modes (compression, bending, buckling, rafter-column connection, frame base, member deflection, sway mode and frame apex connection) appropriated for the Eurocode 5 design criteria. The safety index for each of the eight phenomenal modes were generated from the thirty two member mode using genetic algorithms strategies

included in the program ‘genehunter’. The sytesm reliability analysis was conducted, and the predominant failure mode was idenfified.

Table 4.44 Calibration of Material Safety Factor Based on Specified Target System

| Target System Safety Index $\beta^s$ | Design Critical Section Depth (mm) | Calibrated Safety Index $\gamma_m$ |
|---|---|---|
| 2.0 | 442 | 1.26 |
| 2.5 | 462 | 1.28 |
| 3.0 | 482 | 1.36 |
| 3.5 | 505 | 1.51 |
| 4.0 | 530 | 1.60 |
| 4.5 | 558 | 1.68 |
| 5.0 | 590 | 1.86 |

The third step of the calibration process, was the deterministic design of the structure for preliminary sizing. This step was automatically implemented by the program ‘genehunter’. The fourth step of the calibration, was the development of limit state functions for each of the thirty two modes of failure. The fifth step was the determination of statistical models of material properties of the material properties, geometrical properties and loadings. The statistical models of the material properties were generated from experimental data on five selected low to high density timber specie of Nigerian origin. The sixth step was the selection of target sytem reliability. Using the established

limit state functions system reliability analysis was undertaken using genetic algorithms based on FORM. FORM sensitivity coefficients were also established. Material safety factors were then established based on the selected target system reliability and the relevant design situation.

From the reliability analysis, rafter-column connection failure mode was found to be critical and predominant. This also corresponded with the location of the maximum bending moment. The frame member section is therefore the most critical section. Being a three-hinged portal frame a series system, rafter-column connection constituted the weakest link.

The safety indices for the rafter-column connection (predominant mode) were found to be almost equivalent to the values obtained from the system reliability analysis. In a series system, occurrence of only one failure mode will always lead to system failure. Rafter-column connection failure therefore control the failure of the portal frame. The reliability based analysis and design resulted in the critical frame section at eave that satisfies the selected target reliability. In table 4.44, column 1 contains the target system reliability.

The material safety factors were calibrated based on seven target safety indices of 2,.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5.0. Column 2 contains the critical depth at eave that meet the target system safety index. The calibrated material safety factor are in column 3. From the result, it is clear that, as the target safety index increase, the material safety factor also increased.

Fig. 4.54 shows the variation of the calibrated material safety factors with the system safety index. In the Eurocode 5 (2004), material safety factor of 1.3 was recommended for solid timber. As observed in Fig. 4.54, the Eurocode 5 recommended value correspond to  to system safety index of 2.6.

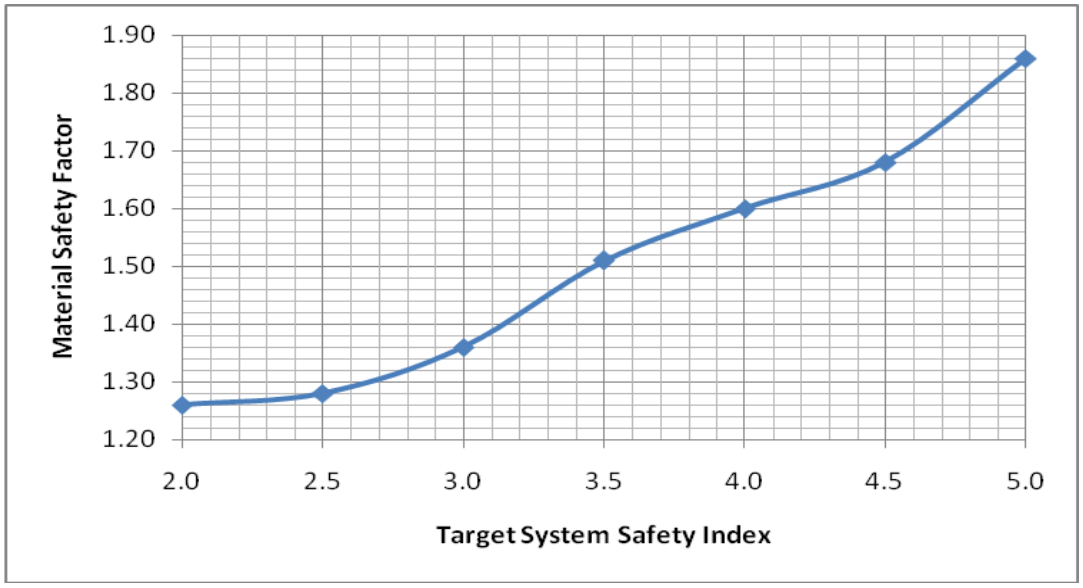



Fig. 4.54 Effect of Target System Safety Index on Material Safety Factors

### 4.3.2 Effect Load Ratio on the Computed Material Safety Factor

Table 4.45 presents the calibrated material safety factors as functions of variable to total load ratio. For timber structures, the variable to total load ratio is in the range of 0.2 (heavy timber structures) to 0.8 (light timber structures) (Ranta-maunus, 2002).  From the results presented, material safety factors decrease with inceasing load ratio. The variation is not highly pnoounced and the variation can be considered negligible.

Table 4.45 Calibration of Material Safety Factor for Different Values of Variable to Total Load Action (Target System Safety Index = 3.8)

| Variable to total Load Ratio | Design Critical Section Depth (mm) | Calibrated Safety Index $\gamma_m$ |
|---|---|---|
| 0.2 | 617 | 1.64 |
| 0.5 | 535 | 1.56 |
| 0.8 | 519 | 1.55 |

### 4.3.3 Effect of Material Coefficient of Variation on the Computed Material Safety Factor

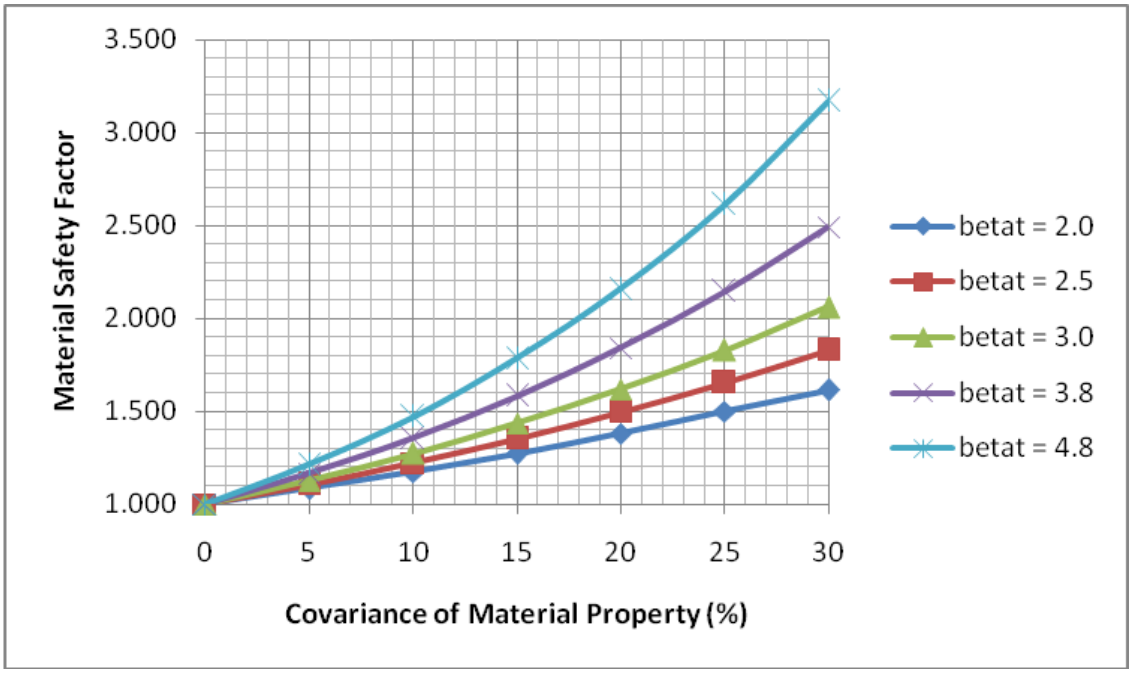



Fig. 4.55 Material Safety Index Against Covaliance of Material Property at Various Target System Safety Index

The impact of the uncertainty in material properties on the estimated safety level must be assessed for calibrating the material safety factors (Gwangha and Hong, 2009). Timber is a material with high variability in material properties, as evident from the statistical analysis conducted in this study on the data on the material properties of five selected timber species. Fig. 4.55, displayed the relatiobnship between calibrated material safety index and coefficient of variation of material properties. It is clear from the plot, that at zero coefficient of variation the safety index at all the cinsidered target system safety index are the same; equal to unity, signifying no uncertainty is associated with the design. As the cioefficient of variation increases, the material safety indiced rises above unitity and increases continously. It is also cleay that the effect of changing coefficient of variation of material propertiy is more pronounced as the targetr safety index increases. There is therefore the need for a trade up between coefficient of variation of material propertiy, and target safety index by reducing the letter, in order to minimise cost.

Regression analysis were performed with the view to develope mathematical prediction model for material safety factors as functions of material covariance and target safety index. The regression lines were presented in Figs 4.56 to 4.59. The result is a clear testimony that the Eurocode 5 recommended material safety factor of 1.3 for solid timber is conservative. Material safety factor is not unique, but uncertainty sensitive. The developed mathematical prediction nmodels are given by equations 5.2, 5.3, 5.4 and 5.5, for target safety index of 2.0, 2.5, 3.0 and 3.8 respectively. These models can be used in the Eurocode 5 resistance model, in order to adequately address the issue of uncertainty in material properties of structural timber. The coefficients of association for each model

is very high, indication of high correlation between the dependent (material safety factor) and the independent parameters (Coefficient of variation).

$$\gamma_m = 0.020\ COV_{material} + 0.981 \qquad (R^2 = 0.995) \qquad (4.2)$$

$$\gamma_m = 0.027\ COV_{material} + 0.967 \qquad (R^2 = 0.993) \qquad (4.3)$$

$$\gamma_m = 0.035\ COV_{material} + 0.949 \qquad (R^2 = 0.980) \qquad (4.4)$$

$$\gamma_m = 0.049\ COV_{material} + 0.913 \qquad (R^2 = 0.983) \qquad (4.5)$$

where $\gamma_m$ is the material safety factor, $COV_{material}$ is the coefficient of variation of material properties (bending strength, modulus of elasticity, density etc).

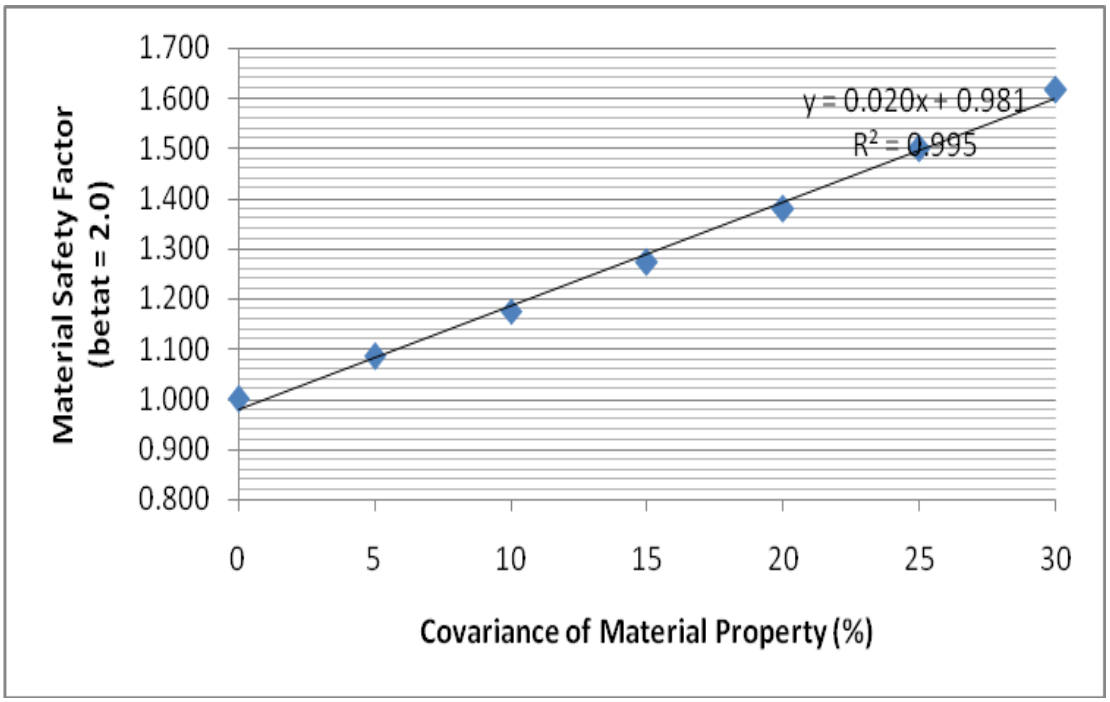



Fig. 4.56 Prediction Model of Material Safety Factors for Solid Timber

(Target Safety Index = 2.0)

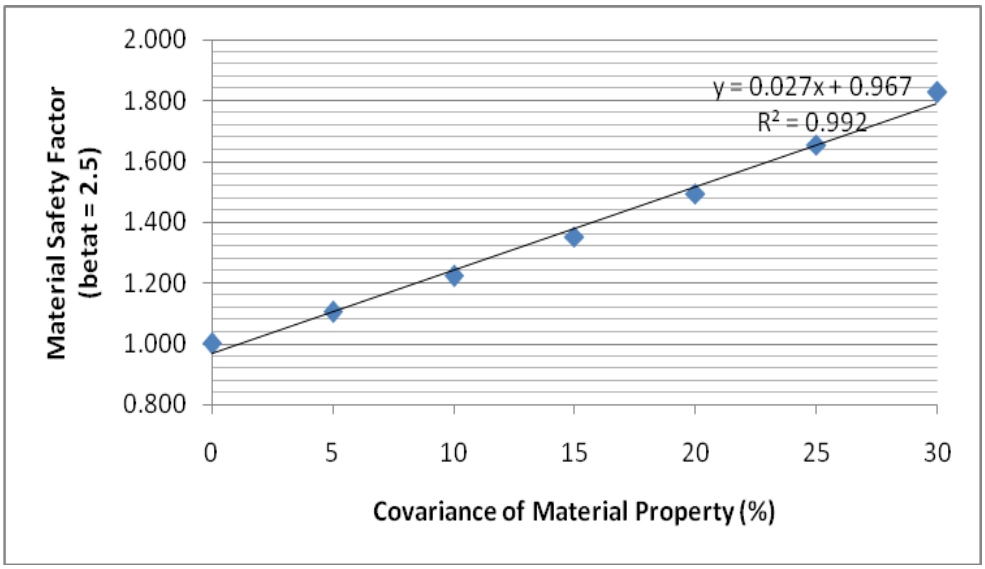



Fig. 4.57 Prediction Model of Material Safety Factors for Solid Timber

(Target Safety Index = 2.5)

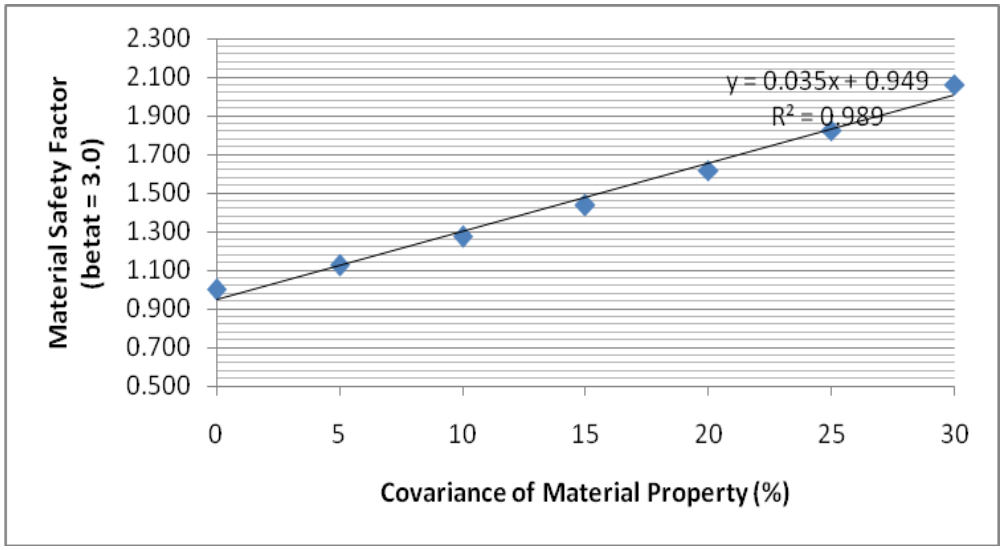



Fig. 4.58 Prediction Model of Material Safety Factors for Solid Timber

(Target Safety Index = 3.0)

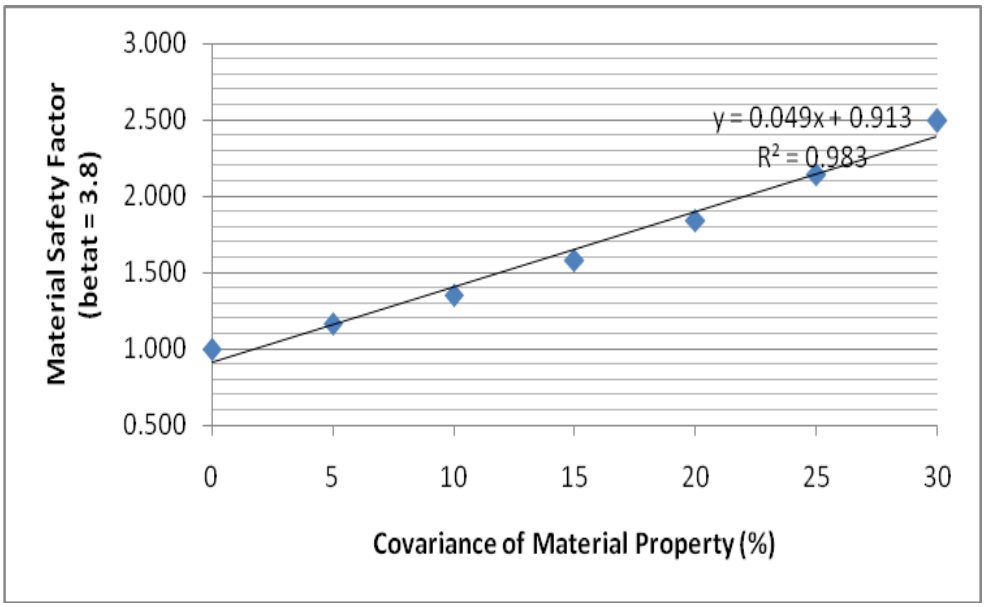



Fig. 4.59 Prediction Model of Material Safety Factors for Solid Timber

(Target Safety Index = 3.8)

Fig. 4.60, show the implied safety indices acheved with the Eurocode 5, recommended material safety factors and the material safety factors generated from the calibrated mathematical prediction models. As observed on the plot, the values of the implied safety indices were higher for the calibrated material safety factors than the Eurocode 5 based.

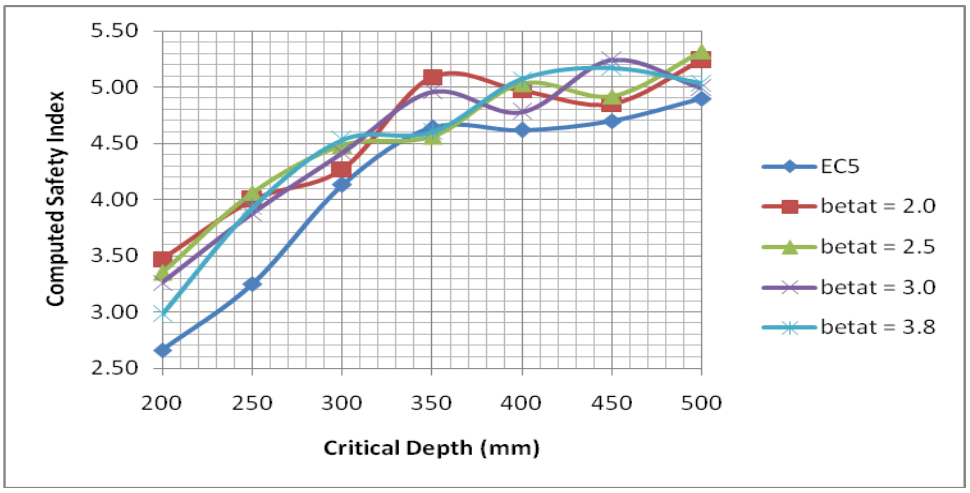



Fig. 4.60 Comparison between the Eurocode 5 and Calibrated material factor.

### 4.3.3 Summary

The stochastic models of modulus of elasticity and bending strength of five selected timber species; *Alstonia boonei*, *Triplochiton scleroxylon*, *Terminalia ivorensis)*, *Terminalia superba and lophira Alata*, were generated from four point bending test. Also, the density and moisture content were established according to the recommendations of EN 384. The generated stochastic models include the mean values, standard deviations, coefficients of variation and theoretical distribution models. These are the necessary inputs for the reliability analysis and design of engineered structures. The result are to be used in the reliability-based calibration of the Eurocode 5 design criteria of solid timber portal frame using constrained optimization technique through GA-based First Order Second Moment Method. The *Alstonia boonei*, *Triplochiton Scleroxylon* and *Terminalia Ivorensis* were allocated to strength class D18, *Terminalia Superba* to strength class D24 and *Lophira Alata* was assigned to strength class D60.

