KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY

FACULTY OF CIVIL AND GEOMATIC ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

INVESTIGATIONS INTO LESSER KNOWN TIMBER SPECIES FOR LIGHT BRIDGE CONSTRUCTION

Thesis submitted to the Department of Civil Engineering, College of Engineering, Kwame Nkrumah University of Science and Technology, in partial fulfilment of the requirements for the award of Master of Philosophy Degree in Civil Engineering.

BY

EMMANUEL APPIAH-KUBI
(BSc. Civil Engineering)

June 2009

DECLARATION

Candidate's Declaration

I hereby declare that this thesis is the result of my original work and that no part of it has been presented for another degree in this University or elsewhere.

Candidate's Signature .... Date. 15-01-2010

Name: Emmanuel Appiah-Kubi

Supervisors' Declaration

We hereby declare that the preparation and presentation of the dissertation were supervised in accordance with the guidelines on supervision of dissertation laid down by the Kwame Nkrumah University of Science and Technology.

Supervisor's Name	Signature	Date
Prof. C. K. Kankam	....	15th Jan 2010
Dr. M. Adom Asamoah	....	15/01/2010
Prof. K. Frimpong-Mensah	....	15/01/2010

ABSTRACT

The mechanical properties of ten lesser known timber species were investigated. Four out of the ten species with the best results in terms of their mechanical properties were selected and further tested in bending, compression and tension. The species with the overall best properties was used to design and constructed a prototype pedestrian bridge on KNUST campus to demonstrate its suitability in construction.

The results of the study showed that two out of the 10 species; Sterculia rhinopetala (Wawabima) and Blighia sapida (Akye) had outstanding properties which made suitable to be used in construction. Wawabima was used in the design of the main structural frame and Akye was used as cladding members in the design because of its relatively higher natural durability.

The local wood technology in the region of Kumasi was assessed to determine the preparedness of the local wood industry in working with the species. The Sawmills visited proved their capacity to saw and mill logs of the said species. Carpentries reviewed, though still using simple hand tools, are familiar with the said species and ready and have enough capacity to work with them.

A pedestrian timber bridge has been constructed with the two species with outstanding mechanical properties, demonstrating their suitability in construction. The bridge is designed with a characteristic live load of 4 kN/m^2. The bridge is therefore designed to accommodate 255 people each weighing about 70 kg at a time. However, the bridge can

accommodate 340 people each weighing 70 kg at a time (when loaded with a characteristic live load of 5.5 kN/m^2) beyond which the bridge will fail. The 340 people weighing about 70 kg is therefore the loading limit which can cause the bridge to fail.

ACKNOWLEDGEMENT

I wish to express my sincere gratitude to my Supervisors; Prof. C.K. Kankam, Prof. K. Frimpong-Mensah and Dr. M. Adom Asamoah for giving me the necessary guidance and spending time to read through every bit of the manuscript. I wish also to specially thank Dr. Mark Adom Asamoah for standing in as the supervisor while the main supervisor was on leave. But for his dedication and desire to ensure that I complete this work, it would surely not have been possible.

My thanks also go to the various Directors and Heads of all the institutions and establishments I visited during the study for their assistance in the collection of questionnaire and data. I wish also to thank all the Laboratory Technicians and Assistants at the Civil Engineering Department and the Wood Science and Technology Department (FRNR) for their immense support during the Laboratory tests.

I wish to also thank Prof. Maurice Brunner of Bern University of Applied Sciences, Biel, Switzerland and the sponsors of the Ghana Timber Bridges Project, Swiss National Science Foundation (SNSF), for the financial support and exposure given me through the project activities. The entire Project team of the Ghana Timber Bridges Project and my colleagues are greatly appreciated for their contributions towards this work. Last, but not least, I wish to register my profound thanks to my dear and lovely wife, Eunice, for her affection and encouragement throughout the period of work.

TABLE OF CONTENTS

LIST OF SYMBOLS

A	cross sectional area, in square millimetres
a	distance between a loading position and the nearest support in a bending test, in millimetres
b	width of cross section in a bending test, or the smaller dimension of the cross section, in millimetres
$E_{m,g}$	global modulus of elasticity in bending, in Newton per square millimetre
$E_{m,l}$	local modulus of elasticity in bending, in Newton per square millimetre
$E_{m,app}$	apparent modulus of elasticity in bending, in Newton per square millimetre
F	load, in Newtons
$F_{c,90}$	compressive load perpendicular to the grain, in Newtons
F_{max}	maximum load, in Newtons
$f_{c,0}$	compressive strength parallel to the grain, in Newton per square millimetre
f_m	bending strength, in Newton per square millimetre
$f_{t,0}$	tensile strength parallel to the grain, in Newton per square millimetre
G	shear modulus, in Newton per square millimetre
h	depth of cross section in a bending test, or the larger dimension of the cross section, in millimetres
I	second moment of area, in millimetre to the fourth power
K, k	coefficients

k_G	coefficient for shear modulus
ℓ	effective span of test beam in bending, in millimetres
l_1	gauge length for the determination of modulus of elasticity, in millimetres
w	deformation, in millimetres

LIST OF TABLES

CHAPTER FIVE

LIST OF FIGURES

CHAPTER FOUR

CHAPTER FIVE

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND TO THE STUDY

In Africa, south of the Sahara there are many rivers which on the one hand make the land quite fertile and on the other hand may be a traffic barrier for school children and the local transport of goods. Many of these barriers could be overcome with small bridges of only 5-10m span, which in Europe are often built in timber. In Ghana many of these short span bridges are usually built with concrete and reinforced concrete or steel. The bridges are usually constructed in reinforced concrete. Since clinker for making cement is imported from Europe, and the steel reinforcement is very highly priced, bridge building in Ghana, is an expensive undertaking. This has limited the number of bridges that can be put up across many of the rivers and streams in our communities serving as traffic barriers. With the availability of several species of timber in our forests, it should be possible to attain more economical solutions for the construction of bridges especially for pedestrians and light traffic.

The exploitation of timber in Ghana and indeed in many parts of Africa is limited to a few of the over 300 known species. Some of the popular species are *Pterygota macrocarpa* (Koto), *Milicia excelsa* (Odum), *Khaya ivorensis* (Mahogany), *Triplochiton scleroxylon* (Wawa), *Terminalia ivorensis* (Emire), *Aningeria altissima* (Asanfina) and *Nesogordonia papaverifera* (Danta). The national and the international demand for these species with excellent (exceptional) properties in terms of their strength and the quality of their finishes, has led to a dangerous overexploitation. Although there are many other timber species, they are hardly ever touched because their material properties have not yet been fully classified. The available data on the mechanical properties of such species has generally been attained with tests on small clear specimens (either 2 x 2 x 30 inches or 1 x 1 x 16 inches as specified by ASTM D 143-52). For structural use the properties of large size specimens, which are generally quite different from those of small clear specimens because of the unavoidable defects such as knots and shakes, have not yet been determined.