Reliability-based calibration of the Eurocode 5 design criteria of timber structures was also implemented, considering a three-hinge solid timber pitched portal frame. Extensive laboratory experiments were conducted on some selected timber species of Nigerian origin in accordance with established procedured in EN standards (EN 408 2003; EN 384, 2003). The data were analysed in accordance with the specifications of EN 384, and appropriate strength classes were assigned to each specie in accordance with EN 338 (2008) specifications. Uncertainties in the reference material properties (density, bemding strength and modulus of elasticity) were established using easyfit statistical package. The derived material properties (tension ans compression strengths parallel and perpendicular to grain, shear strength, and shear modulus) were obtained from empirical models given in EN 384 (2003) and the JCSS (2006). The data on the material properties were fitted to the most commonly used theoretical distribution models (normal, lognormal and gumbel distribution) using kolmogrov smirnov test, facility in the easyfit software. Thirty two failure modes were identified for the three-hinge timber portal frame. Limit state function was developed for each failure mode. Component and system reliability analysis were implemented using genetic algorithms based first order reliability method. Thev rafter-column joint failure mode mode was found to be predominant. The implied safety levels for the predominant mode was found to correspond well with the frame implied system reliability. Material safety factors for used in the Eurocode 5 design criteria of solid timber portal frame was conducted. Uncertainty sensitive mathematical models for the prediction of material safety factors were developed.

# CHAPTER FIVE

# CONCLUSION AND RECOMMENDATION

## 5.1 Conclusion

In the present research work, material properties of five timber species of Nigeria, origin were generated. Relability-based calibration of the Eurocode 5 design criteria of a solid timber portal frame was implenented using Genetic algorithms-based first order reliability method (FORM). The following conclusion are made:

1. The equilibrium moisture contents of the timber species tested in this studey were in the range of 15 % to 21%. *Terminalia superba* has a mean equilibrium moisture content of 20.96%. This is followed by *Alstonia boonei* with mean equilibrim moisture content of 20.59%. *Terminalia ivorenis* has mean equilibrium moisture contents of 19.26%. The mean equilibrium moisture contents for *Triplochiton scleroxylon* and *Lophira alata* were 16.83% and 15.30%, respectively. The 18% standard moisture for Nigeria, has fall within the range of 15% to 21% obtained fron the laboratoty experiments.
2. Based on the Kolmogorov Smirnov test performed in the study, it was found that, density of solid timber can best be modelled with normal distribution. The distribution models of best-fit to bending strength and modulus of elasticity of solid structural timber were found to be lognormal distribution.
3. Each of the tested timber specie was assigned to appropriate strength classes based on the European solid timber strength classification systems (EN 338, 2009). *Alstonia boonei, Triplochiton scleroxylon* and *Terminalia ivorensis were* assigned to solid timber strength class D18. Terminalia superba was assigned to

solid timber strength class D24. Finally, Lophira alata was assigned to solid timber strength class D60. The assignment were based on the reference material properties (density, modulus of elasticity and bending strength) of the species.

4. Failure of column-rafter connection was found to be predominant for timber portal frame. It is therefore the weakest list requiring the greatest attention during deign. Also, the safety indices for the premominant mode of failure was found to be almost equivalent to the system safety index. Example, for a particulay portal frame made with Lophira alata timber specie, designed to meet a predefined target safety index (Tables 4.41 to 4.43 ). The safety index of the predominant mode of failure was 3.86, and the system safety index was found to be 3.86. In conclusion therefore, the system reliability of a three-hinged solid timber portal frame is the reliabilty of its predominant mode of failure.
5. Based on the reliability-based calibration results obtained in this study, it was found that, material safety factor of solid timber is not unique as considered in the deterministic design code. The material safety factor if affected by the required safety levels. For example in Table 4.44, the calibrated material safety factor was 1.26 at target safety index of 2.0. This safety factor changed to 1.6, when the taeget safety index was increased to 4.0.
6. The calibrated material safety factor was also found to be affected by the ratio of variable to total load, as observed in Table 4.45. At load ratio of 0.2 (Heavy frame) the calibrated material safety factor was 1.64. At load ratio of 0.8 (light frame), the calibrated safety factor was found to be 1.55.

7. Coefficient of variation of material properties was found to have the most pronounced effect on material safety factor. As observed in Fig. 4.55, for a frame designed to meet a target safety index of 3.8, the required material safety factor is zero when the material coefficient of variation is zero. As the coefficient of variation of material properties was increased to 30%, required material safety factor changed to 2.5. Regression models were developed to take into account of the effects of changes in the required target reliabity and the coefficient of variation of material properties.
8. The developed uncertainty sensitive material safety factors models can be used for the design of timber structures, but could be more appropriate for the frame designed with any of the five timber species. This is because the research work was based on the material properties of the five timber species.

## 5.2 Recommendations

Based on the results obtained in this study, the following recommendations are made:

1. The strength classification of timber presented in this study was limited to only five timber species. There is the need to consider the classification of other timber species. This is pre-requisite to their use for design of timber structures using the Eurocode 5.
2. Since the material properties of timber are highly uncertain, as attested by the laboratory experimebnts presented in this study, there is the need to integrate reliability-based designed criteria in the design of timber structures in order to fully accommodate the uncertainties.

3. It was fully established in this study that system reliability of the timber portal frame agreed with the reliability of the critical mode of failure of the frame. It is therefore recommended that the system reliability of timber portal frame be taken as the reliability of the frame critical mode of failure.
4. The material properties generated in this study were adjusted to 18% moisture content, to agree with the NCP 2 recommendations. It is recommended that a separate code equivalent to EN 338 be developed for Nigeria, based on 18% moisture content, as against the 12% moisture cotent baseline used in the current EN 338.
5. The timber species tested in this study were first adjusted to 18% before being classified in accordance with EN 338. The strength classes generated could be used for the design of timber structures with Eurocode 5.
6. The material properties for loading used in this study, were obtained from data reported in the international literature. There is the need for a field study to come up with data on the statistics of loading that are peculiar to Nigeria.
7. The partial safety factors recommend for timber structures in the current Eurocode 5 is determistic and conservative. It is recommended that, the mathetical models of material safety factors developed in this study be used when designing timber structures in Nigeria. The partial safety factors were derived from reliability method and are uncertainty sensitive.

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## APPENDICES

## APPENDIX A Moisture Content and Density Test Data

### Table A1: DENSITY AND MOISTURE CONTENT (*Alstonia Boonei*)

| SERIAL NUMBER | BULK WEIGHT (g) | OVEN DRY WEIGHT (g) | MOISTURE CONTENT (%) | DENSITY ($kg/m^3$) |
|---|---|---|---|---|
| A1-1 | 68.6 | 51.9 | 32.0 | 365.7 |
| A1-2 | 71.0 | 60.0 | 18.4 | 378.9 |
| A1-3 | 68.1 | 55.2 | 23.4 | 363.1 |
| A1-4 | 69.6 | 54.7 | 27.3 | 371.1 |
| A1-5 | 69.4 | 60.1 | 15.5 | 370.1 |
| A1-6 | 67.2 | 50.7 | 32.6 | 358.7 |
| A1-7 | 65.8 | 52.8 | 24.7 | 351.1 |
| A1-8 | 65.3 | 60.2 | 8.4 | 348.3 |
| A1-9 | 66.8 | 59.2 | 12.7 | 356.0 |
| A1-10 | 64.5 | 54.9 | 17.6 | 344.2 |
| A1-11 | 69.3 | 55.3 | 25.2 | 369.4 |
| A1-12 | 68.2 | 66.7 | 2.3 | 363.9 |
| A1-13 | 67.8 | 51.4 | 31.9 | 361.7 |
| A1-14 | 67.2 | 59.1 | 13.7 | 358.2 |
| A1-15 | 66.4 | 51.8 | 28.1 | 354.1 |
| A1-16 | 69.8 | 57.4 | 21.6 | 372.3 |
| A1-17 | 68.9 | 53.7 | 28.4 | 367.5 |
| A1-18 | 67.8 | 62.2 | 8.9 | 361.4 |
| A1-19 | 70.3 | 60.0 | 17.1 | 375.0 |
| A1-20 | 67.4 | 53.0 | 27.3 | 359.5 |
| A2-1 | 70.6 | 55.5 | 27.1 | 376.3 |
| A2-2 | 67.4 | 57.5 | 17.2 | 359.6 |
| A2-3 | 68.9 | 51.7 | 33.2 | 367.5 |
| A2-4 | 67.6 | 51.6 | 31.1 | 360.5 |
| A2-5 | 65.1 | 52.6 | 23.7 | 347.2 |
| A2-6 | 67.5 | 55.3 | 22.1 | 360.1 |
| A2-7 | 64.9 | 53.6 | 21.2 | 346.3 |
| A2-8 | 69.9 | 60.6 | 15.3 | 373.1 |
| A2-9 | 68.3 | 54.1 | 26.1 | 364.2 |
| A2-10 | 71.5 | 61.7 | 15.9 | 381.1 |
| A2-11 | 64.9 | 58.2 | 11.6 | 346.3 |

| | | | | |
|---|---|---|---|---|
| A2-12 | 71.1 | 69.7 | 2.0 | 379.0 |
| A2-13 | 65.7 | 49.5 | 32.8 | 350.6 |
| A2-14 | 65.3 | 56.2 | 16.2 | 348.2 |
| A2-15 | 72.3 | 64.9 | 11.4 | 385.5 |
| A2-16 | 63.7 | 51.7 | 23.2 | 339.8 |
| A2-17 | 63.8 | 54.3 | 17.3 | 340.0 |
| A2-18 | 66.0 | 58.8 | 12.2 | 351.8 |
| A2-19 | 66.1 | 56.0 | 18.0 | 352.8 |
| A2-20 | 67.2 | 51.9 | 29.5 | 358.3 |

**Table A2: DENSITY AND MOISTURE CONTENT (*Triplochiton Scleroxylon*)**

| SERIAL NUMBER | BULK WEIGHT (g) | OVEN DRY WEIGHT (g) | MOISTURE CONTENT (%) | DENSITY ($kg/m^3$) |
|---|---|---|---|---|
| B1-1 | 81.4 | 72.1 | 12.8 | 434.0 |
| B1-2 | 65.9 | 59.5 | 10.8 | 351.4 |
| B1-3 | 76.0 | 67.0 | 13.3 | 405.1 |
| B1-4 | 66.9 | 59.2 | 13.1 | 356.9 |
| B1-5 | 68.8 | 53.6 | 28.5 | 367.1 |
| B1-6 | 56.8 | 47.3 | 20.1 | 303.1 |
| B1-7 | 89.2 | 82.8 | 7.7 | 475.7 |
| B1-8 | 63.9 | 55.7 | 14.7 | 340.8 |
| B1-9 | 108.4 | 94.0 | 15.3 | 578.1 |
| B1-10 | 83.2 | 71.4 | 16.5 | 443.8 |
| B1-11 | 60.6 | 52.5 | 15.3 | 323.1 |
| B1-12 | 72.9 | 63.9 | 14.1 | 389.0 |
| B1-13 | 78.5 | 64.3 | 22.0 | 418.5 |
| B1-14 | 84.5 | 72.4 | 16.7 | 450.5 |
| B1-15 | 47.6 | 40.0 | 19.1 | 253.8 |
| B1-16 | 63.7 | 54.3 | 17.4 | 339.9 |
| B1-17 | 80.4 | 71.1 | 13.0 | 428.6 |
| B1-18 | 68.2 | 58.5 | 16.6 | 363.7 |
| B1-19 | 90.5 | 72.4 | 25.0 | 482.6 |
| B1-20 | 50.3 | 43.8 | 14.8 | 268.3 |
| B2-1 | 59.8 | 49.9 | 19.9 | 319.0 |
| B2-2 | 69.5 | 60.0 | 15.8 | 370.6 |
| B2-3 | 73.4 | 61.1 | 20.2 | 391.7 |
| B2-4 | 73.4 | 63.5 | 15.7 | 391.7 |
| B2-5 | 79.6 | 66.5 | 19.7 | 424.4 |
| B2-6 | 80.0 | 63.3 | 26.4 | 426.6 |
| B2-7 | 48.0 | 41.2 | 16.4 | 255.8 |
| B2-8 | 84.1 | 73.6 | 14.3 | 448.6 |
| B2-9 | 78.5 | 64.5 | 21.8 | 418.9 |
| B2-10 | 77.8 | 67.5 | 15.2 | 414.7 |
| B2-11 | 72.5 | 60.2 | 20.5 | 386.8 |
| B2-12 | 86.0 | 68.0 | 26.4 | 458.7 |
| B2-13 | 67.6 | 57.5 | 17.6 | 360.6 |
| B2-14 | 80.4 | 67.6 | 19.0 | 428.8 |

| | | | | |
|---|---|---|---|---|
| B2-15 | 64.9 | 56.4 | 15.0 | 346.0 |
| B2-16 | 66.9 | 55.9 | 19.6 | 356.7 |
| B2-17 | 74.9 | 59.3 | 26.4 | 399.5 |
| B2-18 | 74.0 | 68.1 | 8.6 | 394.6 |
| B2-19 | 76.5 | 64.9 | 17.9 | 408.0 |
| B2-20 | 76.1 | 66.3 | 14.7 | 405.7 |

**Table A3: DENSITY AND MOISTURE CONTENT (*Terminalia Ivorensis*)**

| TEST PIECE MARK | BULK WEIGHT (g) | OVEN DRY WEIGHT (g) | MOISTURE CONTENT (%) | DENSITY ($kg/m^3$) |
|---|---|---|---|---|
| C1-1 | 78.4 | 65.4 | 19.9 | 418.1 |
| C1-2 | 72.0 | 60.3 | 19.5 | 384.1 |
| C1-3 | 76.0 | 66.6 | 14.1 | 405.3 |
| C1-4 | 130.5 | 110.8 | 17.8 | 696.0 |
| C1-5 | 69.1 | 58.8 | 17.5 | 368.3 |
| C1-6 | 81.5 | 72.7 | 12.1 | 434.5 |
| C1-7 | 90.2 | 74.5 | 21.1 | 481.1 |
| C1-8 | 101.2 | 86.1 | 17.5 | 539.9 |
| C1-9 | 94.4 | 76.9 | 22.8 | 503.4 |
| C1-10 | 97.6 | 77.7 | 25.7 | 520.7 |
| C1-11 | 87.2 | 74.2 | 17.5 | 465.1 |
| C1-12 | 91.9 | 76.4 | 20.3 | 490.2 |
| C1-13 | 74.8 | 63.0 | 18.7 | 398.8 |
| C1-14 | 66.3 | 53.9 | 23.0 | 353.4 |
| C1-15 | 88.0 | 78.4 | 12.3 | 469.6 |
| C1-16 | 100.7 | 83.8 | 20.2 | 537.1 |
| C1-17 | 95.4 | 80.2 | 18.9 | 508.8 |
| C1-18 | 70.7 | 58.1 | 21.8 | 377.3 |
| C1-19 | 115.3 | 93.9 | 22.8 | 614.7 |
| C1-20 | 111.9 | 96.0 | 16.5 | 596.8 |
| C2-1 | 81.1 | 69.5 | 16.7 | 432.8 |
| C2-2 | 103.1 | 88.5 | 16.5 | 549.8 |
| C2-3 | 100.3 | 79.9 | 25.5 | 535.1 |
| C2-4 | 75.2 | 64.6 | 16.4 | 400.9 |
| C2-5 | 80.0 | 70.1 | 14.1 | 426.7 |
| C2-6 | 91.0 | 72.9 | 24.7 | 485.1 |
| C2-7 | 102.3 | 89.8 | 13.9 | 545.4 |
| C2-8 | 66.0 | 52.3 | 26.1 | 352.0 |
| C2-9 | 86.4 | 69.9 | 23.7 | 460.8 |
| B2-10 | 72.4 | 62.3 | 16.3 | 386.2 |
| B2-11 | 95.9 | 81.4 | 17.8 | 511.5 |
| B2-12 | 68.5 | 61.9 | 10.7 | 365.3 |
| B2-13 | 53.7 | 47.6 | 12.8 | 286.2 |
| B2-14 | 68.9 | 61.6 | 11.7 | 367.4 |

| | | | | |
|---|---|---|---|---|
| B2-15 | 63.5 | 54.5 | 16.4 | 338.4 |
| B2-16 | 90.4 | 78.9 | 14.6 | 481.9 |
| B2-17 | 87.1 | 75.5 | 15.4 | 464.3 |
| B2-18 | 93.4 | 77.6 | 20.3 | 498.0 |
| B2-19 | 76.0 | 63.3 | 20.1 | 405.5 |
| B2-20 | 88.6 | 71.9 | 23.3 | 472.4 |

**Table A4: DENSITY AND MOISTURE CONTENT (*Terminalia Superba*)**

| TEST PIECE MARK | BULK WEIGHT (g) | OVEN DRY WEIGHT (g) | MOISTURE CONTENT (%) | DENSITY ($kg/m^3$) |
|---|---|---|---|---|
| D1-1 | 105.4 | 84.1 | 25.4 | 562.4 |
| D1-2 | 85.5 | 71.8 | 19.1 | 456.0 |
| D1-3 | 115.8 | 90.1 | 28.5 | 617.6 |
| D1-4 | 69.4 | 55.6 | 24.8 | 369.9 |
| D1-5 | 87.2 | 74.7 | 16.8 | 465.0 |
| D1-6 | 108.8 | 86.3 | 26.1 | 580.1 |
| D1-7 | 56.1 | 48.1 | 16.7 | 299.4 |
| D1-8 | 95.6 | 78.3 | 22.1 | 509.7 |
| D1-9 | 104.0 | 88.2 | 17.9 | 554.6 |
| D1-10 | 104.7 | 85.8 | 22.0 | 558.5 |
| D1-11 | 89.1 | 76.8 | 16.1 | 475.1 |
| D1-12 | 98.6 | 81.1 | 21.6 | 525.8 |
| D1-13 | 121.0 | 99.5 | 21.7 | 645.5 |
| D1-14 | 70.3 | 59.0 | 19.2 | 375.0 |
| D1-15 | 91.1 | 77.3 | 17.8 | 485.9 |
| D1-16 | 111.2 | 90.1 | 23.4 | 593.2 |
| D1-17 | 117.1 | 98.2 | 19.3 | 624.7 |
| D1-18 | 113.4 | 90.7 | 25.1 | 604.9 |
| D1-19 | 114.8 | 95.3 | 20.4 | 612.2 |
| D1-20 | 96.6 | 75.8 | 27.5 | 515.4 |
| D2-1 | 109.3 | 89.8 | 21.7 | 583.0 |
| D2-2 | 93.7 | 76.4 | 22.6 | 499.7 |
| D2-3 | 105.1 | 87.6 | 20.0 | 560.3 |
| D2-4 | 95.0 | 79.3 | 19.8 | 506.7 |
| D2-5 | 88.1 | 74.7 | 17.9 | 469.9 |
| D2-6 | 128.4 | 96.9 | 32.5 | 684.7 |
| D2-7 | 125.0 | 104.2 | 20.0 | 666.5 |
| D2-8 | 89.4 | 75.0 | 19.1 | 476.6 |
| D2-9 | 96.5 | 77.3 | 24.8 | 514.6 |
| D2-10 | 102.7 | 84.4 | 21.7 | 547.8 |
| D2-11 | 84.8 | 75.9 | 11.7 | 452.2 |
| D2-12 | 96.6 | 77.2 | 25.2 | 515.4 |
| D2-13 | 92.8 | 74.4 | 24.8 | 495.1 |
| D2-14 | 75.1 | 61.6 | 21.8 | 400.3 |

| | | | | |
|---|---|---|---|---|
| D2-15 | 91.0 | 73.7 | 23.6 | 485.4 |
| D2-16 | 81.3 | 70.2 | 15.8 | 433.4 |
| D2-17 | 113.7 | 92.2 | 23.3 | 606.3 |
| D2-18 | 99.2 | 86.4 | 14.8 | 529.1 |
| D2-19 | 98.2 | 79.9 | 22.9 | 523.7 |
| D2-20 | 75.1 | 66.4 | 13.0 | 400.4 |

**Table A5: DENSITY AND MOISTURE CONTENT (*Lophira Alata*)**

| TEST PIECE MARK | BULK WEIGHT (g) | OVEN DRY WEIGHT (g) | MOISTURE CONTENT (%) | DENSITY ($kg/m^3$) |
|---|---|---|---|---|
| E1-1 | 173.1 | 154.7 | 11.9 | 923.0 |
| E1-2 | 174.3 | 153.5 | 13.5 | 929.3 |
| E1-3 | 186.5 | 163.0 | 14.4 | 994.9 |
| E1-4 | 172.0 | 147.0 | 16.9 | 917.1 |
| E1-5 | 186.1 | 162.3 | 14.7 | 992.4 |
| E1-6 | 181.4 | 159.9 | 13.5 | 967.7 |
| E1-7 | 173.5 | 149.9 | 15.7 | 925.1 |
| E1-8 | 179.6 | 156.1 | 15.1 | 958.0 |
| E1-9 | 182.2 | 155.6 | 17.1 | 971.7 |
| E1-10 | 189.5 | 164.3 | 15.3 | 1010.6 |
| E1-11 | 192.4 | 171.5 | 12.2 | 1026.4 |
| E1-12 | 170.3 | 145.6 | 17.0 | 908.2 |
| E1-13 | 170.0 | 146.3 | 16.2 | 906.6 |
| E1-14 | 166.9 | 144.5 | 15.5 | 890.3 |
| E1-15 | 182.0 | 161.7 | 12.5 | 970.7 |
| E1-16 | 163.8 | 141.5 | 15.7 | 873.5 |
| E1-17 | 173.5 | 151.6 | 14.5 | 925.3 |
| E1-18 | 185.7 | 161.5 | 14.9 | 990.3 |
| E1-19 | 198.0 | 170.9 | 15.9 | 1056.2 |
| E1-20 | 157.5 | 135.5 | 16.2 | 839.8 |
| E2-1 | 191.4 | 165.9 | 15.3 | 1020.6 |
| E2-2 | 171.6 | 146.3 | 17.3 | 915.5 |
| E2-3 | 188.0 | 162.6 | 15.6 | 1002.7 |
| E2-4 | 201.8 | 173.5 | 16.3 | 1076.4 |
| E2-5 | 179.6 | 154.7 | 16.1 | 958.0 |
| E2-6 | 193.2 | 166.3 | 16.2 | 1030.6 |
| E2-7 | 180.7 | 152.0 | 18.8 | 963.6 |
| E2-8 | 189.6 | 162.1 | 17.0 | 1011.3 |
| E2-9 | 182.5 | 156.7 | 16.5 | 973.5 |
| E2-10 | 177.5 | 154.6 | 14.9 | 946.8 |
| E2-11 | 186.3 | 162.6 | 14.5 | 993.3 |
| E2-12 | 178.1 | 156.8 | 13.6 | 949.8 |
| E2-13 | 175.0 | 154.4 | 13.3 | 933.1 |
| E2-14 | 188.9 | 165.4 | 14.2 | 1007.5 |

| | | | | |
|---|---|---|---|---|
| E2-15 | 170.7 | 149.5 | 14.2 | 910.6 |
| E2-16 | 174.3 | 150.5 | 15.8 | 929.3 |
| E2-17 | 181.7 | 159.5 | 13.9 | 969.1 |
| E2-18 | 185.8 | 160.6 | 15.7 | 990.8 |
| E2-19 | 163.3 | 138.7 | 17.7 | 870.7 |
| E2-20 | 162.2 | 140.0 | 15.8 | 865.2 |

**APPENDIX B: Four Point Bending Test Data**

**Table B 1: 4 POINT BENDING STRENGTH (*Alstonia Boonei*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 13.9 | 66.9 | 50.7 |
| 2 | 12.3 | 59.3 | 45.0 |
| 3 | 10.3 | 49.6 | 37.7 |
| 4 | 11.3 | 54.2 | 41.1 |
| 5 | 12.3 | 59.2 | 44.9 |
| 6 | 11.9 | 57.1 | 43.3 |
| 7 | 12.5 | 59.8 | 45.4 |
| 8 | 10.0 | 48.0 | 36.4 |
| 9 | 12.6 | 60.4 | 45.8 |
| 10 | 9.7 | 46.6 | 35.4 |
| 11 | 13.1 | 63.0 | 47.8 |
| 12 | 12.2 | 58.4 | 44.3 |
| 13 | 11.2 | 53.9 | 40.9 |
| 14 | 10.0 | 48.1 | 36.5 |
| 15 | 12.0 | 57.8 | 43.8 |
| 16 | 13.7 | 65.8 | 49.9 |
| 17 | 11.5 | 55.0 | 41.7 |
| 18 | 14.2 | 68.3 | 51.8 |
| 19 | 11.3 | 54.1 | 41.0 |
| 20 | 10.7 | 51.4 | 39.0 |
| 21 | 13.0 | 62.6 | 47.5 |
| 22 | 13.2 | 63.4 | 48.1 |
| 23 | 12.1 | 58.1 | 44.1 |
| 24 | 10.9 | 52.2 | 39.6 |
| 25 | 12.0 | 57.8 | 43.8 |
| 26 | 10.6 | 51.0 | 38.7 |
| 27 | 9.2 | 44.4 | 33.7 |
| 28 | 9.6 | 46.3 | 35.1 |
| 19 | 13.7 | 65.8 | 49.9 |
| 30 | 12.7 | 61.0 | 46.3 |
| 31 | 14.1 | 67.9 | 51.5 |

| | | | |
|---|---|---|---|
| 32 | 13.6 | 65.2 | 49.5 |
| 33 | 9.8 | 47.1 | 35.7 |
| 34 | 12.3 | 59.2 | 44.9 |
| 35 | 13.1 | 63.0 | 47.8 |
| 36 | 10.8 | 51.6 | 39.2 |
| 37 | 11.0 | 52.6 | 39.9 |
| 38 | 12.2 | 58.7 | 44.5 |
| 39 | 12.3 | 58.9 | 44.7 |
| 40 | 10.0 | 48.0 | 36.4 |

**Table B2: 4 POINT BENDING STRENGTH (*Triplochiton Scleroxylon*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 13.9 | 66.7 | 50.6 |
| 2 | 10.8 | 51.9 | 39.4 |
| 3 | 15.8 | 75.7 | 57.4 |
| 4 | 11.6 | 55.7 | 42.2 |
| 5 | 15.0 | 71.9 | 54.6 |
| 6 | 15.5 | 74.6 | 56.6 |
| 7 | 16.2 | 77.7 | 58.9 |
| 8 | 12.3 | 59.2 | 44.9 |
| 9 | 14.8 | 71.0 | 53.9 |
| 10 | 15.1 | 72.6 | 55.1 |
| 11 | 13.3 | 63.6 | 48.3 |
| 12 | 17.9 | 85.7 | 65.0 |
| 13 | 11.7 | 56.0 | 42.5 |
| 14 | 11.2 | 53.6 | 40.7 |
| 15 | 14.0 | 67.0 | 50.9 |
| 16 | 14.1 | 67.6 | 51.3 |
| 17 | 16.0 | 77.0 | 58.4 |
| 18 | 13.9 | 66.5 | 50.5 |
| 19 | 12.1 | 58.3 | 44.2 |
| 20 | 13.0 | 62.5 | 47.5 |
| 21 | 15.3 | 73.4 | 55.7 |
| 22 | 12.6 | 60.6 | 46.0 |
| 23 | 15.4 | 73.8 | 56.0 |
| 24 | 16.9 | 80.9 | 61.4 |
| 25 | 11.8 | 56.7 | 43.0 |
| 26 | 16.3 | 78.4 | 59.5 |
| 27 | 14.4 | 69.2 | 52.5 |
| 28 | 18.1 | 87.0 | 66.0 |
| 19 | 13.5 | 64.9 | 49.2 |
| 30 | 15.7 | 75.4 | 57.2 |
| 31 | 13.7 | 66.0 | 50.1 |
| 32 | 17.1 | 82.1 | 62.3 |
| 33 | 17.9 | 86.1 | 65.4 |

| | | | |
|---|---|---|---|
| 34 | 15.6 | 74.8 | 56.7 |
| 35 | 18.4 | 88.2 | 66.9 |
| 36 | 13.1 | 62.9 | 47.7 |
| 37 | 21.3 | 102.1 | 77.5 |
| 38 | 19.0 | 91.3 | 69.3 |
| 39 | 12.8 | 61.4 | 46.6 |
| 40 | 16.8 | 80.4 | 61.0 |

**Table B 3: 4 POINT BENDING STRENGTH (*Terminalia Ivorensis*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 20.4 | 97.8 | 74.2 |
| 2 | 13.9 | 66.7 | 50.6 |
| 3 | 17.5 | 83.9 | 63.6 |
| 4 | 24.1 | 115.6 | 87.7 |
| 5 | 21.6 | 103.6 | 78.6 |
| 6 | 21.0 | 100.9 | 76.5 |
| 7 | 16.3 | 78.3 | 59.4 |
| 8 | 22.3 | 107.0 | 81.2 |
| 9 | 22.6 | 108.3 | 82.2 |
| 10 | 23.3 | 112.0 | 84.9 |
| 11 | 19.7 | 94.7 | 71.9 |
| 12 | 24.8 | 119.2 | 90.4 |
| 13 | 18.9 | 90.8 | 68.9 |
| 14 | 12.2 | 58.6 | 44.5 |
| 15 | 18.6 | 89.4 | 67.8 |
| 16 | 17.0 | 81.6 | 61.9 |
| 17 | 22.0 | 105.6 | 80.2 |
| 18 | 19.0 | 91.4 | 69.3 |
| 19 | 18.1 | 86.7 | 65.8 |
| 20 | 19.5 | 93.6 | 71.0 |
| 21 | 18.0 | 86.4 | 65.6 |
| 22 | 15.2 | 73.1 | 55.4 |
| 23 | 20.3 | 97.2 | 73.8 |
| 24 | 18.7 | 89.6 | 68.0 |
| 25 | 10.4 | 49.7 | 37.7 |
| 26 | 14.3 | 68.5 | 52.0 |
| 27 | 16.7 | 80.0 | 60.7 |
| 28 | 14.1 | 67.6 | 51.3 |
| 19 | 26.3 | 126.4 | 95.9 |
| 30 | 17.7 | 84.8 | 64.3 |
| 31 | 16.5 | 79.4 | 60.3 |
| 32 | 22.4 | 107.6 | 81.7 |
| 33 | 20.3 | 97.3 | 73.8 |

| | | | |
|---|---|---|---|
| 34 | 20.4 | 98.1 | 74.5 |
| 35 | 25.9 | 124.4 | 94.4 |
| 36 | 27.3 | 131.3 | 99.6 |
| 37 | 20.4 | 98.0 | 74.4 |
| 38 | 19.0 | 91.2 | 69.2 |
| 39 | 20.5 | 98.6 | 74.8 |
| 40 | 16.9 | 81.1 | 61.6 |

**Table B 4: 4 POINT BENDING STRENGTH (*Terminalia Superba*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 24.3 | 116.7 | 88.5 |
| 2 | 25.4 | 121.9 | 92.5 |
| 3 | 18.4 | 88.6 | 67.2 |
| 4 | 16.0 | 76.8 | 58.3 |
| 5 | 18.4 | 88.2 | 66.9 |
| 6 | 28.5 | 136.8 | 103.8 |
| 7 | 23.5 | 112.6 | 85.4 |
| 8 | 27.4 | 131.3 | 99.6 |
| 9 | 22.8 | 109.4 | 83.0 |
| 10 | 25.1 | 120.5 | 91.4 |
| 11 | 22.4 | 107.4 | 81.5 |
| 12 | 30.9 | 148.2 | 112.4 |
| 13 | 17.3 | 82.9 | 62.9 |
| 14 | 17.5 | 84.1 | 63.8 |
| 15 | 19.7 | 94.5 | 71.7 |
| 16 | 23.9 | 114.6 | 86.9 |
| 17 | 20.3 | 97.3 | 73.8 |
| 18 | 23.2 | 111.1 | 84.3 |
| 19 | 19.3 | 92.5 | 70.1 |
| 20 | 19.3 | 92.6 | 70.3 |
| 21 | 16.8 | 80.8 | 61.3 |
| 22 | 22.7 | 109.1 | 82.8 |
| 23 | 22.4 | 107.7 | 81.7 |
| 24 | 17.6 | 84.6 | 64.2 |
| 25 | 19.3 | 92.6 | 70.2 |
| 26 | 27.1 | 130.2 | 98.8 |
| 27 | 17.0 | 81.6 | 61.9 |
| 28 | 24.9 | 119.4 | 90.6 |
| 19 | 28.7 | 137.9 | 104.7 |
| 30 | 22.4 | 107.3 | 81.4 |
| 31 | 21.3 | 102.3 | 77.6 |
| 32 | 28.9 | 138.7 | 105.2 |
| 33 | 27.2 | 130.7 | 99.1 |

| | | | |
|---|---|---|---|
| 34 | 24.2 | 116.3 | 88.2 |
| 35 | 17.2 | 82.4 | 62.5 |
| 36 | 30.4 | 146.0 | 110.8 |
| 37 | 24.4 | 116.9 | 88.7 |
| 38 | 23.8 | 114.1 | 86.6 |
| 39 | 28.9 | 138.6 | 105.1 |
| 40 | 30.0 | 144.0 | 109.3 |

**Table B 5: 4 POINT BENDING STRENGTH (*Lophira Alata*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 24.8 | 118.9 | 90.2 |
| 2 | 31.2 | 149.8 | 113.6 |
| 3 | 26.9 | 129.2 | 98.0 |
| 4 | 25.7 | 123.1 | 93.4 |
| 5 | 27.3 | 130.9 | 99.3 |
| 6 | 20.5 | 98.6 | 74.8 |
| 7 | 25.8 | 123.8 | 93.9 |
| 8 | 28.1 | 135.1 | 102.5 |
| 9 | 29.9 | 143.5 | 108.9 |
| 10 | 25.6 | 123.0 | 93.3 |
| 11 | 23.0 | 110.6 | 83.9 |
| 12 | 27.7 | 133.0 | 100.9 |
| 13 | 21.8 | 104.8 | 79.5 |
| 14 | 25.4 | 122.0 | 92.6 |
| 15 | 25.7 | 123.2 | 93.5 |
| 16 | 30.5 | 146.5 | 111.2 |
| 17 | 34.7 | 166.6 | 126.4 |
| 18 | 23.9 | 114.8 | 87.1 |
| 19 | 32.4 | 155.7 | 118.1 |
| 20 | 25.1 | 120.7 | 91.6 |
| 21 | 35.9 | 172.3 | 130.7 |
| 22 | 24.8 | 119.2 | 90.5 |
| 23 | 26.1 | 125.5 | 95.2 |
| 24 | 25.2 | 121.2 | 92.0 |
| 25 | 32.9 | 158.0 | 119.8 |
| 26 | 28.6 | 137.1 | 104.0 |
| 27 | 24.4 | 117.3 | 89.0 |
| 28 | 28.1 | 134.7 | 102.2 |
| 19 | 25.8 | 123.8 | 93.9 |
| 30 | 22.4 | 107.5 | 81.6 |
| 31 | 34.2 | 164.2 | 124.6 |
| 32 | 20.5 | 98.3 | 74.6 |
| 33 | 24.3 | 116.8 | 88.6 |

| | | | |
|---|---|---|---|
| 34 | 36.7 | 176.0 | 133.6 |
| 35 | 25.0 | 119.9 | 90.9 |
| 36 | 17.0 | 81.8 | 62.1 |
| 37 | 24.2 | 115.9 | 88.0 |
| 38 | 27.6 | 132.4 | 100.5 |
| 39 | 26.7 | 128.0 | 97.1 |
| 40 | 27.4 | 131.8 | 100.0 |

**Table B 6: 4 POINT MODULUS OF ELASTICITY (*Alstonia Boonei*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 3.8 | 6.6 | 7.5 | 8.5 | 7986.6 |
| 2 | 4.6 | 7.4 | 8.3 | 9.2 | 8009.6 |
| 3 | 3.3 | 6.0 | 6.9 | 7.8 | 8418.6 |
| 4 | 4.2 | 7.0 | 8.0 | 8.9 | 7928.9 |
| 5 | 4.0 | 6.4 | 7.2 | 8.0 | 9202.5 |
| 6 | 3.8 | 6.5 | 7.4 | 8.3 | 8342.2 |
| 7 | 4.5 | 7.0 | 7.9 | 8.7 | 8786.8 |
| 8 | 4.3 | 7.2 | 8.1 | 9.1 | 7795.6 |
| 9 | 3.8 | 6.5 | 7.4 | 8.3 | 8267.0 |
| 10 | 4.0 | 6.6 | 7.4 | 8.3 | 8665.0 |
| 11 | 3.6 | 6.2 | 7.1 | 8.0 | 8450.2 |
| 12 | 3.4 | 5.9 | 6.8 | 7.6 | 8812.2 |
| 13 | 3.4 | 6.1 | 7.0 | 7.9 | 8387.0 |
| 14 | 3.5 | 6.1 | 7.0 | 7.9 | 8468.3 |
| 15 | 4.4 | 7.2 | 8.1 | 9.1 | 7986.2 |
| 16 | 4.1 | 6.9 | 7.9 | 8.8 | 7799.6 |
| 17 | 3.4 | 6.1 | 7.1 | 8.0 | 8046.8 |
| 18 | 4.4 | 7.3 | 8.2 | 9.2 | 7836.7 |
| 19 | 4.0 | 6.8 | 7.7 | 8.6 | 8255.8 |
| 20 | 4.3 | 6.9 | 7.7 | 8.6 | 8645.4 |
| 21 | 4.1 | 6.7 | 7.6 | 8.5 | 8399.2 |
| 22 | 3.4 | 6.1 | 7.0 | 7.9 | 8202.6 |
| 23 | 3.9 | 6.5 | 7.3 | 8.2 | 8695.3 |
| 24 | 5.0 | 7.8 | 8.7 | 9.6 | 7993.4 |
| 25 | 4.8 | 7.5 | 8.5 | 9.4 | 8069.1 |
| 26 | 3.9 | 6.7 | 7.6 | 8.5 | 8105.2 |
| 27 | 4.1 | 6.8 | 7.7 | 8.6 | 8184.9 |
| 28 | 4.5 | 7.6 | 8.6 | 9.6 | 7331.8 |
| 19 | 3.4 | 6.3 | 7.3 | 8.3 | 7546.9 |
| 30 | 3.5 | 6.2 | 7.0 | 7.9 | 8346.4 |
| 31 | 3.8 | 6.6 | 7.6 | 8.5 | 7836.6 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 3.8 | 6.5 | 7.4 | 8.3 | 8398.8 |
| 33 | 3.1 | 6.4 | 7.5 | 8.6 | 6751.5 |
| 34 | 4.0 | 6.7 | 7.6 | 8.4 | 8424.0 |
| 35 | 3.0 | 5.6 | 6.5 | 7.4 | 8385.8 |
| 36 | 2.5 | 5.3 | 6.3 | 7.2 | 7787.1 |
| 37 | 4.2 | 6.7 | 7.5 | 8.3 | 9108.2 |
| 38 | 3.2 | 6.1 | 7.1 | 8.0 | 7704.6 |
| 39 | 4.7 | 7.3 | 8.2 | 9.1 | 8353.2 |
| 40 | 3.3 | 6.1 | 7.1 | 8.0 | 7965.3 |

**Table B 7: 4 POINT MODULUS OF ELASTICITY (*Triplochiton Scleroxylon*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 5.6 | 10.7 | 12.4 | 14.1 | 4388.3 |
| 2 | 5.5 | 9.1 | 10.3 | 11.6 | 6146.0 |
| 3 | 4.8 | 8.6 | 9.8 | 11.1 | 5870.2 |
| 4 | 4.8 | 8.7 | 10.0 | 11.3 | 5707.4 |
| 5 | 4.3 | 6.4 | 7.1 | 7.8 | 10594.4 |
| 6 | 4.9 | 9.8 | 11.4 | 13.0 | 4601.3 |
| 7 | 5.3 | 7.6 | 8.3 | 9.1 | 9716.3 |
| 8 | 5.1 | 8.7 | 9.9 | 11.1 | 6215.2 |
| 9 | 4.7 | 11.2 | 13.3 | 15.5 | 3436.4 |
| 10 | 4.7 | 7.7 | 8.7 | 9.7 | 7379.1 |
| 11 | 5.1 | 8.6 | 9.7 | 10.9 | 6408.5 |
| 12 | 4.6 | 8.1 | 9.2 | 10.4 | 6449.2 |
| 13 | 4.2 | 7.1 | 8.1 | 9.0 | 7751.4 |
| 14 | 4.7 | 8.7 | 10.0 | 11.4 | 5560.7 |
| 15 | 4.7 | 8.7 | 10.0 | 11.3 | 5635.6 |
| 16 | 5.2 | 10.3 | 12.0 | 13.7 | 4341.9 |
| 17 | 4.4 | 7.8 | 8.9 | 10.0 | 6745.6 |
| 18 | 5.4 | 8.8 | 10.0 | 11.1 | 6553.8 |
| 19 | 5.3 | 10.9 | 12.8 | 14.7 | 3982.3 |
| 20 | 4.7 | 9.4 | 10.9 | 12.5 | 4811.1 |
| 21 | 5.2 | 12.0 | 14.3 | 16.5 | 3291.2 |
| 22 | 5.5 | 9.0 | 10.1 | 11.3 | 6437.6 |
| 23 | 5.1 | 9.0 | 10.3 | 11.6 | 5773.8 |
| 24 | 4.8 | 8.8 | 10.1 | 11.4 | 5678.0 |
| 25 | 4.6 | 7.6 | 8.6 | 9.6 | 7428.9 |
| 26 | 5.1 | 7.8 | 8.7 | 9.6 | 8289.8 |
| 27 | 5.6 | 8.6 | 9.6 | 10.6 | 7395.4 |
| 28 | 5.4 | 8.2 | 9.1 | 10.0 | 8113.6 |
| 19 | 4.2 | 10.8 | 13.0 | 15.2 | 3368.8 |
| 30 | 6.0 | 9.6 | 10.8 | 12.0 | 6260.4 |
| 31 | 5.2 | 9.7 | 11.3 | 12.8 | 4883.0 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 5.4 | 8.0 | 8.8 | 9.7 | 8623.8 |
| 33 | 5.1 | 8.3 | 9.3 | 10.4 | 7035.9 |
| 34 | 4.7 | 8.1 | 9.2 | 10.3 | 6610.3 |
| 35 | 4.4 | 7.6 | 8.6 | 9.7 | 7141.4 |
| 36 | 5.9 | 12.3 | 14.4 | 16.5 | 3510.2 |
| 37 | 4.5 | 8.4 | 9.7 | 11.0 | 5678.1 |
| 38 | 4.9 | 8.6 | 9.8 | 11.0 | 6131.3 |
| 39 | 3.8 | 7.8 | 9.1 | 10.5 | 5622.6 |
| 40 | 4.6 | 8.4 | 9.6 | 10.9 | 5943.2 |