In this research work, ten lesser-known timber species have been selected for the study. The species were *Albizia ferruginea* (Awiemfosamina), *Sterculia rhinopetala* (Wawabima), *Blighia sapida* (Akye), *Canarium schweinfurthii* (Bediwonua), *Petersianthus macrocarpus* (Esia), *Sterculia oblonga* (Ohaa), *Cola gigantea* (Watapuo), *Celtis zenkeri* (Esa), *Antiaris toxicaria* (Kyenkyen) and *Amphimas pterocarpoides* (Lati).

Timber is a structurally sound material and compares favourably with concrete, steel, and stone and a variety of other materials in terms of strength per unit mass. Wood in its natural state can be used for many forms of construction (Jayanetti, 1998). As a construction material, wood is strong, light, durable, and flexible and can be easily worked with. In contrast to the substitutes for wood in structural purposes such as brick, metal, concrete and plastics, wood can be produced and transported with little energy consumed and the products are renewable (Koch, 1991).

Within the framework of timber as a construction material, a distinction is made between primary or commercially accepted species and lesser known or less accepted species. For several reasons, the viability of timber in the context of construction is dependent on lesser known timber species rather than commercially accepted species. Freezaillah (1990) defines Lesser-known species (LKS) as a commercially less accepted species left in the forest after a logging operation. But, as stated by Hansom (1983), a better definition is that it is a species that is not being put to best advantage (although many commercial species are not being put to best advantage either). The list of usable species has lengthened to some extent because of advances in technology and promotion and because of a growing scarcity of the more desired species. There has been considerable discussion about the fuller utilization of tropical forests with particular reference to the LKS, but the problem has remained intractable and little has been done (Freezaillah, 1990). Eddowes (1980), in discussing the technical aspects of promoting the LKS in Papua New Guinea, identified inadequate data on physical and mechanical properties; as one of the main problems in promoting the LKS. This problem, however, applies to LKS not only in Papua New Guinea but in all closed tropical forests.

It is important to stress that the term lesser-known timber species does not connote inferiority; many lesser-known timber species have as yet not been characterized and may as well be comparable to the commercial species (Jayanetti, 1998). The definition of LKS is dynamic and the status of a species may change with time.

There is a lack of secure knowledge on the material properties of these lesser-known timber species. This research is therefore undertaken to determine the mechanical properties of the selected lesser known species by testing with full-size structural members.

1.2 OBJECTIVES OF THE STUDY

The objective of the research is to develop and promote light bridges built with lesser known timber species in Ghana. The research activities comprise the following:

- To establish the mechanical and physical properties of a selected number of lesser known timber species necessary for the design of timber structures. Mechanical properties including bending strength, modulus of elasticity, tensile strength, compression strength and shear modulus will be determined. Physical properties including density and moisture content (MC) will be determined.
- To determine the local wood processing technology with regard to the standards of Sawmills and carpentries in Kumasi and the technological improvements that are necessary to undertake bridge construction.
- To design, and construct prototype light timber bridge systems and connections appropriate to the local conditions with regard to sawmill production and workmanship of artisans.

1.3 JUSTIFICATION OF STUDY

Advances in timber engineering has led to timber being accepted as a structural material that can compete with the conventional materials such as steel and concrete for construction. The rising cost of imported basic building materials such as clinker for the manufacture of cement coupled with the increasing demand for housing in Ghana have created an inevitable situation where one needs to ~~seriously take~~ another look at developing the use of indigenous materials, especially timber for structural purposes such as the construction of bridges and buildings. Prolonged and widespread experience elsewhere demonstrates that timber can meet both the logistical and engineering requirements of the construction industry.

Timber is one of Ghana's most readily available natural resources. The natural forest resources occupy an area of some 81,306 square kilometers, approximately one-third of the entire country and hold more than 400 indigenous hardwood species (Anon, 1996; Usher and Ocloo, 1979). In spite of the abundance of this natural resource very little of it is used structurally for construction in Ghana due to the unfounded prejudice against its use and lack of technical data on its properties such as strength and durability of most of the species, especially the lesser known ones. Lesser-known species (LKS) are species yet to be exported, but are now being promoted or have the potential to be promoted in the local market. According to Oteng-Amoako (2006), the forest occurrence of LKS is variable, usually from frequent to sparse, and data on their technological properties are limited, for example, *Berlinea grandiflora* (Berlinea). The LKS are mostly lower-risk species which can be exploited under normal forest harvesting practice.

According to Brazier (1978), commercial hardwood harvested for industrial use often represents only three to ten percent of the timber volume in any given area. This assertion is supported by Youngs (1977) that 90% of log trade in Nigeria has been in only six primary species. Likewise, Ofori (1985) reports that only about 80 tree species out of approximately 600 species in West Africa, which reach sizes suitable for lumber and plywood production, are exploited. In an FAO (1988) inventory project classification of Ghana's high forest tree species, only about 60 species were registered as having been exported from the country between 1973 and 1988. Large numbers of wood species growing in the natural tropical forests are excluded from the international timber market because they have been deemed "undesirable" for a number of reasons including the chemical constituents of their extractives and lack of adequate data on their mechanical properties.

Lack of adequate data (mechanical properties) on the lesser utilized species in Ghana has led to the over-exploitation of the few noble commercial species such as Odum, Mahogany, Iroko etc. whose properties are well known and established (Allotey, 1992). Many of the species such as the ten selected for the research are not being exploited commercially because their material properties have not yet been established. Local farmers have therefore taken advantage of using them as charcoal and firewood, which serve as the main source of fuel in the farming communities.

Some of the species have also been protected from exploitation by the Forestry Commission of Ghana to prevent their depletion in the forest. Some of the protected species possess poisonous and strange chemicals in their extracts, which have not yet been researched. The Commission has therefore banned the exploitation of such species since they may pose threat to users and to end the dangerous over-exploitation which has led to the near depletion of such species in the forest. This has led to limited species which are being over-exploited. There is therefore the need to conduct tests on the mechanical properties of the lesser known species so that their properties can be established for usage. The need to research on these lesser known timber species is because they are local resources and are in abundant supply (Allottey, 1991). They are also unexploited compared to the overexploited few noble species.

Prior to 1970, the characteristics of timber were assessed on the basis of the characteristics of small clear pieces of wood. However, following the extraordinary pioneering work by Madsen (Madsen 1992), it was realized that this could be quite misleading. This is because the strength of structural size timber is heavily influenced by the presence of natural features such as knots, pith, etc. Alik and Nakai (1997a) noted in their work that using the results from full size structural timber was considered to be more reliable to allocate design stresses so as to eliminate the risk of stress assumptions. In addition, the values will reflect more on the actual strength of timber in use. So far, there is lack of information on strength properties of full size structural timber hence engineers, architects, designers and builders tend to use other materials such as concrete and steel for building and construction. The proper and effective utilization of timber as a construction material very much depend on the experience and understanding of their technical data and also their structural behaviour with regards to each particular timber species.