**Table B 8: 4 POINT MODULUS OF ELASTICITY (*Terminalia Ivorensis*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 3.4 | 5.3 | 5.9 | 6.5 | 11913.0 |
| 2 | 3.5 | 5.5 | 6.2 | 6.8 | 11377.3 |
| 3 | 3.1 | 4.8 | 5.3 | 5.9 | 13621.0 |
| 4 | 3.7 | 5.2 | 5.7 | 6.3 | 14286.4 |
| 5 | 3.3 | 5.5 | 6.2 | 6.9 | 10394.3 |
| 6 | 3.8 | 6.1 | 6.9 | 7.6 | 9622.2 |
| 7 | 3.3 | 5.4 | 6.1 | 6.8 | 10461.9 |
| 8 | 3.9 | 5.2 | 5.6 | 6.1 | 17141.3 |
| 9 | 2.9 | 4.4 | 4.9 | 5.4 | 14791.3 |
| 10 | 3.0 | 4.6 | 5.1 | 5.7 | 13978.2 |
| 11 | 3.7 | 5.2 | 5.7 | 6.2 | 14897.3 |
| 12 | 3.3 | 4.7 | 5.2 | 5.7 | 15812.1 |
| 13 | 3.2 | 5.2 | 5.9 | 6.5 | 11204.9 |
| 14 | 3.5 | 6.6 | 7.7 | 8.7 | 7199.1 |
| 15 | 2.7 | 4.1 | 4.6 | 5.1 | 15986.9 |
| 16 | 2.7 | 4.9 | 5.6 | 6.3 | 10282.8 |
| 17 | 3.7 | 5.1 | 5.5 | 6.0 | 16742.1 |
| 18 | 2.9 | 4.8 | 5.5 | 6.2 | 11252.6 |
| 19 | 4.0 | 6.4 | 7.2 | 8.0 | 9418.4 |
| 20 | 3.1 | 5.1 | 5.8 | 6.4 | 11344.6 |
| 21 | 2.3 | 4.1 | 4.6 | 5.2 | 12769.3 |
| 22 | 4.3 | 6.6 | 7.3 | 8.1 | 9810.8 |
| 23 | 4.5 | 5.9 | 6.3 | 6.8 | 16165.5 |
| 24 | 3.0 | 5.1 | 5.8 | 6.4 | 10969.0 |
| 25 | 3.3 | 4.7 | 5.2 | 5.6 | 16237.5 |
| 26 | 2.4 | 4.3 | 4.9 | 5.5 | 12356.7 |
| 27 | 3.9 | 6.2 | 7.0 | 7.7 | 9705.7 |
| 28 | 2.3 | 3.9 | 4.5 | 5.0 | 13932.9 |
| 19 | 3.6 | 5.9 | 6.7 | 7.5 | 9584.4 |
| 30 | 3.1 | 5.0 | 5.6 | 6.2 | 11881.5 |
| 31 | 4.2 | 5.8 | 6.4 | 7.0 | 13444.8 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 3.2 | 5.5 | 6.3 | 7.0 | 9751.3 |
| 33 | 3.4 | 5.1 | 5.6 | 6.2 | 13576.5 |
| 34 | 3.2 | 5.0 | 5.6 | 6.1 | 12771.5 |
| 35 | 2.5 | 5.5 | 6.6 | 7.6 | 7237.9 |
| 36 | 3.5 | 6.9 | 8.0 | 9.1 | 6639.2 |
| 37 | 2.8 | 4.9 | 5.6 | 6.3 | 10650.3 |
| 38 | 4.1 | 5.9 | 6.5 | 7.1 | 12444.8 |
| 39 | 3.6 | 5.5 | 6.1 | 6.8 | 11841.4 |
| 40 | 2.6 | 4.4 | 4.9 | 5.5 | 12943.9 |

**Table B 9: 4 POINT MODULUS OF ELASTICITY (*Terminalia Superba*)**

| TEST PIECE MARK | CENTRAL DELFECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 2.7 | 5.0 | 5.7 | 6.4 | 10089.0 |
| 2 | 3.1 | 5.0 | 5.7 | 6.3 | 11556.2 |
| 3 | 3.1 | 5.5 | 6.2 | 7.0 | 9623.2 |
| 4 | 2.6 | 4.6 | 5.2 | 5.9 | 11304.0 |
| 5 | 2.6 | 4.7 | 5.4 | 6.1 | 10609.2 |
| 6 | 2.3 | 4.8 | 5.6 | 6.5 | 9048.4 |
| 7 | 3.2 | 5.1 | 5.8 | 6.4 | 11662.5 |
| 8 | 2.9 | 4.9 | 5.6 | 6.3 | 11135.2 |
| 9 | 3.6 | 5.4 | 6.0 | 6.6 | 12123.4 |
| 10 | 2.8 | 4.7 | 5.3 | 5.9 | 11941.3 |
| 11 | 2.9 | 4.4 | 4.9 | 5.3 | 15351.3 |
| 12 | 4.3 | 6.3 | 6.9 | 7.6 | 11433.6 |
| 13 | 3.7 | 5.2 | 5.7 | 6.2 | 15125.5 |
| 14 | 2.4 | 3.9 | 4.4 | 5.0 | 14313.7 |
| 15 | 3.6 | 6.5 | 7.4 | 8.4 | 7822.9 |
| 16 | 3.1 | 4.6 | 5.0 | 5.5 | 15482.3 |
| 17 | 3.1 | 5.6 | 6.4 | 7.2 | 8942.7 |
| 18 | 3.1 | 5.1 | 5.8 | 6.5 | 10962.3 |
| 19 | 4.2 | 6.3 | 7.0 | 7.7 | 10790.8 |
| 20 | 2.5 | 4.4 | 5.1 | 5.7 | 11808.7 |
| 21 | 3.5 | 4.8 | 5.2 | 5.7 | 16957.6 |
| 22 | 3.9 | 6.6 | 7.6 | 8.5 | 8045.3 |
| 23 | 2.5 | 4.5 | 5.2 | 5.9 | 11221.0 |
| 24 | 2.2 | 4.7 | 5.6 | 6.4 | 8898.1 |
| 25 | 2.6 | 4.0 | 4.5 | 5.0 | 15599.3 |
| 26 | 3.0 | 5.8 | 6.7 | 7.6 | 8233.8 |
| 27 | 4.7 | 6.4 | 7.0 | 7.6 | 13119.1 |
| 28 | 3.0 | 4.9 | 5.6 | 6.2 | 11633.7 |
| 19 | 2.7 | 4.5 | 5.1 | 5.7 | 12697.3 |
| 30 | 3.3 | 5.4 | 6.0 | 6.7 | 10811.6 |
| 31 | 3.1 | 5.3 | 6.0 | 6.8 | 10075.6 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 2.8 | 4.4 | 4.9 | 5.5 | 13762.6 |
| 33 | 3.1 | 4.6 | 5.1 | 5.6 | 14900.2 |
| 34 | 3.5 | 5.1 | 5.6 | 6.2 | 14110.7 |
| 35 | 2.9 | 5.5 | 6.4 | 7.2 | 8610.8 |
| 36 | 3.2 | 6.9 | 8.1 | 9.3 | 6078.1 |
| 37 | 3.5 | 5.3 | 5.9 | 6.5 | 12304.9 |
| 38 | 3.1 | 4.8 | 5.4 | 6.0 | 13198.7 |
| 39 | 2.4 | 4.4 | 5.1 | 5.8 | 10804.4 |
| 40 | 3.5 | 5.3 | 5.9 | 6.5 | 12388.5 |

**Table B 10: 4 POINT MODULUS OF ELASTICITY (*Lophira Alata*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 5000 N | 8000 N | 10000 N | |
| 1 | 2.1 | 3.0 | 3.3 | 3.5 | 26487.2 |
| 2 | 1.4 | 2.4 | 2.7 | 3.1 | 22250.2 |
| 3 | 2.1 | 3.1 | 3.5 | 3.9 | 20852.2 |
| 4 | 2.3 | 3.3 | 3.6 | 3.9 | 22088.6 |
| 5 | 0.9 | 2.0 | 2.4 | 2.7 | 20823.6 |
| 6 | 2.1 | 3.0 | 3.3 | 3.6 | 24960.7 |
| 7 | 1.5 | 2.8 | 3.2 | 3.7 | 16998.2 |
| 8 | 2.2 | 3.2 | 3.5 | 3.9 | 21894.6 |
| 9 | 1.1 | 1.9 | 2.1 | 2.4 | 27892.3 |
| 10 | 1.6 | 2.6 | 3.0 | 3.3 | 21570.4 |
| 11 | 1.5 | 2.4 | 2.8 | 3.1 | 23500.4 |
| 12 | 1.1 | 2.0 | 2.3 | 2.6 | 25060.6 |
| 13 | 2.3 | 3.2 | 3.6 | 3.9 | 22815.1 |
| 14 | 0.8 | 2.0 | 2.4 | 2.8 | 19386.7 |
| 15 | 1.8 | 2.7 | 3.0 | 3.3 | 25298.4 |
| 16 | 1.2 | 2.1 | 2.5 | 2.8 | 23562.8 |
| 17 | 3.0 | 4.2 | 4.6 | 5.0 | 18271.4 |
| 18 | 1.4 | 2.3 | 2.6 | 2.9 | 24186.4 |
| 19 | 1.4 | 2.4 | 2.7 | 3.1 | 22453.9 |
| 20 | 2.0 | 3.0 | 3.3 | 3.7 | 22919.3 |
| 21 | 2.7 | 3.4 | 3.7 | 3.9 | 29686.2 |
| 22 | 1.5 | 2.5 | 2.9 | 3.2 | 21546.2 |
| 23 | 2.2 | 3.2 | 3.5 | 3.9 | 22120.4 |
| 24 | 1.5 | 2.5 | 2.8 | 3.1 | 22288.1 |
| 25 | 1.3 | 2.2 | 2.6 | 2.9 | 23467.7 |
| 26 | 1.5 | 2.5 | 2.9 | 3.2 | 22105.8 |
| 27 | 1.5 | 2.5 | 2.9 | 3.2 | 22336.2 |
| 28 | 1.8 | 2.9 | 3.2 | 3.5 | 21354.4 |
| 19 | 2.5 | 3.7 | 4.0 | 4.4 | 19940.2 |
| 30 | 2.9 | 3.8 | 4.1 | 4.4 | 24159.8 |
| 31 | 1.1 | 2.0 | 2.3 | 2.6 | 24699.1 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 1.5 | 2.5 | 2.8 | 3.2 | 21602.2 |
| 33 | 0.3 | 1.6 | 2.1 | 2.5 | 16454.1 |
| 34 | 1.6 | 2.7 | 3.1 | 3.4 | 20550.4 |
| 35 | 2.2 | 3.1 | 3.4 | 3.7 | 24402.9 |
| 36 | 1.7 | 2.8 | 3.2 | 3.5 | 20549.5 |
| 37 | 1.6 | 2.4 | 2.7 | 3.0 | 26104.5 |
| 38 | 1.7 | 2.5 | 2.8 | 3.1 | 27087.7 |
| 39 | 1.6 | 2.5 | 2.8 | 3.1 | 25142.4 |
| 40 | 1.8 | 2.9 | 3.2 | 3.6 | 21119.0 |

**APPENDIX C: 3 Point Bending Test Data**

**Table C 1: 3 POINT BENDING STRENGTH (*Alstonia Boonei*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^3$) | ADJUSTED BENDING STRENGTH ($N/mm^3$) |
|---|---|---|---|
| 1 | 5.1 | 36.8 | 27.9 |
| 2 | 6.8 | 49.0 | 37.2 |
| 3 | 6.1 | 43.9 | 33.3 |
| 4 | 5.9 | 42.3 | 32.1 |
| 5 | 5.5 | 39.7 | 30.1 |
| 6 | 5.1 | 36.7 | 27.8 |
| 7 | 5.0 | 36.0 | 27.4 |
| 8 | 5.1 | 36.9 | 28.0 |
| 9 | 5.4 | 39.1 | 29.6 |
| 10 | 4.7 | 33.6 | 25.5 |
| 11 | 3.9 | 28.1 | 21.3 |
| 12 | 7.0 | 50.2 | 38.1 |
| 13 | 4.8 | 34.2 | 26.0 |
| 14 | 3.3 | 23.8 | 18.0 |
| 15 | 5.5 | 39.5 | 30.0 |
| 16 | 4.8 | 34.8 | 26.4 |
| 17 | 4.4 | 31.3 | 23.8 |
| 18 | 5.8 | 41.5 | 31.5 |
| 19 | 4.1 | 29.3 | 22.3 |
| 20 | 6.1 | 43.6 | 33.1 |
| 21 | 4.5 | 32.3 | 24.5 |
| 22 | 6.6 | 47.4 | 36.0 |
| 23 | 3.9 | 28.3 | 21.4 |
| 24 | 5.0 | 36.2 | 27.5 |
| 25 | 6.3 | 45.3 | 34.3 |
| 26 | 4.0 | 28.5 | 21.6 |
| 27 | 5.8 | 41.8 | 31.7 |
| 28 | 6.9 | 49.4 | 37.5 |
| 19 | 3.4 | 24.5 | 18.6 |
| 30 | 6.2 | 44.5 | 33.8 |
| 31 | 4.8 | 34.7 | 26.4 |
| 32 | 5.1 | 36.8 | 27.9 |

| | | | |
|---|---|---|---|
| 33 | 6.5 | 47.0 | 35.7 |
| 34 | 5.6 | 40.1 | 30.4 |
| 35 | 5.3 | 37.8 | 28.7 |
| 36 | 4.2 | 30.3 | 23.0 |
| 37 | 5.1 | 36.9 | 28.0 |
| 38 | 5.3 | 37.9 | 28.7 |
| 39 | 5.0 | 36.2 | 27.5 |
| 40 | 6.0 | 43.2 | 32.8 |

**Table C 2: 3 POINT BENDING STRENGTH (*Triplochiton Scleroxylon*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 8.4 | 60.5 | 45.9 |
| 2 | 7.4 | 53.2 | 40.3 |
| 3 | 7.0 | 50.1 | 38.0 |
| 4 | 6.3 | 45.4 | 34.4 |
| 5 | 7.8 | 55.9 | 42.4 |
| 6 | 7.2 | 52.0 | 39.5 |
| 7 | 6.5 | 46.9 | 35.6 |
| 8 | 6.3 | 45.4 | 34.4 |
| 9 | 7.9 | 56.7 | 43.0 |
| 10 | 8.1 | 58.1 | 44.1 |
| 11 | 5.7 | 41.2 | 31.2 |
| 12 | 7.5 | 54.3 | 41.2 |
| 13 | 7.7 | 55.6 | 42.2 |
| 14 | 6.2 | 44.9 | 34.0 |
| 15 | 6.1 | 44.1 | 33.5 |
| 16 | 7.3 | 52.6 | 39.9 |
| 17 | 6.5 | 47.1 | 35.7 |
| 18 | 7.5 | 53.8 | 40.9 |
| 19 | 7.4 | 53.2 | 40.3 |
| 20 | 7.0 | 50.6 | 38.4 |
| 21 | 6.5 | 46.5 | 35.3 |
| 22 | 6.9 | 49.6 | 37.6 |
| 23 | 7.2 | 51.6 | 39.1 |
| 24 | 6.3 | 45.6 | 34.6 |
| 25 | 6.6 | 47.5 | 36.0 |
| 26 | 8.1 | 58.0 | 44.0 |
| 27 | 7.8 | 55.8 | 42.4 |
| 28 | 8.1 | 58.4 | 44.3 |
| 19 | 6.5 | 46.9 | 35.6 |
| 30 | 5.9 | 42.1 | 32.0 |
| 31 | 7.2 | 51.8 | 39.3 |
| 32 | 7.0 | 50.0 | 38.0 |
| 33 | 6.9 | 49.6 | 37.7 |

| | | | |
|---|---|---|---|
| 34 | 7.4 | 53.5 | 40.6 |
| 35 | 6.8 | 48.8 | 37.0 |
| 36 | 7.0 | 50.1 | 38.0 |
| 37 | 7.7 | 55.8 | 42.3 |
| 38 | 7.1 | 51.1 | 38.8 |
| 39 | 7.3 | 52.6 | 39.9 |
| 40 | 7.6 | 54.4 | 41.2 |

**Table C 3: 3 POINT BENDING STRENGTH (*Terminalia Ivorensis*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 13.3 | 96.0 | 72.8 |
| 2 | 5.9 | 42.6 | 32.3 |
| 3 | 10.2 | 73.6 | 55.8 |
| 4 | 6.2 | 44.8 | 34.0 |
| 5 | 10.8 | 77.7 | 58.9 |
| 6 | 10.9 | 78.4 | 59.5 |
| 7 | 7.4 | 53.3 | 40.4 |
| 8 | 11.3 | 81.3 | 61.7 |
| 9 | 8.0 | 57.4 | 43.6 |
| 10 | 8.3 | 59.6 | 45.2 |
| 11 | 9.7 | 70.1 | 53.2 |
| 12 | 9.0 | 64.6 | 49.0 |
| 13 | 6.4 | 46.1 | 35.0 |
| 14 | 8.8 | 63.7 | 48.3 |
| 15 | 10.8 | 77.6 | 58.9 |
| 16 | 6.4 | 46.0 | 34.9 |
| 17 | 9.1 | 65.9 | 50.0 |
| 18 | 8.7 | 62.6 | 47.5 |
| 19 | 9.4 | 68.0 | 51.6 |
| 20 | 7.5 | 53.7 | 40.7 |
| 21 | 9.0 | 65.0 | 49.3 |
| 22 | 9.5 | 68.4 | 51.9 |
| 23 | 8.2 | 59.0 | 44.8 |
| 24 | 9.7 | 70.1 | 53.2 |
| 25 | 9.3 | 67.1 | 50.9 |
| 26 | 12.5 | 90.3 | 68.5 |
| 27 | 7.0 | 50.3 | 38.2 |
| 28 | 7.4 | 53.1 | 40.3 |
| 19 | 7.6 | 54.8 | 41.6 |
| 30 | 8.1 | 58.6 | 44.5 |
| 31 | 7.6 | 55.0 | 41.7 |
| 32 | 7.8 | 55.9 | 42.4 |
| 33 | 9.3 | 67.2 | 51.0 |

| | | | |
|---|---|---|---|
| 34 | 5.1 | 36.8 | 27.9 |
| 35 | 10.4 | 75.2 | 57.1 |
| 36 | 12.1 | 87.4 | 66.3 |
| 37 | 8.9 | 63.7 | 48.3 |
| 38 | 8.6 | 62.1 | 47.1 |
| 39 | 11.8 | 85.0 | 64.5 |
| 40 | 10.7 | 77.3 | 58.7 |

**Table C 4: 3 POINT BENDING STRENGTH (*Terminalia Superba*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 12.2 | 87.7 | 66.5 |
| 2 | 9.3 | 66.9 | 50.7 |
| 3 | 12.1 | 86.9 | 65.9 |
| 4 | 11.3 | 81.6 | 61.9 |
| 5 | 9.7 | 69.7 | 52.9 |
| 6 | 14.8 | 106.7 | 81.0 |
| 7 | 12.8 | 92.1 | 69.9 |
| 8 | 11.3 | 81.6 | 61.9 |
| 9 | 12.0 | 86.3 | 65.5 |
| 10 | 10.7 | 76.9 | 58.4 |
| 11 | 10.6 | 76.3 | 57.9 |
| 12 | 9.0 | 65.1 | 49.4 |
| 13 | 13.5 | 97.3 | 73.8 |
| 14 | 7.2 | 51.8 | 39.3 |
| 15 | 12.6 | 90.6 | 68.7 |
| 16 | 9.1 | 65.8 | 49.9 |
| 17 | 11.2 | 80.3 | 61.0 |
| 18 | 9.7 | 69.7 | 52.9 |
| 19 | 10.6 | 76.4 | 57.9 |
| 20 | 8.2 | 59.2 | 44.9 |
| 21 | 9.8 | 70.4 | 53.4 |
| 22 | 11.9 | 86.0 | 65.3 |
| 23 | 14.7 | 105.7 | 80.2 |
| 24 | 10.6 | 76.3 | 57.9 |
| 25 | 9.5 | 68.3 | 51.9 |
| 26 | 8.2 | 59.3 | 45.0 |
| 27 | 10.6 | 76.7 | 58.2 |
| 28 | 9.6 | 69.3 | 52.6 |
| 29 | 12.5 | 89.8 | 68.1 |
| 30 | 12.9 | 93.1 | 70.6 |
| 31 | 11.2 | 80.5 | 61.1 |
| 32 | 10.9 | 78.6 | 59.7 |
| 33 | 12.4 | 89.2 | 67.7 |

| | | | |
|---|---|---|---|
| 34 | 14.5 | 104.7 | 79.4 |
| 35 | 12.5 | 89.7 | 68.1 |
| 36 | 11.3 | 81.6 | 61.9 |
| 37 | 9.5 | 68.7 | 52.1 |
| 38 | 10.1 | 72.5 | 55.0 |
| 39 | 12.6 | 90.6 | 68.8 |
| 40 | 11.2 | 80.7 | 61.2 |