Wood may be described as an orthotropic material that is it has unique and independent mechanical properties in the directions of three mutually perpendicular axes: longitudinal, radial and tangential. Mechanical properties most commonly measured and represented as strength properties for design include bending stress parallel to grain, modulus of elasticity in bending, compressive stress perpendicular to grain, and shear strength parallel to grain. Additional measurements are often made to evaluate work to maximum load in bending, impact bending strength, tensile strength perpendicular to grain, and hardness. Strength properties less commonly measured in clear wood include torsion, toughness, rolling shear, and fracture toughness. Other properties involving time under load include creep, creep rupture or duration of load and fatigue (Forest Products General Technical Report, 1999).

1.4 SCOPE OF THESIS

Chapter two of the thesis deals with the review of related literature on the work. Information on existing timber bridges in and around Kumasi were gathered and reviewed. Research trends in the field of timber and wood science were also discussed in the chapter. Topics discussed in the chapter also include the physical structure of wood, stress grading and the various grading systems used for classifying timber, and the structural use of timber. Literature on the selected lesser known species for the study has also been reviewed.

In Chapter Three, laboratory tests conducted on the ten species and results are presented. Analysis of the results was done with the aid of graphs (curves), tables of summary of results and observations obtained from the testing. The chapter also describes the various tests conducted and how the various mechanical properties such as bending stress perpendicular to the grain, compressive stress, tensile stress and shear modulus were determined. Comparison of results of the ten species was made and four species with the overall best results were selected for further test for a more detailed analysis. The species with the overall best structural properties which makes it suitable for construction was classified under the European standard for classification of timber, EN 338.

Chapter four deals with the assessment of the local wood processing technology and the preparedness of the local industry to process and use lumber for the construction of bridges. Existing timber bridges constructed in the region of Kumasi were researched into. Information on the state of the existing bridges and measures to improve their durability have been outlined in the chapter. Questionnaires were sent to ascertain the technologies of existing sawmills and carpentries in Kumasi. Results and analysis of the data gathered from the questionnaire have been presented. The chapter sought to assess the capacity of sawmills to process other secondary species for the local market for their use in the construction of bridges; and the capacity and/or preparedness of local carpentries or carpenters to work with these species in the construction of bridges.

The design and construction of a prototype pedestrian timber bridge on KNUST is presented in Chapter five. The bridge was constructed with two of the species selected for the study, to demonstrate their suitability for construction. The chapter outlines the design criteria, the layout of the bridge, the structural form of the bridge, and the erection of the bridge.

Chapter six presents the conclusions drawn from the study and recommendations made for further research.

CHAPTER TWO

LITERATURE REVIEW

2.0 INTRODUCTION

In this research, an investigation into lesser-known timber species for the construction of light bridges is the topic under study. Therefore an extensive literature on research trends in the field of timber (wood science) and the engineering basis of analysis is paramount. The physical structure of wood, the benefits of timber, stress grading and the various grading systems used, structural use of timber and the evolution of timber design are discussed. In addition, research trend on Ghanaian timber species, timber bridges, and the strength properties of timber have also been reviewed. Literature on the selected species for the study, and the concept of natural durability of wood has been provided.

2.1 PHYSICAL STRUCTURE OF WOOD

Wood is a cellular organic material made up principally of cellulose, which comprises the structural units (cells), and lignin, which cements the structural units together. It also contains hemicelluloses, extractives, and ash-forming minerals. Wood cells are hollow, and they vary from about 0.04 to 0.33ins in length and from 0.0004 to 0.0033in in diameter (Halperin & Bible, 1994). Most cells are elongated and are oriented vertically in the growing tree, but some, called rays, are oriented horizontally and extend from the bark toward the centre or pitch of the tree.

Wood is a versatile raw material and is widely used in construction, especially in countries such as Canada, Sweden, Finland, Norway, Poland etc where there is an abundance of good quality timber. Timber can be used in a range of structural applications including marine works: construction of wharves, piers; heavy civil works: bridges, piles, pylons; domestic housing: roofs, floors, partitions; formwork for pre-cast and in-situ concrete etc.

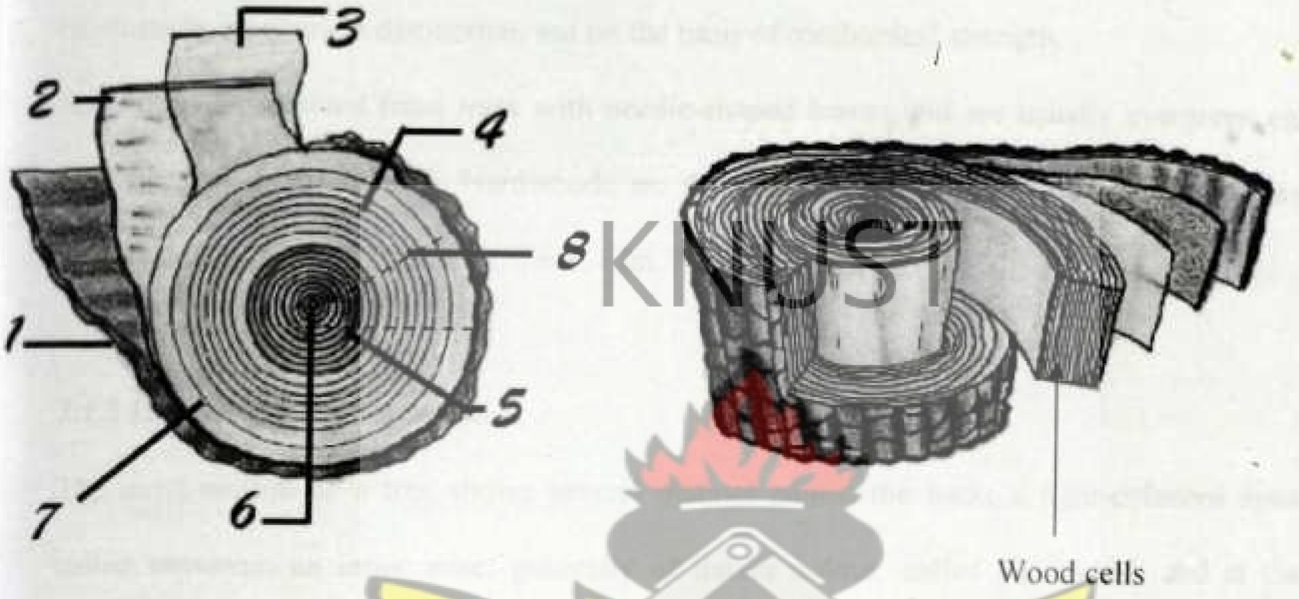


Figure 2.1: Main parts or structure of a wood (Source: Adapted from Oduro & Gotah, 2008)

Key to Figure

1 – Outer bark - protects the tree from extreme temperatures, bad weather, insects and fungi.

2 – Phloem (inner bark) which conveys food to the various parts of the tree

3 – Cambium layer is a thin layer of cells which produce phloem on one side and sapwood on the other.

4 – Sapwood is the living wood in the tree through which the raw sap rises from the roots to the leaves.