**Table C 5: 3 POINT BENDING STRENGTH (*Lophira Alata*)**

| TEST PIECE MARK | FAILURE LOAD (kN) | BENDING STRENGTH ($N/mm^2$) | ADJUSTED BENDING STRENGTH ($N/mm^2$) |
|---|---|---|---|
| 1 | 15.3 | 110.4 | 83.8 |
| 2 | 12.8 | 91.8 | 69.7 |
| 3 | 17.0 | 122.4 | 92.8 |
| 4 | 12.3 | 88.8 | 67.4 |
| 5 | 10.0 | 72.2 | 54.7 |
| 6 | 12.0 | 86.7 | 65.8 |
| 7 | 12.2 | 87.9 | 66.7 |
| 8 | 10.7 | 76.8 | 58.3 |
| 9 | 14.1 | 101.6 | 77.1 |
| 10 | 8.7 | 62.8 | 47.7 |
| 11 | 10.8 | 77.5 | 58.8 |
| 12 | 15.1 | 108.8 | 82.5 |
| 13 | 14.1 | 101.5 | 77.0 |
| 14 | 10.4 | 75.2 | 57.0 |
| 15 | 8.4 | 60.5 | 45.9 |
| 16 | 13.7 | 98.6 | 74.8 |
| 17 | 10.3 | 74.0 | 56.1 |
| 18 | 10.9 | 78.6 | 59.7 |
| 19 | 14.3 | 103.2 | 78.3 |
| 20 | 13.6 | 97.7 | 74.1 |
| 21 | 14.6 | 105.4 | 80.0 |
| 22 | 12.2 | 87.7 | 66.5 |
| 23 | 15.8 | 113.8 | 86.3 |
| 24 | 9.7 | 69.8 | 52.9 |
| 25 | 13.9 | 100.1 | 75.9 |
| 26 | 12.0 | 86.6 | 65.7 |
| 27 | 9.3 | 67.1 | 50.9 |
| 28 | 10.7 | 77.2 | 58.6 |
| 19 | 15.4 | 111.1 | 84.3 |
| 30 | 15.8 | 113.5 | 86.1 |
| 31 | 14.5 | 104.7 | 79.5 |
| 32 | 16.2 | 116.9 | 88.7 |
| 33 | 11.5 | 83.1 | 63.0 |
| 34 | 10.2 | 73.1 | 55.4 |

| | | | |
|---|---|---|---|
| 35 | 12.3 | 88.4 | 67.1 |
| 36 | 16.6 | 119.4 | 90.6 |
| 37 | 12.1 | 87.0 | 66.0 |
| 38 | 12.7 | 91.3 | 69.2 |
| 39 | 13.9 | 100.1 | 76.0 |
| 40 | 14.3 | 103.3 | 78.4 |

**Table C 6: 3 POINT MODULUS OF ELASTICITY (*Alstonia Boonei*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 1500 N | 2000 N | 3000 N | 5000 N | |
| 1 | 5.5 | 8.9 | 10.1 | 11.2 | 6488.9 |
| 2 | 5.6 | 9.3 | 10.5 | 11.7 | 6118.7 |
| 3 | 5.2 | 8.8 | 10.1 | 11.3 | 6197.4 |
| 4 | 5.9 | 9.7 | 10.9 | 12.2 | 5923.7 |
| 5 | 5.3 | 8.1 | 9.0 | 10.0 | 7996.7 |
| 6 | 6.3 | 9.5 | 10.6 | 11.7 | 6884.0 |
| 7 | 5.6 | 8.7 | 9.7 | 10.7 | 7306.7 |
| 8 | 4.9 | 7.7 | 8.7 | 9.7 | 7769.1 |
| 9 | 5.3 | 8.1 | 9.0 | 9.9 | 8042.2 |
| 10 | 5.1 | 8.1 | 9.1 | 10.1 | 7458.6 |
| 11 | 4.8 | 8.0 | 9.0 | 10.1 | 7135.8 |
| 12 | 5.0 | 7.8 | 8.8 | 9.7 | 7833.8 |
| 13 | 6.6 | 10.1 | 11.3 | 12.4 | 6360.6 |
| 14 | 6.3 | 9.4 | 10.5 | 11.5 | 7070.4 |
| 15 | 5.5 | 8.4 | 9.4 | 10.4 | 7528.8 |
| 16 | 5.7 | 8.6 | 9.6 | 10.6 | 7682.2 |
| 17 | 5.5 | 8.5 | 9.5 | 10.5 | 7500.9 |
| 18 | 5.6 | 8.2 | 9.1 | 10.0 | 8574.7 |
| 19 | 5.8 | 8.6 | 9.5 | 10.4 | 8011.4 |
| 20 | 5.5 | 8.9 | 10.0 | 11.1 | 6691.0 |
| 21 | 5.9 | 9.2 | 10.4 | 11.5 | 6710.4 |
| 22 | 5.7 | 8.7 | 9.8 | 10.8 | 7291.4 |
| 23 | 6.9 | 9.9 | 10.9 | 11.9 | 7425.9 |
| 24 | 5.7 | 9.0 | 10.1 | 11.2 | 6714.8 |
| 25 | 4.7 | 7.7 | 8.7 | 9.7 | 7452.0 |
| 26 | 6.5 | 9.7 | 10.7 | 11.8 | 7054.2 |
| 27 | 5.5 | 8.2 | 9.1 | 10.0 | 8139.7 |
| 28 | 5.9 | 9.0 | 10.1 | 11.1 | 7092.5 |
| 19 | 5.6 | 9.0 | 10.2 | 11.3 | 6501.4 |
| 30 | 5.5 | 8.8 | 9.9 | 11.0 | 6790.0 |
| 31 | 6.3 | 9.1 | 10.0 | 10.9 | 7959.9 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 5.6 | 8.5 | 9.5 | 10.4 | 7765.2 |
| 33 | 6.2 | 9.3 | 10.3 | 11.3 | 7294.7 |
| 34 | 5.4 | 8.4 | 9.5 | 10.5 | 7385.0 |
| 35 | 6.1 | 8.9 | 9.8 | 10.7 | 7983.1 |
| 36 | 6.3 | 9.2 | 10.1 | 11.0 | 7844.3 |
| 37 | 5.7 | 8.8 | 9.8 | 10.9 | 7223.8 |
| 38 | 5.9 | 8.8 | 9.7 | 10.7 | 7877.4 |
| 39 | 6.2 | 8.9 | 9.9 | 10.8 | 8074.7 |
| 40 | 6.4 | 9.5 | 10.5 | 11.5 | 7229.4 |

**Table C 7: 3 POINT MODULUS OF ELASTICITY (*Triplochiton Scleroxylon*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 1000 N | 2000 N | 3000 N | 4000 N | |
| 1 | 8.5 | 12.5 | 13.9 | 15.2 | 5537.2 |
| 2 | 8.2 | 11.5 | 12.7 | 13.8 | 6634.9 |
| 3 | 8.0 | 11.3 | 12.4 | 13.5 | 6799.7 |
| 4 | 8.6 | 11.8 | 12.8 | 13.9 | 7092.0 |
| 5 | 8.3 | 11.0 | 11.9 | 12.8 | 8264.1 |
| 6 | 8.8 | 11.7 | 12.7 | 13.7 | 7658.2 |
| 7 | 8.4 | 11.5 | 12.6 | 13.7 | 7050.7 |
| 8 | 8.2 | 11.1 | 12.1 | 13.0 | 7760.4 |
| 9 | 8.3 | 10.7 | 11.5 | 12.3 | 9364.6 |
| 10 | 8.7 | 11.9 | 13.0 | 14.1 | 6908.5 |
| 11 | 8.2 | 12.0 | 13.3 | 14.6 | 5814.8 |
| 12 | 8.9 | 11.4 | 12.2 | 13.1 | 8892.2 |
| 13 | 8.3 | 11.5 | 12.5 | 13.6 | 7060.2 |
| 14 | 7.7 | 11.6 | 12.9 | 14.2 | 5683.2 |
| 15 | 8.7 | 12.4 | 13.7 | 14.9 | 5983.5 |
| 16 | 8.9 | 11.7 | 12.6 | 13.6 | 7905.5 |
| 17 | 8.2 | 11.3 | 12.4 | 13.5 | 7042.2 |
| 18 | 8.7 | 11.5 | 12.4 | 13.4 | 8045.1 |
| 19 | 8.3 | 11.3 | 12.3 | 13.3 | 7548.3 |
| 20 | 8.5 | 11.3 | 12.3 | 13.2 | 7911.5 |
| 21 | 8.9 | 12.3 | 13.5 | 14.7 | 6410.1 |
| 22 | 7.8 | 10.8 | 11.8 | 12.8 | 7485.8 |
| 23 | 8.3 | 10.8 | 11.6 | 12.5 | 9047.9 |
| 24 | 8.5 | 11.7 | 12.8 | 13.9 | 6871.0 |
| 25 | 7.8 | 12.0 | 13.3 | 14.7 | 5424.6 |
| 26 | 8.6 | 12.0 | 13.1 | 14.3 | 6583.5 |
| 27 | 8.2 | 11.1 | 12.1 | 13.1 | 7533.0 |
| 28 | 8.3 | 11.2 | 12.2 | 13.1 | 7675.7 |
| 19 | 8.5 | 12.5 | 13.9 | 15.3 | 5470.6 |
| 30 | 7.6 | 12.4 | 14.1 | 15.7 | 4638.2 |
| 31 | 8.5 | 11.8 | 12.8 | 13.9 | 6916.5 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 7.7 | 10.2 | 11.0 | 11.8 | 9014.7 |
| 33 | 9.2 | 12.4 | 13.4 | 14.5 | 6978.2 |
| 34 | 7.5 | 9.9 | 10.7 | 11.5 | 9183.7 |
| 35 | 8.2 | 12.2 | 13.5 | 14.8 | 5626.6 |
| 36 | 8.7 | 12.5 | 13.7 | 15.0 | 5873.6 |
| 37 | 8.7 | 12.7 | 14.0 | 15.4 | 5576.9 |
| 38 | 8.4 | 14.2 | 16.2 | 18.1 | 3853.4 |
| 39 | 8.8 | 13.3 | 14.7 | 16.2 | 5056.8 |
| 40 | 8.3 | 11.5 | 12.6 | 13.7 | 6979.4 |

**Table C 8: 3 POINT MODULUS OF ELASTICITY (*Terminalia Ivorensis*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 4.8 | 6.8 | 7.5 | 8.2 | 10979.7 |
| 2 | 5.9 | 7.8 | 8.5 | 9.1 | 11463.1 |
| 3 | 5.3 | 6.9 | 7.4 | 7.9 | 14182.8 |
| 4 | 4.3 | 6.2 | 6.9 | 7.5 | 11693.1 |
| 5 | 5.1 | 6.9 | 7.5 | 8.1 | 12209.7 |
| 6 | 5.4 | 7.6 | 8.3 | 9.0 | 10178.7 |
| 7 | 6.1 | 8.1 | 8.7 | 9.4 | 11613.8 |
| 8 | 5.7 | 7.6 | 8.2 | 8.8 | 12102.5 |
| 9 | 4.5 | 6.1 | 6.6 | 7.1 | 14133.3 |
| 10 | 5.2 | 7.0 | 7.6 | 8.2 | 12352.2 |
| 11 | 5.0 | 6.8 | 7.4 | 8.0 | 12667.0 |
| 12 | 4.8 | 6.5 | 7.1 | 7.6 | 13215.5 |
| 13 | 4.9 | 7.3 | 8.1 | 9.0 | 9098.1 |
| 14 | 5.0 | 7.7 | 8.5 | 9.4 | 8441.6 |
| 15 | 4.7 | 6.8 | 7.5 | 8.2 | 10695.9 |
| 16 | 4.5 | 6.3 | 6.9 | 7.5 | 12305.9 |
| 17 | 4.5 | 6.7 | 7.5 | 8.2 | 10176.1 |
| 18 | 4.8 | 6.5 | 7.1 | 7.7 | 13151.3 |
| 19 | 4.8 | 7.1 | 7.8 | 8.6 | 9944.9 |
| 20 | 5.0 | 6.7 | 7.2 | 7.8 | 13330.3 |
| 21 | 4.8 | 6.8 | 7.4 | 8.1 | 11472.6 |
| 22 | 5.9 | 7.7 | 8.3 | 9.0 | 12022.3 |
| 23 | 4.6 | 6.4 | 7.0 | 7.6 | 12310.2 |
| 24 | 4.3 | 6.4 | 7.1 | 7.7 | 10815.0 |
| 25 | 5.1 | 6.9 | 7.5 | 8.1 | 12470.1 |
| 26 | 4.6 | 6.8 | 7.6 | 8.4 | 9838.6 |
| 27 | 4.5 | 6.5 | 7.1 | 7.8 | 11373.4 |
| 28 | 5.4 | 7.1 | 7.7 | 8.3 | 12731.4 |
| 19 | 5.2 | 7.0 | 7.6 | 8.1 | 12865.9 |
| 30 | 4.4 | 6.3 | 6.9 | 7.5 | 11843.7 |
| 31 | 5.7 | 7.6 | 8.2 | 8.8 | 12213.1 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 5.0 | 7.0 | 7.7 | 8.3 | 11246.3 |
| 33 | 5.4 | 7.0 | 7.6 | 8.1 | 13930.0 |
| 34 | 4.7 | 6.7 | 7.4 | 8.1 | 10787.9 |
| 35 | 5.1 | 7.0 | 7.6 | 8.3 | 11644.3 |
| 36 | 4.8 | 6.6 | 7.2 | 7.8 | 12523.0 |
| 37 | 4.8 | 6.6 | 7.2 | 7.9 | 11968.9 |
| 38 | 5.9 | 8.5 | 9.4 | 10.3 | 8519.0 |
| 39 | 4.7 | 6.4 | 6.9 | 7.5 | 13359.2 |
| 40 | 5.0 | 6.9 | 7.5 | 8.1 | 11715.7 |

**Table C 9: 3 POINT MODULUS OF ELASTICITY (*Terminalia Superba*)**

| TEST PIECE MARK | CENTRAL DELFECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 3.8 | 5.7 | 6.4 | 7.0 | 11628.8 |
| 2 | 3.7 | 5.1 | 5.6 | 6.0 | 15610.1 |
| 3 | 3.5 | 5.2 | 5.8 | 6.4 | 12685.9 |
| 4 | 3.5 | 4.9 | 5.4 | 5.9 | 15653.8 |
| 5 | 4.0 | 6.7 | 7.6 | 8.5 | 8165.6 |
| 6 | 5.3 | 7.1 | 7.7 | 8.3 | 12274.6 |
| 7 | 4.2 | 6.0 | 6.5 | 7.1 | 12471.7 |
| 8 | 4.5 | 6.1 | 6.6 | 7.1 | 14614.3 |
| 9 | 3.9 | 5.3 | 5.8 | 6.3 | 16052.9 |
| 10 | 4.1 | 5.7 | 6.2 | 6.8 | 13600.7 |
| 11 | 4.5 | 6.5 | 7.2 | 7.9 | 10737.2 |
| 12 | 4.0 | 5.9 | 6.5 | 7.2 | 11832.6 |
| 13 | 4.7 | 6.3 | 6.8 | 7.4 | 13729.8 |
| 14 | 4.3 | 6.1 | 6.7 | 7.3 | 12193.9 |
| 15 | 3.8 | 5.6 | 6.2 | 6.8 | 12540.6 |
| 16 | 4.7 | 6.4 | 6.9 | 7.5 | 13388.1 |
| 17 | 4.1 | 5.9 | 6.5 | 7.1 | 12227.4 |
| 18 | 4.5 | 6.2 | 6.7 | 7.3 | 13081.5 |
| 19 | 4.4 | 6.4 | 7.1 | 7.8 | 10983.4 |
| 20 | 5.0 | 6.9 | 7.5 | 8.1 | 12176.0 |
| 21 | 5.1 | 6.7 | 7.3 | 7.8 | 13483.4 |
| 22 | 4.3 | 6.2 | 6.8 | 7.4 | 12173.7 |
| 23 | 4.9 | 6.3 | 6.8 | 7.3 | 16026.6 |
| 24 | 4.7 | 6.8 | 7.4 | 8.1 | 11075.4 |
| 25 | 3.3 | 5.1 | 5.7 | 6.3 | 12375.9 |
| 26 | 3.7 | 5.5 | 6.2 | 6.8 | 11892.7 |
| 27 | 3.8 | 5.3 | 5.7 | 6.2 | 15194.9 |
| 28 | 4.2 | 5.9 | 6.5 | 7.1 | 13031.9 |
| 19 | 4.6 | 6.9 | 7.6 | 8.4 | 9712.9 |
| 30 | 4.1 | 6.1 | 6.8 | 7.5 | 11137.9 |
| 31 | 3.9 | 5.2 | 5.7 | 6.1 | 16868.5 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 3.9 | 5.1 | 5.5 | 5.8 | 19291.4 |
| 33 | 4.5 | 6.0 | 6.5 | 7.1 | 14640.9 |
| 34 | 3.8 | 6.4 | 7.2 | 8.1 | 8786.4 |
| 35 | 4.6 | 5.8 | 6.2 | 6.6 | 17850.1 |
| 36 | 4.1 | 5.3 | 5.7 | 6.1 | 19288.4 |
| 37 | 3.8 | 5.3 | 5.8 | 6.4 | 14395.9 |
| 38 | 4.4 | 6.7 | 7.5 | 8.3 | 9683.9 |
| 39 | 3.8 | 5.7 | 6.3 | 7.0 | 11877.8 |
| 40 | 3.9 | 5.4 | 5.9 | 6.3 | 15283.8 |

**Table C 10: 3 POINT MODULUS OF ELASTICITY (*Lophira Alata*)**

| TEST PIECE MARK | CENTRAL DEFLECTION (mm) | | | | MODULUS OF ELASTICITY ($N/mm^2$) |
|---|---|---|---|---|---|
| | 2000 N | 3000 N | 4000 N | 5000 N | |
| 1 | 3.5 | 4.6 | 4.9 | 5.2 | 21639.6 |
| 2 | 3.6 | 4.9 | 5.3 | 5.7 | 18405.6 |
| 3 | 4.0 | 5.4 | 5.8 | 6.3 | 16448.7 |
| 4 | 3.6 | 5.0 | 5.5 | 6.0 | 15833.7 |
| 5 | 3.0 | 4.2 | 4.7 | 5.1 | 17814.9 |
| 6 | 3.1 | 4.4 | 4.8 | 5.2 | 17370.2 |
| 7 | 2.7 | 3.8 | 4.2 | 4.6 | 20028.1 |
| 8 | 3.0 | 4.0 | 4.4 | 4.7 | 21108.1 |
| 9 | 3.5 | 4.6 | 5.0 | 5.4 | 19611.3 |
| 10 | 3.0 | 4.2 | 4.5 | 4.9 | 19947.4 |
| 11 | 2.8 | 4.0 | 4.4 | 4.8 | 19233.5 |
| 12 | 3.8 | 5.1 | 5.6 | 6.0 | 16952.7 |
| 13 | 3.7 | 5.1 | 5.5 | 5.9 | 17133.5 |
| 14 | 2.7 | 4.0 | 4.4 | 4.9 | 17655.1 |
| 15 | 2.9 | 4.3 | 4.8 | 5.3 | 15493.7 |
| 16 | 3.1 | 4.4 | 4.8 | 5.2 | 17646.7 |
| 17 | 3.8 | 4.8 | 5.1 | 5.4 | 22603.2 |
| 18 | 2.8 | 4.2 | 4.7 | 5.1 | 16104.4 |
| 19 | 4.5 | 5.6 | 5.9 | 6.3 | 21007.4 |
| 20 | 2.9 | 4.4 | 4.9 | 5.4 | 14821.3 |
| 21 | 3.4 | 4.7 | 5.1 | 5.5 | 17935.8 |
| 22 | 2.7 | 3.8 | 4.1 | 4.5 | 21063.7 |
| 23 | 2.2 | 3.8 | 4.3 | 4.8 | 14254.9 |
| 24 | 2.6 | 4.2 | 4.7 | 5.3 | 14240.5 |
| 25 | 4.2 | 5.2 | 5.6 | 5.9 | 22594.7 |
| 26 | 2.9 | 4.5 | 5.0 | 5.6 | 13740.0 |
| 27 | 3.2 | 4.4 | 4.8 | 5.2 | 19068.2 |
| 28 | 3.3 | 4.8 | 5.3 | 5.8 | 15276.2 |
| 19 | 3.3 | 4.3 | 4.7 | 5.0 | 21660.0 |
| 30 | 3.4 | 4.9 | 5.3 | 5.8 | 15617.5 |
| 31 | 2.6 | 4.0 | 4.4 | 4.9 | 16826.4 |

| | | | | | |
|---|---|---|---|---|---|
| 32 | 4.3 | 5.6 | 6.1 | 6.5 | 16984.7 |
| 33 | 2.7 | 3.9 | 4.4 | 4.8 | 17876.2 |
| 34 | 3.5 | 4.8 | 5.2 | 5.6 | 17795.8 |
| 35 | 3.5 | 4.7 | 5.1 | 5.4 | 19265.8 |
| 36 | 3.3 | 4.6 | 5.0 | 5.5 | 16701.4 |
| 37 | 3.6 | 5.1 | 5.6 | 6.1 | 14965.8 |
| 38 | 2.5 | 3.5 | 3.9 | 4.3 | 21100.1 |
| 39 | 3.7 | 5.0 | 5.4 | 5.8 | 18017.5 |
| 40 | 3.6 | 4.8 | 5.2 | 5.6 | 18317.1 |