5 – Heartwood consists of old cells. This is the dead part of tree that nevertheless provides structural strength

6 – Pith is the central core of the tree. He is often cut out in order to avoid cupping or splitting of boards

7 – Annual rings are the new cells formed every year in the tree, arranged in concentric circles. The annual rings show the growth or the amount of wood produced during one growing season.

8 – Medullary rays also known as pith ray or wood ray, are located from the pith to the bark and these are the radial transmission of sap to the parts of the tree

2.1.1 Hardwood and Softwood

There is an enormous variety of timber species. They are divided into softwoods and hardwoods, a botanical distinction, not on the basis of mechanical strength.

Softwoods are derived from trees with needle-shaped leaves and are usually evergreen eg. Fir, larch, pine, spruce etc. Hardwoods are derived from trees with broad leaves and are usually deciduous eg. Iroko, Oak, Teak, Elm, Mahogany etc.

2.1.2 Heartwood and Sapwood

The cross-section of a tree shows several distinct zones; the bark; a light-coloured zone called sapwood; an inner zone; generally of darker colour, called heartwood; and at the centre, the pith (Figure 2.1). A tree increases in diameter by adding new layers of cells from the pith outward. For a time, this new layer functions as living cells that conduct sap and store food, but eventually, as the tree increases in diameter, cells toward the centre become inactive and serve only as support for the tree. The inactive inner layer is the heartwood and the outer layer containing the living cells is the sapwood. There is no consistent difference between the weight and strength properties of heartwood and sapwood. In some species, heartwood is significantly more resistant to decay fungi than sapwood.

2.1.3 Annual Growth Rings

In climates where temperature limits the growing season of a tree, each annual increment of growth usually is readily distinguishable. Such an increment is known as the annual growth ring which is noticed on the cross-section of a log.

2.1.4 Grain and Texture

Grain and texture are used in many ways to describe the characteristics of wood. Grain often refers to the width of the annual rings, as in "close-grained" or "coarse-grained". Sometimes it indicates whether the fibres are parallel to or at an angle with the sides of the pieces, as in "straight-grained" or "cross-grained". Texture usually refers to the fineness of wood structure rather than to the annual rings.

2.1.5 Growth Characteristics

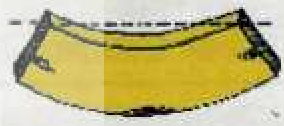
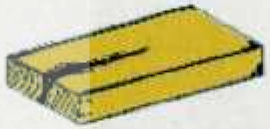
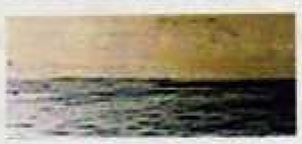
Wood contains certain natural growth characteristics such as knots, slope of grain, compression wood and shakes, which may depending on their size, number, and location in a structural member, adversely affect the strength properties of that member. These growth characteristics are known as defects.

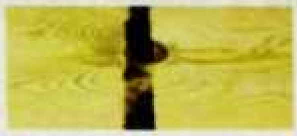
2.1.5.1 Some Common Defects in Wood

Defects may be naturally occurring or can be man-made. Natural defects can be due to many reasons such as environmental factors, growth patterns, soil composition, etc. An example of a natural defect is a knot. Man-made defects can occur at many points from the felling of the tree, transport, storage, sawing, drying, etc. An example of a man-made defect is a twist.

Although you can work around some defects such as knots, or cut off defects such as splits, boards that are heavily twisted, bowed, cupped, or crooked usually are not usable. A piece of wood with a straight grain without knots, holes, or cracks has the highest strength possible for its species. Below is a table of common defects which affect the strength of lumber.

Table 2.1: Some Common Defects in lumber

Bow	A curve along the face of a board that usually runs from end to end.	
Checking	A crack in the wood structure of a piece, usually running lengthwise. Checks are usually restricted to the end of a board and do not penetrate as far as the opposite side of a piece of sawn timber.	
Crook	Warping along the edge from one end to the other. This is most common in wood that was cut from the centre of the tree near the pith.	
Cupping	Warping along the face of a board across the width of the board. This defect is most common of plain-sawn lumber.	
Split	A longitudinal separation of the fibres which extends to the opposite face of a piece of sawn timber.	
Twist	Warping in lumber where the ends twist in opposite directions.	
Wane	The presence of bark or absence of wood on corners of a piece of lumber.	
Blue Stain	A discoloration that penetrates the wood fibre. It can be any colour other than the natural colour of the piece in which it is found. It is classed as light, medium or heavy and is generally blue or brown.	

Machine Burn	A darkening of the wood due to overheating by the machine knives or rolls when pieces are stopped in a machine.	
Pitch	An accumulation of resinous material on the surface or in pockets below the surface of wood. Also called gum or sap.	
Loose Knot	A knot that cannot be relied upon to remain in place in the piece. Caused by a dead branch that was not fully integrated into the tree before it was cut down.	
Tight Knot	A knot fixed by growth or position in the wood structure so that it firmly retains its place in the surrounding wood.	
Wormholes	Small holes in the wood caused by insects and beetles.	

Source of Pictures: Oduro & Gotah, 2008

2.1.6 Moisture Content

The amount of moisture in a living tree varies among species, in individual trees within the same species, in different parts of the same tree, and between heartwood and sapwood.

Moisture content (MC) is the weight of the water contained in wood, expressed as a percentage of the weight of the oven-dry wood. An oven-dry condition is reached when no further loss of weight is experienced on subsequent oven drying. Green wood has moisture both within the cell wall and in the cell cavity itself. As wood dries, the moisture leaves the ~~cell cavity~~ before leaving the cell wall. The point where the cell cavity is empty but the cell walls are full is known as the fibre saturation point. The fibre saturation point is quite variable between species but averages about 30% MC for most species (Halperin & Bible, 1994). Change in moisture content above the fibre saturation point has no effect on the wood's properties; all shrinkage occurs as the wood dries from 30% to a lower MC value. The increase in strength of lumber also occurs in this range as the wood dries i.e. decreasing moisture content below 30% MC increases the strength of a piece of lumber. Alik and Nakai (1997b) in their study also showed that above fibre saturation point, the strength values of timber appeared to be constant as the moisture content increased. In this case, within the green condition of a timber, an increase or loss of moisture has no effect on the strength of a timber.

Wood shrinks at different rates, depending on the direction relative to grain. The most shrinkage is tangential to the grain; shrinkage radial to grain is somewhat less; and shrinkage parallel to grain is so small as to be neglected. Figure 2.2 identifies these directions.

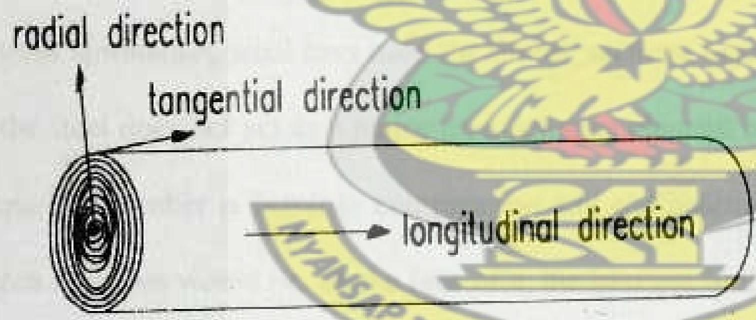


Figure 2.2: Radial, tangential and longitudinal directions of wood

Wood in use gives off or takes on moisture from the surrounding atmosphere with changes in temperature ~~and~~ relative humidity until it attains a balance relative to the atmospheric conditions. The moisture content at this point of balance is the equilibrium moisture content (EMC).