# APPENDIX D: PROGRAM LISTING

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%                                                                        %
%     PROGRAM genehunter.m                                               %
%                                                                        %
%     THE MAIN PROGRAM                                                   %
%                                                                        %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%                                                                        %
%     This is a MATLAB Program for:                                      %
%          1. Reliability assessment of the                              %
%             Eurocode 5 Design Criteria of timber portal frame          %
%          2. Determination of partial safety coefficients               %
%                                                                        %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%                                                                        %
%                written by Engr. JIBRIN MOHAMMED KAURA                  %
%                         PhD/ENG/05540/2008-2009                        %
%                                                                        %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%                                                                        %
%    Part of PhD Program titled 'Reliability-based calibration of       %
%    Eurocode 5 Design Criteria of timber portal frame using             %
%                         GENETIC ALGORITHMS                             %
%                                                                        %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%                                                                        %
%                         MAJOR SUPERVISOR                               %
%                         Dr. I. Abubakar                                %
%                                                                        %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%
% Genetic a;gorithms (GAs) are stochastic, iterative, evolutionary, general
% purpose search strategies based on the principles of population genetics
% and natural selection - the GA strategies employ the notion of mating and
% survival of the fittests to evolve a solution to a population. GA are
% pioneered by Holland, who was interested in natural adaptive systems and
% their simulation in software.
% This research work involve the application of GA in structural
% reliability analysisis. In line with this objective, this computer
% program was written
%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%
% THE FOLLOWING FAILURE CRITERIA WERE CONSIDERED:
% Excessive compressive stresses in the column and rafter members
% Excessive bending stresses in the column and rafter members
% Flexural buckling of column and rafter members
% Failure of the moment resisting connection between column and rafter
% Block shear failure of the column ends at column-foundation connection
% Excessive frame member deflection
% Excessive lateral displacement (drift) of the entire portal frame
% BASED ON THE ABOVE, THIRTY ONE LIMIT STATE STATE EQUATIONS
% WERE FORMULATED BASED ON THE THIRTY ONE POTENTIAL FAILURE MODES
%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%
clear; clc; close all
fid12 = fopen('RESULT_12.m','w');
```

```
fprintf(fid12,'SUBROUTINE FOURTY SEVEN\n');
t = clock;
%
% FLOW CHART OF THE WORKING PRINCIPLE OF A GENETIC ALGORITHM
%
clc
disp('             This is a MATLAB program for               ');
disp('   Reliability-based calibration of three-hinge timber  ');
disp('       Portal frame based on Genetic Algorithms         ');
disp('         Code developed by Engr Mohammed J. K.          ');
disp('      Under the supervision of Dr. Idris Abubakar       ');
disp('---------------------------------------------------------- ');
disp('   1:  Reliability/Plasticity Analysis Routine');
disp('   2:  System Reliability-Based Design ');
disp('   3:  Fire Critical Frame');
disp('   4:  Wind Critical Frame');
disp('   5:  Effect of Varring Load Ratio');
disp('   6:  Effect of Load Contribution');
disp('   7:  Effect of Changing Load Distribution Model');
disp('   8:  Effect of Changing Resistance Distribution Model');
disp('   9:  Reliability-Based Calibration');
disp('  10:  Effect of Varring Factors of Safety (1) ')
disp('  11:  Effect of Varring Factors of Safety (2) ');
disp('  12:  Sway Mode Analysis ');
disp('  13:  Plasticity Check (GA Operators) ');
disp('  14:  System Reliability Check ');
disp('  15:  Reliability_based Design  (Variation of Load Ratio) ');
disp('  16:  Reliability-Based Analysis Routine (Slenderness Effect) ');
disp('  17:  Calibration(Effect of Material Covariance)');
disp('   0:  Exit');
disp('       ');
analysistype = input('   CHOOSE OPTION FROM THE LIST ABOVE: ');
clc
disp('   Type 1: if the default rafter slope is to be used')
disp('   Type 2: if a different rafter slope is to be used')
disp('    ')
s_lop = input('    SELECT OPTION FROM ABOVE');
clc
if(s_lop == 1)
    SLOPE = 19.8;
elseif(s_lop == 2)
SLOPE = input('   input pitch (slope) of the frame rafter in degree');
end
clc
FACTOR = input('   input frame size factor');
clc
[BC,XD,XL,Z,FSPACE,FRAME_LENGTH,...
    xvspacing,xtspacing,xvcolumn,xtcolumn,XN_fastener,...
    no_of_rows,output_filename]= frame_prodata(FACTOR);
switch analysistype
%
case 0 % ---- EXIT
case 1 % ---- RELIABILITY ANALYSIS
%
disp(' The reliability analysis from this program is one')
disp(' of the most important')
disp(' steps for the calibration of the EUROCODE 5 material safety ')
disp(' material safety coefficients for solid timber.')
disp(' It is an important strategy, aimed at investigating the safety')
disp(' variations in the code design design criteria considering')
```

```
disp(' a simple statically determinate "Three hinge portal frame"')
disp(' The aim is to come up with appropriate target reliability')
disp(' required for the code calibration')
disp('       ');
d_limit = 1.0;
clc
disp('   ENTER 1 IF FRAME IS MADE OF SOLID SAWN TIMBER')
disp('   ENTER 2 IF FRAME IS MADE OF GLUE LAMINATED TIMBER')
disp('     ')
timber_type = input('   Select option for timber type');
disp('     ')
clc
disp('   Type 1 if the lab-based material properties are to be used')
disp('   Type 2 if EN 338 and GLULAM stremgth values are to be used')
disp('      ')
m_pro = input('   Select option from above');
if(m_pro == 1)
disp('MATERIAL PROPERTIES:')
disp('         ')
disp('ENTER 1 FOR ALSTONIA BOONEI')
disp('ENTER 2 FOR TRIPLOCHITON SCLEROXYLON')
disp('ENTER 3 FOR TERMINALIA IVORENSIS')
disp('ENTER 4 FOR TERMINALIA SUPERBA')
disp('ENTER 5 FOR LOPHIRA ALATA')
disp('   ')
class = input('   select option from above');
[fmk,fcprk,ftprk,fvk,emean,ecen,...
    rhok] = material_properties(class);
elseif(m_pro == 2)
    clc
disp('MATERIAL PROPERTIES FOR SOLID TIMBER (SOFTWOOD):')
disp('ENTER 1 FOR STRENGTH CLASS C14 ACCORDING TO EN338, 2008')
disp('ENTER 2 FOR STRENGTH CLASS C16 ACCORDING TO EN338, 2008')
disp('ENTER 3 FOR STRENGTH CLASS C18 ACCORDING TO EN338, 2008')
disp('ENTER 4 FOR STRENGTH CLASS C20 ACCORDING TO EN338, 2008')
disp('ENTER 5 FOR STRENGTH CLASS C22 ACCORDING TO EN338, 2008')
disp('ENTER 6 FOR STRENGTH CLASS C24 ACCORDING TO EN338, 2008')
disp('ENTER 7 FOR STRENGTH CLASS C27 ACCORDING TO EN338, 2008')
disp('ENTER 8 FOR STRENGTH CLASS C30 ACCORDING TO EN338, 2008')
disp('ENTER 9 FOR STRENGTH CLASS C35 ACCORDING TO EN338, 2008')
disp('ENTER 10 FOR STRENGTH CLASS C40 ACCORDING TO EN338, 2008')
disp('ENTER 11 FOR STRENGTH CLASS C45 ACCORDING TO EN338, 2008')
disp('ENTER 12 FOR STRENGTH CLASS C50 ACCORDING TO EN338, 2008')
disp('        ')
disp('MATERIAL PROPERTIES FOR SOLID TIMBER (HARDWOOD):')
disp('ENTER 13 FOR STRENGTH CLASS D30 ACCORDING TO EN338, 2008')
disp('ENTER 14 FOR STRENGTH CLASS D35 ACCORDING TO EN338, 2008')
disp('ENTER 15 FOR STRENGTH CLASS D40 ACCORDING TO EN338, 2008')
disp('ENTER 16 FOR STRENGTH CLASS D50 ACCORDING TO EN338, 2008')
disp('ENTER 17 FOR STRENGTH CLASS C60 ACCORDING TO EN338, 2008')
disp('ENTER 18 FOR STRENGTH CLASS C70 ACCORDING TO EN338, 2008')
disp('        ')
disp('MATERIAL PROPERTIES FOR GLUE LAMINATED TIMBER:')
disp('ENTER 19 FOR STRENGTH CLASS GL24')
disp('ENTER 20 FOR STRENGTH CLASS GL28')
disp('ENTER 21 FOR STRENGTH CLASS GL32')
disp('ENTER 22 FOR STRENGTH CLASS GL36')
disp('        ')
class_2 = input('   SELECT OPTION FROM ABOVE');
[fmk,fcprk,fcpdk,ftprk,ftpdk,fvk,emean,ecen,eper,gmean,...
```

```
    rhok] = material_properties_2(class_2);
else
    disp('   An invalid entry is made, program may crumble')
disp(' ');
disp('  1: For EN 1990 Load combination 1 (Equation 6.10)');
disp('  2: For EN 1990 Load combination 2 (Equation 6.10 a)');
disp('  3: For EN 1990 Load combination 3 (Equation 6.10 b)');
disp(' ');
ident = input('   Select load combination');
clc
disp('     ')
disp('     THREE SERVICE CLASSES ARE SPECICIFIED IN EUROCODE 5')
disp('     ')
disp('     SERVICE CLASS 1: MOISTURE CONTENT 12% OR LOWER')
disp('     SERVICE CLASS 2: MOISTURE CONTENT ABOVE 12%')
disp('                      AND LOWER THAN 20%')
disp('     SERVICE CLASS 3: MOISTURE CONTENT ABOVE 20%')
disp('     ')
SERVICE_CLASS = input('Select option for timber service class');
clc
disp('      ')
disp('   type 1: If DOL effect models are to be used')
disp('   typr 2: If Eurocode 5 kmod is to be used')
disp('   typr 3: If no modification for load duration')
disp('      ')
DOL_effect = input('   Select option for DOL effect code');
clc
if(DOL_effect == 2)
clc
disp('      ')
disp('      ENTER 1 FOR PERMANENT ACTION')
disp('      ENTER 2 FOR LOMG TERM ACTION')
disp('      ENTER 3 FOR MEDIUM TERM ACTION')
disp('      ENTER 4 FOR SHORT TERM ACTION')
disp('      ENTER 5 FOR INSTANTANEOUS ACTION')
disp('      ')
LOAD_DURATION = input('   Select option for load duration');
    [XKMOD1,XKMOD2,XKMOD3,XKMOD4,XKMOD5] = service(SERVICE_CLASS);
if(LOAD_DURATION==1)
    XKMOD = XKMOD1;
elseif(LOAD_DURATION==2)
    XKMOD = XKMOD2;
elseif(LOAD_DURATION==3)
    XKMOD = XKMOD3;
elseif(LOAD_DURATION==4)
    XKMOD = XKMOD4;
elseif(LOAD_DURATION==5)
    XKMOD = XKMOD5;
end
elseif(DOL_effect == 1)
  timeb = input('   enter load duration in years');
t1 = timeb*360.0*24.0;
disp('   enter 1: for Medison curve')
disp('   enter 2: for garhards damage accumulation model')
disp('   enter 3: for LEFM fracture mechanics model')
disp('   ')
damage = input('   enter damage identifier');
if(damage == 1)
    XKMOD =  (90.4 - (6.3*(log10(t1))))/100;
elseif(damage == 2)
```

```
    XKMOD = (0.951 - (0.063*(log10(t1))));
elseif(damage == 3)
    XKMOD = (1.27)/(sqrt(1.0+((t1/7.2)^(0.112))));
end
elseif(DOL_effect == 3)
    XKMOD = 1.0;
end
fire_effect = 2;
clc
if(fire_effect == 1)
fire_time = input('   Input fire exposure time in minutes');
if(fire_time<=20)
    k_nought = 1;
elseif(fire_time > 20)
    k_nought = (fire_time/20.0);
end
else
    fire_time = 1;
end
load_contribution = 2;
if(load_contribution == 1)
disp('   input dead load contribution')
alpha1 = input(   'minimum 0.1 and maximum is 0.9');
alpha2 = 1-alpha1;
elseif(load_contribution == 2)
    alpha1 = 1;
    alpha2 = 1;
end
BWSPEED = 35.0;
disp('       ')
clc
gen = input('   Input number of population generations');
clc
c_prob = input('   Input crossover probability (> 0 and < = 1');
clc
m_prob = input('   Input mutation probability (> 0 and < = 1');
clc
disp('       ')
clc
cool = input('   Input live to gravity load ratio');
clc
cool2 = 1e-10;
clc
DEPTH_LOW = input('   Lower limit of frame maximum depth in mm');
clc
DEPTH_HIGH = input('   Upper limit of frame maximum depth in mm');
clc
GAMMAM  = input('   Input Material Safety Coefficient');
GAMMAGK = input('   Input Dead Load Safety Coefficient');
GAMMAQK = input('   Input Live Load Safety Coefficient');
GAMMAWK = input('   Input Wind Load Safety Coefficient');
 test_ouputfile = exist('   output_filename','var');
if test_ouputfile == 1
    filename = output_filename;
else
end
disp('     ')
disp('   PRESS ANY KEY TO INITIATE ITERATION:')
disp('   For the computation of reliability indices using')
disp('   Genetic Algorithm (GA)-based First Order Second Moment Approach')
```

```
pause%-------------------------------------------------------------------
m = 0;beco = 0;
for HC = DEPTH_LOW:50:DEPTH_HIGH
    H_rafter  = HC; H_column  = HC;
    H_BASE = 0.7*HC; H_RIDGE = 0.3*HC;
    HRDEF = (0.7*HC/2) + (0.3*HC);
    HCDEF = (0.3*HC/2) + (0.7*HC);
    m = m+1;
    BWSPEED = 35.0;
XP = (pi*SLOPE)/180.0; XH2 = (XL/2.0)*(tan(XP)); XH1 = (Z-XH2);
H_maximum = (((XH1*1000.0)/15.0)+((XL*1000.0)/30.0));
tapered_slope = 3.0;
XP2 = (pi*tapered_slope)/180.0;
HC2 = HC/1000.0;
fck = fcprk; FM = fmk; FMKR = fmk;
vert = 10; nvert = fix(vert); tension_length = 300.0*(nvert-1);
DENK = rhok; BB = BC;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%% FITNESS EVALUTION %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%   The fitness of each failure mode is defined by its safety index.     %
% Failure modes with lowest safety index has the the highest probability %
%   of failure.Less fit individuals are those with high safety indices.  %
%   What is more interested to the analyst is those failure modes that   %
% contribute most to the probability of failure of the entire structure. %
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% The program noe defines population size, the number of genes in the
% chromosomes, the crossover probability, the mutation probability, the
% tournament selection parameter (defining the probability of selecting the
% better individual in a tournament involving two individuals.
QK = 0.75; GK = ((QK*(1-cool))/cool);
WKw = cool2*QK; WKl = 0.75*WKw;
beta1 = 3;
global_load_ratio = (QK + WKw)/(GK + QK + WKw);
if(ident == 1)
    COMBGK = 1.0; COMBQK = 1.0; COMBWK = 0.6;
elseif(ident == 2)
    COMBGK = 1.0; COMBQK = 0.7; COMBWK = 0.6;
elseif(ident == 3)
    COMBGK = 0.85; COMBQK = 1.0; COMBWK = 0.6;
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%% CRITICAL CROSS SECTION %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
xlen = XL/(1.0 + (HC2/H_rafter));
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%% DEPTH OF BEAM AT CRITICAL CROSS SECTION %%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%5%%%%
number_of_genes = 8;
cross_over_probability = c_prob; mutation_probability = m_prob;
tournament_selection_parameter = 0.75;
population_size = 12;
fitness = zeros(population_size,1);
population = initialize_population;
chromosome = population(1,:);
disp(population)
xpi = 1.0; xkdef = 0.6; XU = (XH1*1000)/250; xfr = 0.000000000032;
SECTION_MODULUS_RAFTER = ((BC*H_rafter^2)/6.0)/(1000);
SECTION_MODULUS_COLUMN = ((BC*H_column^2)/6.0)/(1000);
CTR = 1.0; CTS = 1.0;
A1 = GAMMAGK*COMBGK*FSPACE; A2 = GAMMAQK*COMBQK*FSPACE;
```

```
A3 = GAMMAWK*COMBWK*FSPACE;
XA1 = FSPACE; XA2 = FSPACE;
XA3 = FSPACE;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%% GENES DECODING %%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
for j = 1:1:gen
for i = 1:1:population_size
disp(population)
disp('      ')
disp('   "genehunter" is running, ')
disp('   (by Engr. J. K. mohammed) please wait... ')
disp('   MAJOR SUPERVISOR:  Dr. Idris Abubakar')
disp('  ')
    chromosome = population(i,:);
%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%
COVI1     = population(i,1);
COVI2     = population(i,2);
COVI3     = population(i,3);
COVI4     = population(i,4);
COVI5     = population(i,5);
COVI6     = population(i,6);
COVI7     = population(i,7);
COVI8     = population(i,8);
COVI9     = population(i,9);
COVI10     = population(i,10);
COVI11     = population(i,11);
COVI12     = population(i,12);
COVI13     = population(i,13);
COVI14     = population(i,14);
COVI15     = population(i,15);
COVKMOD   = COV1; COVFM     = COV2; COVFMKR   = COV2;
COVFVK    = COV3; COVFC     = COV4; COVE      = COV5;
COVFTPR   = COV6; COVDENK   = COV7; COVGK     = COV8;
COVQK     = COV9; COVWKw    = COV10; COVWKl    = COV11;
COVCTR    = COV12;COVCTS    = COV13; COVU      = COV14;
COVHC     = COV15;
%
DISTYPE1 = 2; DISTYPE2 = 2; DISTYPE3 = 1; DISTYPE4 = 1;
DISTYPE5 = 3; DISTYPE6 = 3; DISTYPE7 = 3; DISTYPE8 = 1;
%
xyi = sqrt((H_BASE^2)/12);
slender = (XH1*1000)/xyi;
B1 = A3*XH1^2/(2.0*XL); B2 = A1*XL/2.0; B3 = A2*XL/2.0;
C1 = (A1*XL)-B2; C2 = (A2*XL)-B3;
D1 = ((A1*XL^2)/(8.0*(XH2+XH1)) - ((C1*XL)/(2.0*(XH2+XH1))));
D2 = ((A2*XL^2)/(8.0*(XH2+XH1)) - ((C2*XL)/(2.0*(XH2+XH1))));
D31 = ((A3*XH1)*((XH1/2.0)+XH2))/(XH2+XH1);
D32 = (B1*XL)/(2.0*(XH2+XH1)); D3 = D31+D32;
D4 = (B1*XL)/(2.0*(XH2+XH1));
E1 = A3*XH1-D3; E2 = A3*XH1-D4;
F1 = (A1*XH1^2)/2.0; F2 = D4*XH1; F3 = D1*XH1; F4 = D2*XH1;
G1 = E1*XH1; G2 = E2*XH1 - (A1*XH1^2/2.0); G3 = D1*XH1; G4 = D2*XH1;
P1 = (A1*XH1 - D3)/cos(XP); P2 = D4/cos(XP); P3 = D1/cos(XP); P4 = D2/cos(XP);
HB = H_column; xlv = xvspacing - XD; xlt = xtspacing - XD;
sumvcol = (xvcolumn+1)*xlv; sumtcol = xtcolumn*xlt;
fracture_length = xtspacing*2; xno1 = 2*nvert;
xno2 = 2*nvert; XT2 = BC; XT1 = BC/2.0;
```