2.2 BENEFITS OF TIMBER

Timber is the original sustainable construction material as it can be replaced by replanting after it has been harvested. It is carbon neutral if it is burnt as it only returns the carbon dioxide to the atmosphere that it extracted during growth. Even better, if it is used in a structure, it locks carbon on the ground.

Timber has an acceptable public face. People like to see it and it is easy to make it look attractive. Unfortunately it is not regarded as a major structural element because it is edible (it rots) and is seen as inferior to steel and concrete.

Structural designers as well as the general public share this view, which is why much effort and money is currently being spent to raise its profile. Timber is not as strong in tension and compression as steel and reinforced concrete but is versatile, with some ductility and can easily be combined with other materials in a composite construction to compensate for its relative weakness.

In stress lamination, steel bars hold the timber sections together to help them perform better but the steel does not act as a major load-resisting element of the structure. As a direct stress comparison, timber is better in compression and end bearing than in tension and bending so an arch structure would be, on the face of it, the best use of the material structurally.

In countries where an established glue-laminated industry exists, laminated-veneer lumber and pulverized-strand lumber are sometimes used as the basic structural elements in place of solid timber sections. These materials are only available as imports in the UK at present so economically they are not viable. The reason why such engineered timber products are used is that larger sections can be made and at any length. This means spans can be greater and the stress reduction for jointing is removed.

2.3 STRUCTURAL USE OF TIMBER

The structural use of timber is as old, or older, than civilization. Long before the invention of mathematical sciences, early man was building rude shelters which, over the ages, by trial and no doubt considerable error, lead to patterns of timber construction which left their permanent imprints on the stone and marble temples of classical Greece (Chudnoff & Youngs, 1980).

In spite of its antiquity, timber has suffered some neglect at the hands of general practitioners in materials science and engineering, with the results that most engineering text books tend to give the impression that the application of structural theory is limited to steel or other man made materials. Yet, weight for weight, timber is as strong as or stronger than steel and much stronger than most of its structural competitors.

With the onset of industrial revolution, rot iron and steel sections became increasingly available and engineers devised effective way of fastening them together because the method of joining was easily available. Welding enabled effective and predictable fastening systems to develop for iron bridges and tall steel buildings (TRADA, 1985). They were also able to use reinforced concrete as a predictable material for construction so they tended to move away from wood as a structural material because of the following reasons: (1) wood behaviour was not predictable and you can only predict the behaviour of wood if sawn into standard size free from defects which are strength reducing and at an agreed moisture content and (2) fastening systems for wood did not keep pace with those available for steel. It was not until the 1950s that finger-jointing technology developed and also improved glue systems were developed.

The net result of structural research and development in the last 70 years has been the virtual abandonment of the structural techniques established so slowly and painfully in the preceding millennia (TRADA, 1985). The break with the past is nearly complete, the contemporary timber structure being different from its predecessor in both kind and degree. The raw material is the same but our understanding and control of its strengths and weaknesses are very different.

From purely structural point of view, if one had to pick out a single factor, which, more than any other is responsible for the change, it has to be the development of efficient tension joints. The pre-scientific age of timber construction had no effective means of utilizing the very high tensile strength of timber. Traditional carpentry aimed therefore at putting all joints into compression, which meant that the missing tensile forces had to be compensated with heavy masonry walls, buttresses and foundations.

The development of modern glues and timber connectors has enabled a wide range of structures to contain both tensile and compression forces within themselves, with consequent economy in their supporting elements (Hashemi & Smith, 2006). The introduction of tension joints has made a radical difference to timber structures, making them lighter and apparently more fragile. Except in very short lengths, all metals and timber are stronger in tension than in compression and in comparing modern structures with the massive buildings of the past it is perhaps salutary to give some thought to all the temples, churches and cathedrals which fell down to pay the price for those standing.

The selection of lumber for structural use depends greatly on the grade of lumber; the presence of defects and strength properties. Lumber is therefore classified into grades according to their stress properties.

2.4 STRESS GRADING

Lumber, as it is sawn from a log, is quite variable in its mechanical properties. Individual pieces may differ in strength by as much as 100% (American Institute of Timber Construction, 1994). For simplicity and economy in use, pieces of lumber of similar mechanical properties (such as bending strength, density and modulus of elasticity) can be placed in a single class known as a grade. The properties of a particular grade depend on the sorting criteria.

The strength of a timber is a function of several parameters including the moisture content, density, size of specimen and the presence of various strength-reducing characteristics such as knots, slope of grain, fissures and wane. Prior to the introduction of BS 5268, 1984 (British Standard for *Structural Use of Timber*), the strength of timber was determined by carrying out short-term loading tests on small timber specimens free from all defects. The data were 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 accidental overload, simplifying assumptions made during design and design inaccuracies, together with poor workmanship. Basic Stress was defined as the stress which could safely be permanently sustained by timber free from any strength-reducing characteristics (Arya, 2003)

Basic stress, however, was not directly applicable to structural size timber since structural size timber invariably contains defects, which further reduces its strength. To take account of this, timber was visually classified into one of four grades, namely 75, 65, 50, and 40, which indicated the percentage free from defects (Arya, 2003). The grade stress for structural timber was finally obtained by multiplying the grade designations expressed as a percentage (e.g. 75%, 65% etc.) by the basic stress for the timber.

With the introduction of BS 5268, the concept of basic stress was largely abandoned and a revised procedure for assessing the strength of timber adopted. From then on, the first stage involved grading structural size timber. Grading was still carried out visually, although it was a common practice to do this mechanically.

2.4.1 Visual Grading

Visual grading is the oldest timber grading method. It is based on the premise that mechanical properties of timber differ from mechanical properties of clear wood because of naturally occurring characteristics that can be seen and judged by eye (Kretschmann and Green, 1999). It is done based on both appearance strength factors (Chen et al, 2006). These visual characteristics are used to sort the timber into grades and include, but are not limited to, density, slope of grain, size and location of knots, checks and splits. When BS 5268 was published in 1984, the numbered grades (i.e. 75, 65, 50, and 40) were withdrawn and replaced by two visual grades: General Structural (GS) and Special Structural (SS).

The SS grade timber was used as the basis for strength and modulus of elasticity determinations by subjecting a large number of structural sized specimens to short-term load tests. The results were used to obtain the fifth percentile stresses, defined as the value below which not more than 5% of test results fell (Figure 2.3). The fifth percentile values for other grades of the same species were derived using grade relativity factors established from the same series of tests.