```
Anet_v = fracture_length*XT2; Anet_t = tension_length*XT2;
cap1 = 1.5*Anet_t*ftprk; cap2 = 0.7*Anet_v*fvk; cap3 = 0.9*Anet_v*fvk;
block_shear_capacities = [cap1,cap2,cap3];
Q1 = E1/cos(XP); Q2 = (E2/cos(XP))-(A1*XH1/cos(XP));
Q3 = D1/cos(XP); Q4 = D2/cos(XP);
REV = (B1*WKw)+(B1*WKl)+(B2*GK)+(B3*QK);% vertical reaction at
% support E
fcdpr = 0.6*fcprk/1.3; area1 = REV*1000.0/fcdpr;
RAV = (C1*GK)+(C2*QK)-(B1*WKw)-(B1*WKl);% vertical reaction at
% support A
area2 = RAV*1000.0/fcdpr;
AX1 = 1.0/GAMMAM; AX2 = C1*1000/(H_column*BC);
AX3 = C2*1000/(H_column*BC); AX4 = B1*1000/(H_column*BC);
AX11 = 1.0/GAMMAM; AX12 = P3*1000/(H_rafter*BC); AX13 = P4*1000/(H_rafter*BC);
AX14 = P1*1000/(H_rafter*BC); AX15 = P2*1000/(H_rafter*BC);
AX21 = 1.0/GAMMAM; AX22 = Q3*1000/(H_rafter*BC); AX23 = Q4*1000/(H_rafter*BC);
AX24 = Q1*1000/(H_rafter*BC); AX25 = Q2*1000/(H_rafter*BC);
AC1 = 1.0/GAMMAM; AC2 = B1*1000/(H_column*BC);
AC3 = B2*1000/(H_column*BC); AC4 = B3*1000/(H_column*BC);
AY1 = 1.0/GAMMAM; AY2 = F1*6000000/(BC*(H_column^2));
AY3 = F2*6000000/(BC*(H_column^2)); AY4 = F3*6000000/(BC*(H_column^2));
AY5 = F4*6000000/(BC*(H_column^2));
AYY1 = 1.0/GAMMAM; AYY2 = F1*6000000/(BC*(H_rafter^2));
AYY3 = F2*6000000/(BC*(H_rafter^2)); AYY4 = F3*6000000/(BC*(H_rafter^2));
AYY5 = F4*6000000/(BC*(H_rafter^2));
AD11 = 1.0/GAMMAM; AD21 = G1*6000000/((H_column^2)*BC);
AD31 = G2*6000000/((H_column^2)*BC); AD41 = G3*6000000/((H_column^2)*BC);
AD51 = G4*6000000/((H_column^2)*BC);
ADD11 = 1.0/GAMMAM; ADD21 = G1*6000000/((H_rafter^2)*BC);
ADD31 = G2*6000000/((H_rafter^2)*BC); ADD41 = G3*6000000/((H_rafter^2)*BC);
ADD51 = G4*6000000/((H_rafter^2)*BC);
% COMPUTATION OF BUCKLING LENGTH Blass, 1995
NABR = (P1*WKw - P2*WKl - P3*GK - P4*QK)*1000.0;
NADR = (Q1*WKw + Q2*WKl - Q3*GK - Q4*QK)*1000.0;
FBC = NABR; FCD = NADR;
five_percentile_moe = ecen;
char_density = rhok;
K_ser = ((char_density^1.5)*XD)/23;
K_u = (2*K_ser)/3;
r_outer = (0.5*HC)-(4.0*XD);
r_inner = r_outer-(5.0*XD);
n_outer = (2.0*(3.142)*r_outer)/(6.0*XD);
n_inner = (2.0*(3.142)*r_inner)/(6.0*XD);
K_r = 2*K_u*((n_outer*(r_outer^2)) + (n_inner*(r_inner^2)));
I_effective = (2*(BC/2)*(HC^3))/12;
I_0 = (BC*HC^3)/12;
s_gross = (XL/(2*(cos(XP)))*1000);
s_net = 0.65*s_gross;
h_gross = XH1*1000;
h_net = 0.65*h_gross;
value_1 = ((I_effective*s_gross)/(I_0*h_gross));
value_2 = ((ecen*I_effective)/(h_gross*K_r));
value_3 = 4.0 + (3.2*value_1) + (10*value_2);
%
value_4 = I_0*RAV*1000; value_5 = I_effective*FBC;
value_6 = sqrt(value_4/value_5);
%
value_7 = I_0*REV*1000; value_8 = I_effective*FCD;
value_9 = sqrt(value_7/value_8);
effective_lenght_c = h_gross*sqrt(value_3);
```

```
effective_lenght_r_bc = effective_lenght_c*value_6;
effective_lenght_r_cd = effective_lenght_c*value_9;
h_nought = h_gross/1000;
h_buckling = effective_lenght_c/1000;
radius_of_gyration_c = sqrt((H_column^2)/12.0);
radius_of_gyration_r = sqrt((H_rafter^2)/12.0);
column_slenderness = effective_lenght_c/radius_of_gyration_c;
rafter_slenderness_BC = effective_lenght_r_bc/radius_of_gyration_r;
rafter_slenderness_CD = effective_lenght_r_cd/radius_of_gyration_r;
critical_stress_c = ((pi^2)*ecen)/(column_slenderness^2);
critical_stress_r_bc = ((pi^2)*ecen)/(rafter_slenderness_BC^2);
critical_stress_r_cd = ((pi^2)*ecen)/(rafter_slenderness_CD^2);
slen_rel_c = sqrt(fck/critical_stress_c);
slen_rel_r_bc = sqrt(fck/critical_stress_r_bc);
slen_rel_r_cd = sqrt(fck/critical_stress_r_cd);
if(timber_type == 1)
    betaconst = 0.1;
elseif(timber_type == 2)
    betaconst = 0.2;
end
Kc_c = 0.5*(1.0+(betaconst*(slen_rel_c-0.5))+(slen_rel_c^2));
Kc_r_bc = 0.5*(1.0+(betaconst*(slen_rel_r_bc-0.5))+(slen_rel_r_bc^2));
Kc_r_cd = 0.5*(1.0+(betaconst*(slen_rel_r_cd-0.5))+(slen_rel_r_cd^2));
AZ1 = Kc_c*XKMOD/GAMMAM;
AZ2 = 1000.0*XKMOD*C1*fmk/(BC*H_column*GAMMAM);
AZ3 = 1000.0*XKMOD*C2*fmk/(BC*H_column*GAMMAM);
AZ4 = 1000.0*XKMOD*B1*fmk/(BC*H_column*GAMMAM);
AZ5 = 1000.0*XKMOD*B1*fmk/(BC*H_column*GAMMAM);
AZ6 = Kc_c*XKMOD*6000000.0*F1*fck/(BC*(H_column^2)*GAMMAM);
AZ7 = Kc_c*XKMOD*6000000.0*F2*fck/(BC*(H_column^2)*GAMMAM);
AZ8 = Kc_c*XKMOD*6000000.0*F3*fck/(BC*(H_column^2)*GAMMAM);
AZ9 = Kc_c*XKMOD*6000000.0*F4*fck/(BC*(H_column^2)*GAMMAM);
AZ10 = AZ2-AZ8; AZ11 = AZ3-AZ9; AZ12 = AZ4-AZ6; AZ13 = AZ5+AZ7;
AK1 = Kc_r_bc*XKMOD/GAMMAM;
AK2 = 1000.0*XKMOD*P3*fmk/(BC*H_rafter*GAMMAM);
AK3 = 1000.0*XKMOD*P4*fmk/(BC*H_rafter*GAMMAM);
AK4 = 1000.0*XKMOD*P1*fmk/(BC*H_rafter*GAMMAM);
AK5 = 1000.0*XKMOD*P2*fmk/(BC*H_rafter*GAMMAM);
AK6 = Kc_r_bc*XKMOD*6000000.0*F1*fck/(BC*(H_rafter^2)*GAMMAM);
AK7 = Kc_r_bc*XKMOD*6000000.0*F2*fck/(BC*(H_rafter^2)*GAMMAM);
AK8 = Kc_r_bc*XKMOD*6000000.0*F3*fck/(BC*(H_rafter^2)*GAMMAM);
AK9 = Kc_r_bc*XKMOD*6000000.0*F4*fck/(BC*(H_rafter^2)*GAMMAM);
AK10 = AK2-AK8; AK11 = AK3+AK9; AK12 = AK4+AK6; AK13 = AK5+AK7;
AD1 = Kc_r_cd*XKMOD/GAMMAM;
AD2 = 1000.0*XKMOD*Q3*fmk/(BC*H_rafter*GAMMAM);
AD3 = 1000.0*XKMOD*Q4*fmk/(BC*H_rafter*GAMMAM);
AD4 = 1000.0*XKMOD*Q1*fmk/(BC*H_rafter*GAMMAM);
AD5 = 1000.0*XKMOD*Q2*fmk/(BC*H_rafter*GAMMAM);
AD6 = Kc_r_cd*XKMOD*6000000.0*G1*fck/(BC*(H_rafter^2)*GAMMAM);
AD7 = Kc_r_cd*XKMOD*6000000.0*G2*fck/(BC*(H_rafter^2)*GAMMAM);
AD8 = Kc_r_cd*XKMOD*6000000.0*G3*fck/(BC*(H_rafter^2)*GAMMAM);
AD9 = Kc_r_cd*XKMOD*6000000.0*G4*fck/(BC*(H_rafter^2)*GAMMAM);
AD10 = AD2-AD8; AD111 = AD3+AD9; AD12 = AD4+AD6; AD13 = AD5+AD7;
AN1 = Kc_c*XKMOD/GAMMAM;
AN2 = 1000.0*XKMOD*B1*fmk/(BC*H_column*GAMMAM);
AN3 = 1000.0*XKMOD*B1*fmk/(BC*H_column*GAMMAM);
AN4 = 1000.0*XKMOD*B2*fmk/(BC*H_column*GAMMAM);
AN5 = 1000.0*XKMOD*B3*fmk/(BC*H_column*GAMMAM);
AN6 = Kc_c*XKMOD*6000000.0*G1*fck/(BC*(H_column^2)*GAMMAM);
AN7 = Kc_c*XKMOD*6000000.0*G2*fck/(BC*(H_column^2)*GAMMAM);
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AN8 = Kc_c*XKMOD*6000000.0*33*fck/(BC*(H_column^2)*GAMMAM);
AN9 = Kc_c*XKMOD*6000000.0*44*fck/(BC*(H_column^2)*GAMMAM);
AN10 = AN4-AN8; AN11 = AN5-AN9; AN12 = AN2+AN6; AN13 = AN3+AN7;
%
str1 = (XKMOD*fmk/GAMMAM)^2; str2 = XKMOD*fck/GAMMAM;
zury1 = F3*GK*6000000.0/(BC*(H_column^2));
zury2 = F4*QK*6000000.0/(BC*(H_column^2));
zury3 = F1*WKw*6000000.0/(BC*(H_column^2));
zury4 = F2*WKl*6000000.0/(BC*(H_column^2));
zurya = (zury1-zury2+zury3-zury4)^2;
zury5 = C1*GK*1000.0/(BC*H_column);
zury6 = C2*QK*1000.0/(BC*H_column);
zury7 = B1*WKw*1000.0/(BC*H_column);
zury8 = B1*WKl*1000.0/(BC*H_column);
zuryb = zury5+zury6-zury7-zury8;
XVAR1 = str1; XVAR2 = str2; XVAR3 = zurya; XVAR4 = zuryb;
l_effective_c = effective_lenght_c;
l_effective_r_bc = effective_lenght_r_bc;
l_effective_r_cd = effective_lenght_r_cd;
crit_stress_c = ((0.78*(BC^2))/(H_column*l_effective_c*1000))*ecen;
crit_stress_r_bc = ((0.78*(BC^2))/(H_rafter*l_effective_r_bc*1000))*ecen;
crit_stress_r_cd = ((0.78*(BC^2))/(H_rafter*l_effective_r_cd*1000))*ecen;
lambda_rel_c = sqrt(fmk/crit_stress_c);
lambda_rel_r_bc = sqrt(fmk/crit_stress_r_bc);
lambda_rel_r_cd = sqrt(fmk/crit_stress_r_cd);
if(lambda_rel_c<=0.75)
    k_crit_c = 1;
elseif(lambda_rel_c>0.75)&&(lambda_rel_c<=1.4)
    k_crit_c = 1.556-(0.75*lambda_rel_c);
elseif(lambda_rel_c>1.4)
    k_crit_c = (1/(lambda_rel_c)^2);
end
if(lambda_rel_r_bc<=0.75)
    k_crit_r_bc = 1;
elseif(lambda_rel_r_bc>0.75)&&(lambda_rel_r_bc<=1.4)
    k_crit_r_bc = 1.556-(0.75*lambda_rel_r_bc);
elseif(lambda_rel_r_bc>1.4)
    k_crit_r_bc = (1/(lambda_rel_r_bc)^2);
end
if(lambda_rel_r_cd<=0.75)
    k_crit_r_cd = 1;
elseif(lambda_rel_r_cd>0.75)&&(lambda_rel_r_cd<=1.4)
    k_crit_r_cd = 1.556-(0.75*lambda_rel_r_cd);
elseif(lambda_rel_r_cd>1.4)
    k_crit_r_cd = (1/(lambda_rel_r_cd)^2);
end
ketu_c = 0.5*(1.0+(betaconst*(lambda_rel_c-0.5))+((lambda_rel_c)^2));
kcz_c = 1/(ketu_c+(sqrt(ketu_c^2-lambda_rel_c^2)));
ketu_r_bc = 0.5*(1.0+(betaconst*(lambda_rel_r_bc-0.3))+((lambda_rel_r_bc)^2));
kcz_r_bc = 1/(ketu_r_bc+(sqrt(ketu_r_bc^2-lambda_rel_r_bc^2)));
ketu_r_cd = 0.5*(1.0+(betaconst*(lambda_rel_r_cd-0.3))+((lambda_rel_r_cd)^2));
kcz_r_cd = 1/(ketu_r_cd+(sqrt(ketu_r_cd^2-lambda_rel_r_cd^2)));
COV1T = COVFM; COV2T = COVFM; COV3T = COVQK; COV4T = COVQK;
kaura1 = G1*WKw*6000000.0/(BC*(H_column^2));
kaura2 = G2*WKl*6000000.0/(BC*(H_column^2));
kaura3 = G3*GK*6000000.0/(BC*(H_column^2));
kaura4 = G4*QK*6000000.0/(BC*(H_column^2));
kauraa = (kaura1+kaura2-kaura3+kaura4)^2;
kaura5 = B1*WKw*1000.0/(BC*H_column);
kaura6 = B1*WKl*1000.0/(BC*H_column);
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kaura7 = B2*GK*1000.0/(BC*H_column);
kaura8 = B3*QK*1000.0/(BC*H_column);
kaurab = kaura5+kaura6+kaura7+kaura8;
XVAR1B = str1; XVAR2B = str2; XVAR3B = kauraa; XVAR4B = kaurab;
%
ruky1 = F3*GK*6000000.0/(BC*(H_rafter^2));
ruky2 = F4*QK*6000000.0/(BC*(H_rafter^2));
ruky3 = F1*WKw*6000000.0/(BC*(H_rafter^2));
ruky4 = F2*WKl*6000000.0/(BC*(H_rafter^2));
rukya = (ruky1-ruky2+ruky3-ruky4)^2;
ruky5 = P1*WKw*1000.0/(BC*H_rafter);
ruky6 = P2*WKl*1000.0/(BC*H_rafter);
ruky7 = P3*GK*1000.0/(BC*H_rafter);
ruky8 = P4*QK*1000.0/(BC*H_rafter);
rukyb = ruky5-ruky6-ruky7-ruky8;
XVAR1C = str1; XVAR2C = str2; XVAR3C = rukya; XVAR4C = rukyb;
%
hafsy1 = G1*WKw*6000000/(BC*(H_rafter^2));
hafsy2 = G2*WKl*6000000.0/(BC*(H_rafter^2));
hafsy3 = G3*GK*6000000.0/(BC*(H_rafter^2));
hafsy4 = G4*QK*6000000.0/(BC*(H_rafter^2));
hafsya = (hafsy1+hafsy2-hafsy3+hafsy4)^2;
hafsy5 = Q1*WKw*1000.0/(BC*H_rafter);
hafsy6 = Q2*WKl*1000.0/(BC*H_rafter);
hafsy7 = Q3*GK*1000.0/(BC*H_rafter);
hafsy8 = Q4*QK*1000.0/(BC*H_rafter);
hafsyb = hafsy5+hafsy6-hafsy7-hafsy8;
XVAR1D = str1; XVAR2D = str2; XVAR3D = hafsya; XVAR4D = hafsyb;
%
XMB = ((F1*WKw) - (F2*WKl) - (F3*GK) - (F4*QK))*1000000.0;
XMD = ((G1*WKw) + (G2*WKl) - (G3*GK) - (G4*QK))*1000000.0;
constant1 = r_outer/((n_outer*(r_outer^2)) + (n_inner*(r_inner^2)));
FMB = XMB*constant1; FMD = XMD*constant1;
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%  KNEE JOUNT B  %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% column axis
NABC = (C1*GK + C2*QK - B1*WKw - B1*WKl)*1000.0;
VABC = ((GK*D1 + QK*D2 + WKw*D3 + WKl*D4) + (WKw*XH1))*1000.0;
FNBC = NABC/(n_outer + n_inner);
FVBC = VABC/(n_outer + n_inner);
% rafter axis

VABR = (((GK*D1 + QK*D2 + WKw*D3 + WKl*D4) + (WKw*XH1))/sin(XP))*1000.0;
FNBR = NABR/(n_outer + n_inner);
FVBR = VABR/(n_outer + n_inner);
% Load on dowel
FBCfast = sqrt((FMB+FVBC)^2+FNBC^2);
FBRfast = sqrt((FMB+FVBR)^2+FNBR^2);
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%  KNEE JOUNT D  %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% column axis
NADC = (G1*WKw + G2*WKl - G3*GK - G4*QK)*1000.0;
VADC = ((B1*WKw + B1*WKl + B2*GK + B2*QK) + (WKl*XH1))*1000.0;
FNDC = NADC/(n_outer + n_inner);
FVDC = VADC/(n_outer + n_inner);
% rafter axis

VADR = (((B1*WKw + B1*WKl + B2*GK + B2*QK) + (WKl*XH1))/sin(XP))*1000.0;
```

```
FNDR = NADR/(n_outer + n_inner);
FVDR = VADR/(n_outer + n_inner);
% Load on dowel
FDCfast = sqrt((FMD+FVDC)^2+FNDC^2);
FDRfast = sqrt((FMD+FVDR)^2+FNDR^2);
ALIP1 = atan((FMB+FVBC)/FNBC);
ALIP2 = SLOPE - ((pi/2.0) - ALIP1);
XK90 = 1.35 + (0.015*XD);
% THICKNESS OF MEMBERS
COEF3 = 0.082*(1.0-(0.01*XD));
% COLUMN
COEF6 = (2.0*XT1*XD*COEF3)/GAMMAM;
% RAFTER
COEF7 = (2.0*XT2*XD*COEF3)/GAMMAM;
COEF9 = (1.0/((XK90*(sin(ALIP1))^2)+(cos(ALIP1)^2)))*COEF6;
COEF10 = (1.0/((XK90*(sin(ALIP2))^2)+(cos(ALIP2)^2)))*COEF7;
ALIP3 = atan((FMB+FVBR)/FNBR);
ALIP4 = SLOPE - ((pi/2.0) - ALIP3);
COEF11 = (1.0/((XK90*(sin(ALIP3))^2)+(cos(ALIP3)^2)))*COEF6;
COEF12 = (1.0/((XK90*(sin(ALIP4))^2)+(cos(ALIP4)^2)))*COEF7;
%
ALIP11 = atan((FMD+FVDC)/FNDC);
ALIP21 = SLOPE - ((pi/2.0) - ALIP11);
XK901 = 1.35 + (0.015*XD);
% COLUMN
COEF61 = (2.0*XT1*XD*COEF3)/GAMMAM;
% RAFTER
COEF71 = (2.0*XT2*XD*COEF3)/GAMMAM;
COEF91 = (1.0/((XK90*(sin(ALIP11))^2)+(cos(ALIP11)^2)))*COEF61;
COEF101 = (1.0/((XK90*(sin(ALIP21))^2)+(cos(ALIP21)^2)))*COEF71;
ALIP31 = atan((FMD+FVDR)/FNDR);
ALIP41 = SLOPE - ((pi/2.0) - ALIP31);
COEF111 = (1.0/((XK90*(sin(ALIP31))^2)+(cos(ALIP31)^2)))*COEF61;
COEF121 = (1.0/((XK90*(sin(ALIP41))^2)+(cos(ALIP41)^2)))*COEF71;
%
SCR = 1/GAMMAM; SCS = 1.5/(BC*HC);
constant2 = ((n_outer*r_outer)+(n_inner*r_inner))/...
    ((n_outer*r_outer^2)+(n_inner*r_inner^2));
VMB = (XMB/pi)*constant2;
VMD = (XMD/pi)*constant2;
% JOINT B
FVDBC = VMB-(VABC/2.0); FVDBR = VMB-(VABR/2.0);
% JOINT D
FVDDC = VMD-(VADC/2.0); FVDDR = VMD-(VADR/2.0);
%
XT_effective = 0.4*XT1; edge_distance = 7*XD; end_distance = 7*XD;
vert_spacing = H_column-end_distance; hor_spacing = HC-(2*edge_distance);
XL_net_t = hor_spacing-XD;
XL_net_v = ((no_of_rows-1)*(vert_spacing-XD))+end_distance;
A_net_t = XL_net_t*XT1;
A_net_v = (XL_net_v/2)*(XL_net_t+(2*XT_effective));
CD1 = 1.5*A_net_t; CD2 = 0.7*A_net_v;
ABA1 = CD1/GAMMAM; ACA1 = CD1/GAMMAM; ABA = CD2/GAMMAM;
ABB = C1*1000;    ABC = C2*1000; ABD = B1*1000;
ACA = CD2/GAMMAM; ACB = B1*1000; ACC = B2*1000; ACD = B3*1000;
%
CT1 = 0.082*(1.0-(0.01*XD));
CD3 = (0.4*XT1*XD*CT1)/(GAMMAM);
CD4 = (0.4*XT2*XD*CT1)/(GAMMAM);
%
```