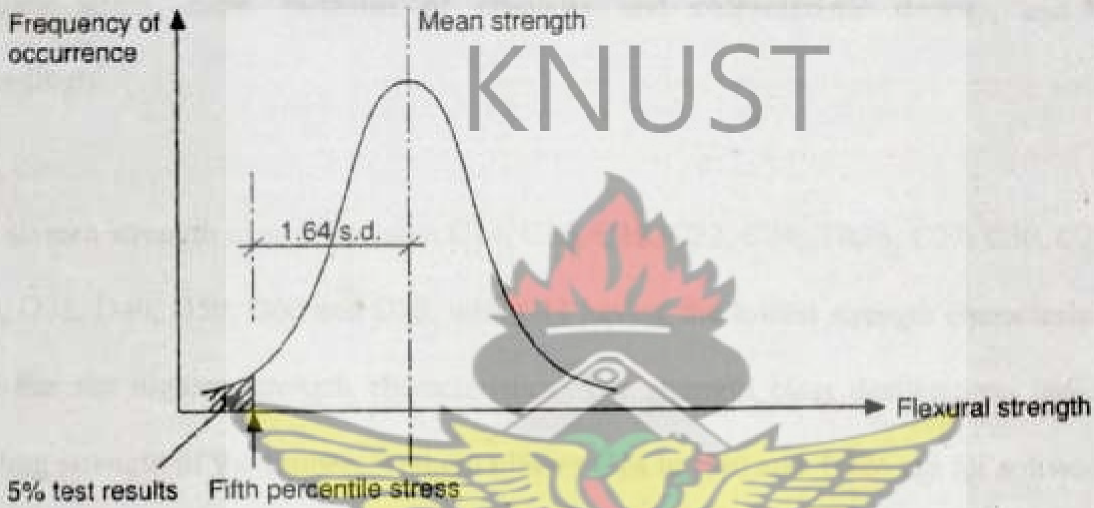


Figure 2.3: Fifth percentile stress from mean distribution diagram

2.4.2 Mechanical Stress Grading

Mechanical stress grading is based on the fact that there is a direct relationship between the modulus of elasticity measured over a relatively short span, i.e. stiffness, and bending strength. The E value of a timber is thus a major sorting criterion for this method of grading. The stiffness is assessed non-destructively by feeding individual pieces of timber through a series of rollers on a machine which automatically applies small transverse loads over short successive lengths and measures the deflections. These are compared with permitted deflections appropriate to given stress grades and the machine assesses the grade of the timber over its entire length.

2.4.3 Strength Classes

In the latest version of BS 5268 published in 2002, machine graded timber is now graded directly to one of sixteen strength classes defined in BS EN 519, principally on the basis of bending stress, mean modulus of elasticity and characteristic density, and marked accordingly.

The sixteen strength classes include: C14, C16, C18, C22, C24, TR26, C27, C30, C35, C40, D30, D35, D40, D50, D60 and D70, with C14 having the lowest strength characteristics and D70 has the highest strength characteristics. The strength class designations indicate the bending strength of the timber. Strength classes C14 to C40 and TR26 are for softwoods and D30 to D70 are for hardwoods. The letters 'C' and 'D' denote softwood (Coniferous) and hardwood (Deciduous) respectively and the values 14, 16, up to 70 denote the bending strength in N/mm^2. TR26 is softwood and is intended for use in the design of trussed rafters ('TR' denotes 'Trussed Rafter').

2.5 EVOLUTION OF TIMBER DESIGN

Man's earliest form of habitation was provided by nature. As time went on he was forced to construct habitations for himself and not rely on what was provided for him. His first building materials were branches of trees, broken off by the wind laid against each other and tied at the top with supple twigs to form a free-standing tent-like structure, covered by turf, leaves or skin. Gradually this gave way to the circular hut, which spread out, to a rectangular building on plan with the branches sloping up to a ridge from either long side.

More recently, however, engineers and architects have learned to design wood structures in ways that are based on engineering principles. Thus today's designers, using more rigorous design procedures, are able to ensure that a particular design will achieve the desired level of structural safety and stiffness, as well as economy. Much progress has been made in moving from rule of thumb to modern day engineered designs. For example, in New South Wales, timber truss road bridges evolved over at least five distinct stages culminating in the impressive trimmer, more efficient and unique composite designs of the early 1900s (Grant, 2006). To design effectively, a structural designer must be familiar with the properties and behaviour of the material to be used.

Unfortunately, the properties and behaviour of wood are unlike those for other building materials and much more complex. Wood is a natural material – one over which, as yet, we have little control. Through forest management, we are beginning to control certain characteristics of the lumber a forest will produce. However, we still cannot force a tree to produce a particular quality of material, nor compel different trees in a forest to produce exactly the same quality of material.

Timber has a number of advantages. This includes its versatility - can be finished in a number of different ways to different products (roofs, bridges, doors, windows, panels, furniture, etc.); beauty; cost effectiveness and environmentally friendly. Timber is easy to ~~work, light~~ but strong, available in a wide range of species, sizes and finishes. Timber is an excellent insulator and uses less energy in its production than any other building material. It is a natural resource, and sustainable.

For most structural applications, fortunately, wood's advantages far outweigh its disadvantages. However, structural designers must learn to cope with (1) wood's variability in its mechanical properties and (2) its response to environmental conditions. Variability is the more serious of these. Wood properties vary from species to species, from one position to another in the tree, from one tree to another grown in the same locality, and between trees grown in one locality and those grown in another. Strides are being made to control the quality of wood a tree produces by means of selective tree farming. It is hoped that by such means straighter and fast growing trees may be developed with more nearly uniform properties than found in trees from natural forests (Stalnaker and Harris, 1989).

The moisture content of wood installed in a structure may change with time, eventually reaching equilibrium moisture content that depends on the average relative humidity of the surroundings. However, as the relative humidity within a building may not be constant, the equilibrium moisture content may vary with time. With any change of moisture content, wood will either shrink or swell and warp. Dimension and shape change due to moisture change can be reduced or avoided by proper seasoning and by proper attention to details of design (Stalnaker and Harris, 1989).

Wood may be described as an orthotropic material that is it has unique and independent mechanical properties in the directions of three mutually perpendicular axes: longitudinal, radial and tangential (Halperin & Bible, 1994). Mechanical properties most commonly measured and represented as strength properties for design include modulus of rupture in bending, compressive stress perpendicular to grain, and shear strength parallel to grain. Other properties involving time under load include creep, creep rupture or duration of load and fatigue. (Forest Products Research Laboratory, 1998)

2.6 RESEARCH TRENDS ON GHANAIAN TIMBER SPECIES

Timber trade in Ghana, as is the case in most developing countries, is export oriented; consequently, research into wood microbiology as with all research in the sector has been geared towards improving the trade (Moor, 1940; Freas *et al*, 1973). Since Ghana was noted as being among countries with relatively few species in the timber trade (Freas et al 1973), a lot of research into some of the secondary species has been done by the Forest Research Institute of Ghana in Kumasi. Some of the works concern mechanical wood properties (Ashiabor, 1967; Bentum, 1969; Bentum, 1970), wood seasoning characteristics (Ofori 1985), electrical resistivity of wood species in relation to moisture (Okoh, 1977b), anatomical properties (Ayensu and Bentum, 1974; Ocloo and Laing, 1991). Work on the uses of the species was done by (Bolza and Keating, 1972; Ayensu and Bentum, 1974); weathering performance (Bentum and Addo-Ashong, 1977) and corrosion resistance (Ofori, 1987). These tests have so far looked mainly at the traditional commercial species. Lately, there have been a number of researches on lesser-used species. A number of the researches on lesser known timber species have been conducted at the Forestry Research Institute of Ghana ~~(FORIG)~~. Most of the works or researches conducted centred on durability and the extractives of the lesser known species. Works done on their bending and other mechanical properties were performed with clear small size specimens.