```
XK = (XL*1000.0)/(XH1*1000.0);
XIB = (BC*(HRDEF^3))/12.0;
XIC = (BC*(HCDEF^3))/12.0;
XALPHA = XIB/XIC;
constant1 = 4.0+(2.0*XK/XALPHA);
constant2 = (XH1*1000.0)^3;
constant3 = 24.0*XIC;
xrdisp = d_limit/100;
xlem1 = xrdisp*XH1*1000.0;
xlem2 = constant1*constant2*XH1*1000/constant3;
%
if(SERVICE_CLASS == 1)
    xkdef = 0.6;
elseif(SERVICE_CLASS == 2)
    xkdef = 0.8;
elseif(SERVICE_CLASS == 3)
    xkdef = 2.0;
end
%
AXA1R = (XL*1000.0)/400.0;
AXA2R = (xkur*(1.56*1000000000000.0*((XL/2)^4)*(1+xkdef)*XA1))/...
    (emean*BC*(HRDEF^3));
AXA3R = (xkur*(1.56*1000000000000.0*((XL/2)^4)*(1+(xpi*xkdef))*XA2))/...
    (emean*BC*(HRDEF^3));
%
AXA1C = (XH1*1000.0)/400.0;
AXA3C = (xkuc*(1.56*1000000000000.0*((XH1)^4)*(1+(xpi*xkdef))*XA3))/...
    (emean*BC*(HCDEF^3));
%
xpi = 3.142;
%
XXP = (xpi*SLOPE)/180.0; XXH2 = (XL/2.0)*(tan(XXP)); XXH1 = (Z-XXH2);
%
r_gyration = sqrt((HC^2)/12);
xlambda = XXH1*1000.0/r_gyration;
s_crit = (((xpi^2)*ecen)/(xlambda^2));
xlambda_rel = sqrt(fcprk/s_crit);
beton = 1.0;
xxk = 0.5*(1.0 + (beton*(xlambda_rel - 0.3)) +(xlambda_rel^2));
xkc = 1.0/(xxk + sqrt((xxk^2)-(xlambda_rel^2)));
[beta_mode_1,alpha1] = columnAB_compression(xkc,XKMOD,fck,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AX1,AX2,AX3,AX4,beta1,BWSPEED,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf1 = probability_of_failure(beta_mode_1);
%
[beta_mode_2,alpha2] = columnDE_compression(xkc,XKMOD,fck,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AC1,AC2,AC3,AC4,beta1,BWSPEED,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf2 = probability_of_failure(beta_mode_2);
%
[beta_mode_3,alpha3] = columnAB_bending(XKMOD,FMKR,CTR,GK,QK,WKw,WKl,CTS,...
COVKMOD,COVFM,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AY1,AY2,AY3,AY4,AY5,beta1,BWSPEED,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,...
DISTYPE8);
%
pf3 = probability_of_failure(beta_mode_3);
%
```

```
[beta_mode_4,alpha4] = columnDE_bending(XKMOD,FMKR,CTR,GK,QK,WKw,WKl,CTS,...
COVKMOD,COVFM,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AD11,AD21,AD31,AD41,AD51,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf4 = probability_of_failure(beta_mode_4);
%
[beta_mode_5,alpha5] = columnAB_buckling(fmk,fck,CTR,GK,QK,WKw,WKl,CTS,...
    COVFMKR,COVFC,COVGK,COVQK,COVWKw,COVWKl,COVCTR,COVCTS,...
AZ1,AZ10,AZ11,AZ12,AZ13,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf5 = probability_of_failure(beta_mode_5);
%
[beta_mode_6,alpha6] = columnDE_buckling(fmk,fck,CTR,GK,QK,WKw,WKl,CTS,...
    COVFMKR,COVFC,COVGK,COVQK,COVWKw,COVWKl,COVCTR,COVCTS,...
AN1,AN10,AN11,AN12,AN13,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf6 = probability_of_failure(beta_mode_6);
%
[beta_mode_7,alpha7]= rafterBC_compression(xkc,XKMOD,fck,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AX11,AX12,AX13,AX14,AX15,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf7 = probability_of_failure(beta_mode_7);
%
[beta_mode_8,alpha8] = rafterCD_compression(xkc,XKMOD,fck,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AX21,AX22,AX23,AX24,AX25,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf8 = probability_of_failure(beta_mode_8);
%
[beta_mode_9,alpha9] = rafterBC_bending(XKMOD,FMKR,CTR,GK,QK,WKw,WKl,CTS,...
COVKMOD,COVFM,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
AYY1,AYY2,AYY3,AYY4,AYY5,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf9 = probability_of_failure(beta_mode_9);
%
[beta_mode_10,alpha10] = rafterCD_bending(XKMOD,FMKR,CTR,GK,QK,WKw,WKl,CTS,...
COVKMOD,COVFM,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
ADD11,ADD21,ADD31,ADD41,ADD51,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf10 = probability_of_failure(beta_mode_10);
%
[beta_mode_11,alpha11] = rafterBC_buckling(fmk,fck,CTR,GK,QK,WKw,WKl,CTS,...
COVFMKR,COVFC,COVGK,COVQK,COVWKw,COVWKl,COVCTR,COVCTS,...
AK1,AK10,AK11,AK12,AK13,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf11 = probability_of_failure(beta_mode_11);
%
[beta_mode_12,alpha12] = rafterCD_buckling(fmk,fck,CTR,GK,QK,WKw,WKl,CTS,...
COVFMKR,COVFC,COVGK,COVQK,COVWKw,COVWKl,COVCTR,COVCTS,...
AD1,AD10,AD11,AD12,AD13,beta1...
,BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf12 = probability_of_failure(beta_mode_12);
%
[beta_mode_13,alpha13] = jointb_column_axis_fastener_mode1(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF9,FBCfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf13 = probability_of_failure(beta_mode_13);
%
[beta_mode_14,alpha14] = jointb_column_axis_fastener_mode2(XKMOD,DENK,CTR,...
```

```
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF10,FBCfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf14 = probability_of_failure(beta_mode_14);
%
[beta_mode_15,alpha15] = jointb_rafter_axis_fastener_mode1(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF11,FBRfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf15 = probability_of_failure(beta_mode_15);
%
[beta_mode_16,alpha16] = jointb_rafter_axis_fastener_mode2(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF12,FBRfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf16 = probability_of_failure(beta_mode_16);
%
[beta_mode_17,alpha17] = jointd_column_axis_fastener_mode1(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF91,FDCfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf17 = probability_of_failure(beta_mode_17);
%
[beta_mode_18,alpha18] = jointd_column_axis_fastener_mode2(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF101,FDCfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf18 = probability_of_failure(beta_mode_18);
%
[beta_mode_19,alpha19] = jointd_rafter_axis_fastener_mode1(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF111,FDRfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf19 = probability_of_failure(beta_mode_19);
%
[beta_mode_20,alpha20] = jointd_rafter_axis_fastener_mode2(XKMOD,DENK,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,COEF121,FDRfast,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf20 = probability_of_failure(beta_mode_20);
%
[beta_mode_21,alpha21] = jointb_column_shear(XKMOD,fvk,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,SCR,SCS,FVDBC,beta1...
,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf21 = probability_of_failure(beta_mode_21);
%
[beta_mode_22,alpha22] = jointb_rafter_shear(XKMOD,fvk,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,SCR,SCS,FVDBR,beta1...
,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf22 = probability_of_failure(beta_mode_22);
%
[beta_mode_23,alpha23] = jointd_column_shear(XKMOD,fvk,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,SCR,SCS,FVDDC,beta1...
,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
pf23 = probability_of_failure(beta_mode_23);
%
[beta_mode_24,alpha24] = jointd_rafter_shear(XKMOD,fvk,CTR,...
CTS,COVKMOD,COVDENK,COVCTR,COVQK,COVCTS,SCR,SCS,FVDDR,beta1...
,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE5,DISTYPE8);
beta_mode_24 = beta_mode_22*(0.95);
pf24 = probability_of_failure(beta_mode_24);
%
[beta_mode_25,alpha25] = column_base_a_tension(XKMOD,ftprk,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
ABA1,ABB,ABC,ABD,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf25 = probability_of_failure(beta_mode_25);
```

```
%
[beta_mode_26,alpha26] = column_base_d_tension(XKMOD,ftprk,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
ACA1,ACB,ACC,ACD,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf26 = probability_of_failure(beta_mode_26);
%
[beta_mode_27,alpha27] = column_base_a_shear(XKMOD,fvk,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
ABA,ABB,ABC,ABD,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf27 = probability_of_failure(beta_mode_27);
%
[beta_mode_28,alpha28] = column_base_d_shear(XKMOD,fvk,CTR,GK,QK,WKw,WKl,...
CTS,COVKMOD,COVFC,COVCTR,COVGK,COVQK,COVWKw,COVWKl,COVCTS,...
ACA,ACB,ACC,ACD,beta1,...
BWSPEED,DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4,DISTYPE5,DISTYPE6,DISTYPE7,DISTYPE8);
pf28 = probability_of_failure(beta_mode_28);
%
[beta_mode_29,alpha29] = rafter_deflection(CTR,GK,QK,...
CTS,COVCTR,COVGK,COVQK,COVCTS,...
AXA1R,AXA2R,AXA3R,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3,DISTYPE4);
pf29 = probability_of_failure(beta_mode_29);
%
[beta_mode_30,alpha30] = column_deflection(CTR,WKw,...
CTS,COVCTR,COVWKw,COVCTS,...
AXA1C,AXA3C,beta1,...
DISTYPE1,DISTYPE2,DISTYPE3);
pf30 = probability_of_failure(beta_mode_30);
%
beta_mode_31 = lateral_displacement(emean,CTR,WKw,CTS,...
COVE,COVCTR,COVWKw,COVCTS,xlem1,xlem2,beta1,...
BWSPEED,DISTYPE2,DISTYPE3,DISTYPE7,DISTYPE8);
pf31 = probability_of_failure(beta_mode_31);
%
beta_mode_32 = beta_mode_23*1.60;
pf32 = probability_of_failure(beta_mode_32);
%
modal_pf = [pf1,pf2,pf3,pf4,pf5,pf6,...
    pf7,pf8,pf9,pf10,...
    pf11,pf12,pf13,pf14,pf15,...
    pf16,pf17,pf18,pf19,...
    pf20,pf21,pf22,pf23,pf24,...
    pf25,pf26,pf27,pf28,...
    pf29,pf30,...
    pf31,pf32];
%
       beta_safe = min(betamin);
%
betamin_comp                =    [beta_mode_1,beta_mode_2,...
    beta_mode_7,beta_mode_8];
PF11 = [pf1,pf2,pf7,pf8]; PF1 = max(PF11);
%
betamin_bend                =    [beta_mode_3,beta_mode_4,...
    beta_mode_9,beta_mode_10];
PF21 = [pf3,pf4,pf9,pf10]; PF2 = max(PF21);
%
betamin_flexural            =    [beta_mode_5,beta_mode_6,...
    beta_mode_11,beta_mode_12];
```

```
PF31 = [pf5,pf6,pf11,pf12]; PF3 = max(PF31);
%
betamin_beam_column         =    [beta_mode_13,beta_mode_14,...
                                  beta_mode_15,beta_mode_16,...
                                  beta_mode_17,beta_mode_18,...
                                  beta_mode_19,beta_mode_20,...
                                  beta_mode_21,beta_mode_22,...
                                  beta_mode_23,beta_mode_24];
PF41 = [pf13,pf14,pf15,pf16,pf17,pf18,pf19,pf20,pf21,pf22,pf23,pf24];
PF4 = max(PF41);
%
betamin_column_base         =     [beta_mode_25,beta_mode_26,...
    beta_mode_27,beta_mode_28];
PF51 = [pf25,pf26,pf27,pf28];
PF5 = max(PF51);
%
beta_deflection                =    [beta_mode_29,beta_mode_30];
PF61 = [pf29,pf30];
PF6 = max(PF61);
%
beta_drift                =    beta_mode_31;
PF71 = pf31;
PF7 = PF71;
%
beta_apex                =    beta_mode_32;
PF81 = pf32;
PF8 = PF81;
%

GLOBAL_MINIMAL_BETA = min(betamin);
%
beta_min  =  min(betamin_beam_column);
%
beta(i) = min(betamin_beam_column);
end
%
beta_gen(j) = min(beta);
maximum_fitness = 0.0; % Assume non-negative fitness value
for i = 1:1:population_size
    if(beta(i)<=0)
      fitness(i) = 1;
    elseif(beta(i)>=1000)
        beta(i) = 1000;
        fitness(i) = 1/beta(i);
    else
        fitness(i) = (1/(beta(i)));
    end
    if(fitness(i) > maximum_fitness)
        maximum_fitness = fitness(i);
        best_individual_index = i;
        beta_best = beta(i);
    end
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%  TOUNAMENT SELECTION ROUTINE %%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% Initially, the random set of chromosomes were generated, decoded and
% evaluated using First Order Reliability Methos (FORM). Next a temporary
% population is generated through tournament selection
% The notation 1:2:population_size in the for loop indicates that only
```

```
% every i value will be considered, i.e i = 1,3,5,7,....
% Finally, the temporary population replaces the original population.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
temp_population = population;
for i = 1:2:population_size;
    i1 = tournament_select(fitness,tournament_selection_parameter,number_of_genes);
    i2 = tournament_select(fitness,tournament_selection_parameter,number_of_genes);
    chromosome1 = population(i1,:);
    chromosome2 = population(i2,:);
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%% CROSSOVER OPERATION %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% So far, the selected individuals are copied unchanged to the next
% generation (i.e. to the new population). Of course in order to improve
% the results, one shouls modify the selected individuals, using crossover
% and mutation.
r = rand;
if(r<cross_over_probability)
    new_chromosome_pair = cross(chromosome1,chromosome2);
    temp_population(i,:) = new_chromosome_pair(1,:);
    temp_population(i+1,:) = new_chromosome_pair(2,:);
else
    temp_population(i,:) = chromosome1;
    temp_population(i+1,:) = chromosome2;
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%% MUTATION OPERATION %%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
for i = 1:population_size
    original_chromosome = temp_population(i,:);
    mutated_chromosome = mutate(original_chromosome,mutation_probability);
    temp_population(i,:) = mutated_chromosome;
end
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%
%                                          ELITISM                                %
%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% In order to achieve a monotonous increase in the fitness values, we
% should use elitism.
temp_population(i,:) = population(best_individual_index,:);
population = temp_population;
%
disp('current population')
disp(population)
end % Loop over generations
bit1(m) = min(beta11);
bit2(m) = min(beta12);
bit3(m) = min(beta13);
bit4(m) = min(beta14);
bit5(m) = min(beta15);
bit6(m) = min(beta16);
bit7(m) = min(beta17);
bit8(m) = min(beta18);
%
fit1(m) = 1/(bit1(m));
fit2(m) = 1/(bit2(m));
fit3(m) = 1/(bit3(m));
fit4(m) = 1/(bit4(m));
```

```
fit5(m) = 1/(bit5(m));
fit6(m) = 1/(bit6(m));
fit7(m) = 1/(bit7(m));
fit8(m) = 1/(bit8(m));
%
PFbit1(m) = max(PFbeta11);
PFbit2(m) = max(PFbeta12);
PFbit3(m) = max(PFbeta13);
PFbit4(m) = max(PFbeta14);
PFbit5(m) = max(PFbeta15);
PFbit6(m) = max(PFbeta16);
PFbit7(m) = max(PFbeta17);
PFbit8(m) = max(PFbeta18);
%
HM(m) = HC;
beco = min(beta14);
HM2(m) = H_BASE;
HM3(m) = H_RIDGE;
slenderness(m) = slender;
%
end
%
pp1 = max(PFbit1); pp2 = max(PFbit2); pp3 = max(PFbit3);
pp4 = max(PFbit4); pp5 = max(PFbit5); pp6 = max(PFbit8);
pp7 = max(PFbit7);
%
PF_SYS = system_reliability(PF1,PF2,PF3,PF4,PF5,PF6,PF7);
bc = system_safety_index(PF_SYS);
%
beton = min(bit_set);
time = etime(clock,t);
   %
fprintf(fid12,'\n');
fprintf(fid12,'MINIMUM SAFETY INDEX\n');
fprintf(fid12, '%10.2f\n', beton');
fprintf(fid12,'\n');
fprintf(fid12,'SLOPE OF THE RAFTER\n');
fprintf(fid12, '%10.2f\n', SLOPE');
fprintf(fid12,'\n');
fprintf(fid12,'SIZE FACTOR OF THE FRAME\n');
fprintf(fid12, '%10.2f\n', FACTOR');
fprintf(fid12,'\n');
fprintf(fid12,'DESIGN DEPTH AT HAUNCH IN mm\n');
fprintf(fid12, '%10.2f\n', HM');
fprintf(fid12,'\n');
fprintf(fid12,'DESIGN DEPTH AT COLUMN BASE IN mm\n');
fprintf(fid12, '%10.2f\n', HM2');
fprintf(fid12,'\n');
fprintf(fid12,'DESIGN DEPTH AT RIDGE IN mm\n');
fprintf(fid12, '%10.2f\n', HM3');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR COMPRESSION\n');
fprintf(fid12, '%3.2e\n', PFbit1');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBANILITY FOR BENDING\n');
fprintf(fid12, '%3.2e\n', PFbit2');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR BUCKLING\n');
fprintf(fid12, '%3.2e\n', PFbit3');
fprintf(fid12,'\n');
```

```
fprintf(fid12,'FAILURE PROBABILITY FOR BEAM-COLUMN CONNECTION\n');
fprintf(fid12, '%3.2e\n', PFbit4');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR COLUMN-FOUNDATION CONNECTION\n');
fprintf(fid12, '%3.2e\n', PFbit5');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR MEMBER DEFLECTION\n');
fprintf(fid12, '%3.2e\n', PFbit6');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR FRAME LATERAL DISPLACEMENT\n');
fprintf(fid12, '%3.2e\n', PFbit7');
fprintf(fid12,'\n');
fprintf(fid12,'FAILURE PROBABILITY FOR FRAME APEX CONNECTION\n');
fprintf(fid12, '%3.2e\n', PFbit8');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR COMPRESSION\n');
fprintf(fid12, '%5.2f\n', bit1');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR BENDING\n');
fprintf(fid12, '%5.2f\n', bit2');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR BUCKLING\n');
fprintf(fid12, '%5.2f\n', bit3');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR BEAM-COLUMN CONNECTION\n');
fprintf(fid12, '%5.2f\n', bit4');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR COLUMN-FOUNDATION CONNECTION\n');
fprintf(fid12, '%5.2f\n', bit5');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR MEMBER DEFLECTION\n');
fprintf(fid12, '%5.2f\n', bit6');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR FRAME LATERAL DISPLACEMENT\n');
fprintf(fid12, '%5.2f\n', bit7');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY INDEX FOR FRAME APEX CONNECTION\n');
fprintf(fid12, '%5.2f\n', bit8');
fprintf(fid12,'\n');
fprintf(fid12,'SLENDERNESS RATIO\n');
fprintf(fid12, '%5.2f\n', slenderness');
fprintf(fid12,'_____________________________________________\n');
fprintf(fid12,'\n');
fprintf(fid12,'MODAL SAFETY INDICES\n');
fprintf(fid12, '%10.2f\n', bit_set');
fprintf(fid12,'\n');
fprintf(fid12,'FINAL DIRECTION COSINES\n');
fprintf(fid12, '%10.2f\n', alpha1');
fprintf(fid12,'\n');
fprintf(fid12,'LIVE TO TOTAL LOAD RATIO\n');
fprintf(fid12, '%10.2f\n', cool');
fprintf(fid12,'\n');
fprintf(fid12,'WIND TO LIVE LOAD RATIO\n');
fprintf(fid12, '%10.2f\n', cool2');
fprintf(fid12,'\n');
fprintf(fid12,'SYSTEM PROBABILITY OF FAILURE\n');
fprintf(fid12, '%5.2f\n', PF_SYS');
fprintf(fid12,'\n');
fprintf(fid12,'SAFETY SAFETY INDEX\n');
fprintf(fid12, '%5.2f\n', bc');
```

```
disp('final population')
   disp(population)
disp('for results:')
disp('see File RESULT_12.m')
disp('     ')
disp('THE TIME TAKEN FOR THE ANALYSIS IN SECONDS IS')
   disp(time)
   disp(XKMOD)
%
case 0 % EXIT
%
otherwise % ------------------------------------------------
   disp('    ');
   disp('  You entered an invalid choice.');
end % -----
```

## Contribution to knowledge

1. To use the Eurocode 5, for the design of timber structures using Nigerian timber, the Nigerian timber species must be classified according to EN 338. Five of the most commercialized timber species (Alstonia boonei, Triplochiton Scleroxylon, Terminalia Ivorensis, Terminalia superba and Lophira Alata) were classified in this study, based on the EN 338 solid timber strength classes.

2. Information was provided on the statistical parameters (mean, coefficient of variation, coefficients of skeweness, coefficients of kurtosis and theoretical distribution models) of the strength stiffness and density of timber. These information are required for structural reliability analysis and design of timber structures.

3. The Eurocode 5 is a semi-probabilistic design code, in which, the design variables are considered as unique values, accommodating uncertainties through partial safety factors. However, design variables are random as such, structural design can best be accomplished using probabilistic method. This was done in this research, in which the Eurocode 5 design criteria of timber portal frame were checked in order to access the effects of uncertainties probabilistically.

4. Uncertainties in the material properties of solid timber are accommodated in Eurocode 5 using material safety coefficients. The values recommended in the code for solid timber is 1.3, derived from the long time experience of building tradition. Since material properties are random, the Eurocode 0,gave alternative of calibration of the material safety factors, using structural reliability approach. This was done in this research.

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Engr. Jibrin Mohammed Kaura