Anatomical properties of the lesser used species have been conducted by (Oteng-Amoako et al, 1998), natural decay resistance by (Kumi-Woode, 1996), decay tests using white rot fungi and brown rot fungi (Huang, 2004). Addae-Mensah, 1998, investigated the industrial utilization and marketing of some Ghanaian lesser-used timber species.

Kumi-Woode (1996) did some work on 14 less-utilized species; including *Amphimas pterocarpoides* (Yaya), *Antiaris toxicaria* (Kyenkyen) and *Canarium schweinfurthii* (Bediwonua). He found out that *Amphimas pterocarpoides* (Yaya) was not durable when subjected to fungal attack. Nyarko (1999) also subjected *Amphimas pterocarpoides* (Yaya) to termite attack and found out that, its durability was highly increased when treated with creosote and moderately high when untreated. Huang (2004), also working on some lesser-utilized Ghanaian tropical hardwoods, analysing decay by white rot and brown rot fungi, published that *Sterculia rhinopetala* (Wawabima) is very durable after 12 weeks exposure to various fungi.

2.7 RESEARCH TRENDS ON TIMBER BRIDGES

In Ghana bridges are usually constructed by using concrete and reinforced concrete, steel, and the composition of steel, timber or concrete. Major bridges linking highways over rivers and railways were made of steel e.g. the Adomi Bridge spanning about 242m (805 ft) and has a width of about 7m (22 ft), which crosses the Volta Lake at Atimpoku, 10km from Akosombo, is made of steel. Smaller streams are crossed by the construction of concrete culverts. Timber in bridges is usually used in composition with other materials either as the walking surface or guardrail. An example is the pedestrian walkway at Railways near Asafo, Kumasi crossing the railway to Adum in Kumasi (Figure 2.4). Its structural frame is made of steel trusses and the walking surface is made of timber (Milicia *excelsa* - Odum) beams.

Figure 2.4: Pedestrian walkway at Railways near Asafo, Kumasi

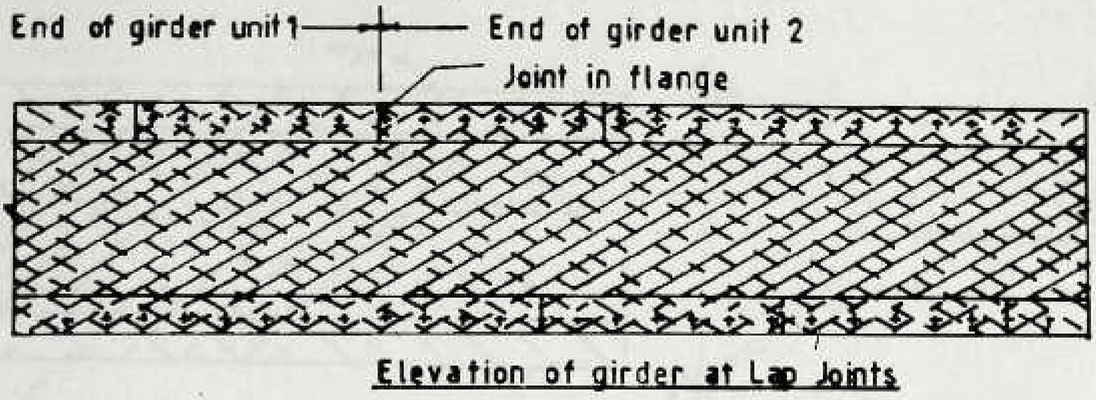
2.7.1 Non-Engineered Bridges

Local people in communities with water barriers have tried to put up timber bridges to aid movement from one part to another. These bridges which are non-engineered are constructed easily and deteriorate in no time. There are however several pedestrian timber bridges across the country in many communities constructed by local people who have no technical know how. This puts the lives of people in danger for using bridges in such deplorable states. Since these bridges are not designed and constructed by technicians and carpenters specialised in timber structures, it leads to expensive, unsafe and low durability timber bridges. The actual state of degradation of these bridges shows a very negative picture of the use of wood as a structural material.

There are many streams in towns and village, farm roads which need bridges. Properly engineered bridges made of lesser known timber species which have been researched can provide safe and economic solutions to these numerous transportation problems. It is therefore important to research into wood technology in the construction and rehabilitation of timber bridges.

2.7.2 Engineered Bridges

In 1990, Dr. Allotey, a Principal Research Officer at the Building and Road Research Institute (BRRI) of the CSIR, worked on the use of timber for bridges by introducing a new timber plate girder built out of timber and bolts. According to Allotey, the reasons for the selection of this type of girder for his design were: (1) to minimize the use of steel gusset plates which are imported; (2) to continue the traditional use of beam girders for bridge construction which Ghanaian contractors are familiar with; (3) to increase the options available for design in timber for river crossings; and (4) to select a unit that could be easily fabricated in a workshop before being sent to the construction site. Minimizing the use of steel gusset plates by Allotey increases the use of too much timber in the design. This in turn increases the cost of timber for constructing such bridges. The first prototype was built in 1991 at Kaasi near the Ahodwo Roundabout in Kumasi. The substructure of the bridge was constructed using 22 timber end bearing piles which continue to act as abutment columns. The spacing of the piles is 1.83m and Dahoma pile caps support the bridge girders. The girders are 7.62m spans from bearing to bearing with an overall length of 8.23m. The height of each girder is 1.143m. The bridge floor deck is built from 50mm x 150mm sawn timber brought together by nails. There is a wide spectrum of timber species in Ghana's forests. However, three different species were used in Allotey's design: *Strombosia glaucescens* (Afina) was used for the piles; *Piptadeniastrum africana* (Dahoma) was used for the pile cap and floor deck; and *Nauclea diderrichii* (Kusia) was used for the retaining wall. Afina is a durable and strong wood which was being used as poles, posts and as sleepers. It does not grow big in diameter, usually up to 70cm, which makes it suitable to be used as piles. Dahoma and Kusia were used for the decking and retaining walls respectively because they are all very durable and strong wood, with minimum felling diameter of 70cm and 110cm respectively (Oteng-Amoako et al, 2006). Kusia was more durable than Dahoma so was used as protective (cladding) members. Dahoma was also more available in the market and so could be obtained easily in large quantities for the pile capping and decking (which needed more wood). The use of many species of timber in a construction makes material selection cumbersome and can increase considerably the construction time.



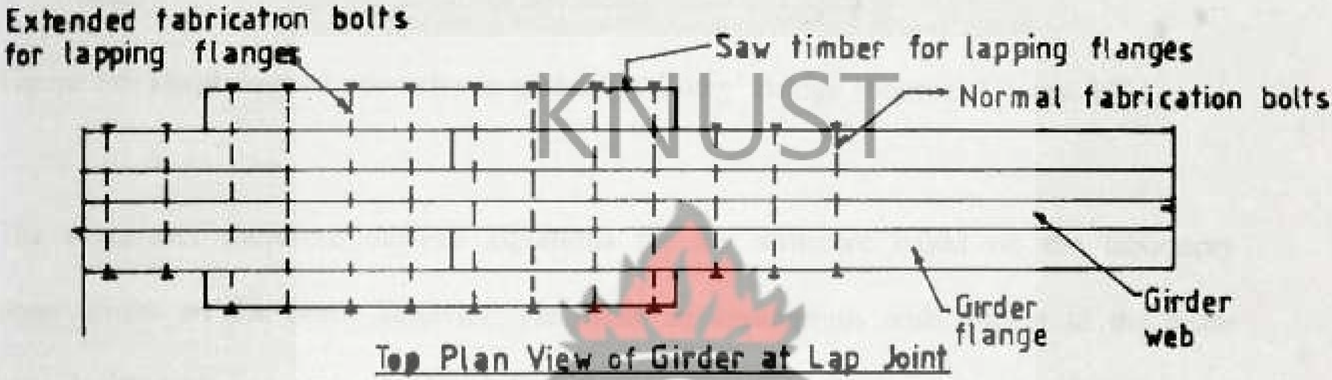


Figure 2.5: Elevation and Plan views of Allotey's Bridge (Source: Allotey, 1991)

Allotey's theoretical analysis for the design was made by considering every bolt position in the beam as a joint and the material between bolts as a beam member. In this way finite element solution may be applied to the beam to obtain strains and beam stresses in the beam. Such an analysis has two defects. These are:

i. the process is cumbersome and does not lend itself easily to office design.

ii. due to the number of bolts, friction forces arise between members and produces results different from what the above may predict.

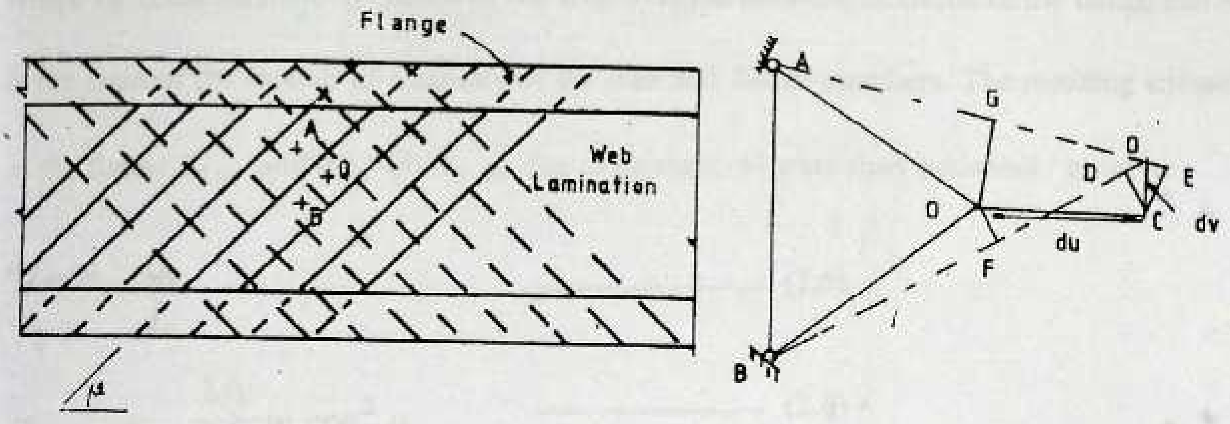


Figure 2.6: Displacement geometry in girder of Allotey' Bridge (Source: Allotey, 1991)

The researcher therefore derived equations for the structure based on the laboratory observations on the beam behavior. He made an assumption with respect to the beam behavior that, plane sections in the girder before bending, remain plane after bending.

Based upon these assumptions, and the displacement geometry (Figure 2.6), the general strain-displacement equation for the web lamination may be derived as

$$\varepsilon_w = \frac{\partial u}{\partial x}\cos^2\mu \pm \frac{\partial v}{\partial x}\sin\mu\cos\mu \qquad (2.3)$$

where ε_w is the strain in web and u, v are horizontal and vertical displacements. This displacement equation was first derived by Ghaboussi (1986), and μ is the angle the orientation of web members makes with the position of bolts (bolt line) in the members.

Stresses due to moments were determined using equation 2.3 and the above assumption, the effective moment of inertia of the section was derived as:

$$I_e = mI_w\cos^4\mu + I_f \qquad (2.4)$$

where I_W is the moment of inertia of the web. I_f is the moment of inertia of the flange and m is the ratio of the moduli of elasticity of the web and flange members. The resulting stresses in the flange $\sigma_{f,m}$ and the web $\sigma_{w,m}$ due to moment, M were then evaluated / given as:

$$\sigma_{f,m} = \frac{My}{I_e} \qquad (2.5)$$

$$\sigma_{w,m} = \frac{My}{I_e} m \cos^2 \mu \qquad (2.6)$$

where y is the vertical distance from the neutral axis

Allotey's work has shown that, for a developing country which abounds with wood, the utilization of wood for bridges is cost effective than to import cement and steel.

The use of the structural equations (some stated above), enables the structural proportioning for moment, shear and deflection to be effected in the design. However, the researcher could have considered using practical situations instead of basing analysis on only laboratory observations of beam behaviour. Friction forces arising between the members due to the bolting of the members could produce results different from what the above may predict.

By making use of the geometrical relationship developed in the design, the girder can be correctly setout which makes it easy to be constructed by local artisans.

Lecomte (2006) worked on the use of *eucalyptus* timber log species for construction to supplement the use of tropical sawn timber in Brazil. According to the paper, high or inaccessible prices and lack of regular supply of tropical sawn lumber by the lumber industry necessitated the research into the use of the exotic species (*eucalyptus*) which is fast growing and adapted excellently to their climate and soil conditions. This new species, which was introduced by the paper and cellulose industry, with time were then consolidated as a viable alternative for wood supply in Brazil. The researcher demonstrated the use of the eucalyptus logs in the construction of two prototype houses in Brasilia (one in a rural dwelling and the other in an urban dwelling) in combination with sawn lumber. This greatly reduced the cost of building and enabled the construction of medium cost houses and bridges in central Brazil. *Eucalyptus* logs are being patronized by Architects, Engineers, and Builders and accepted as an appropriate material in the building industry of Brazil.

In 2001, a Timber Bridge Program was started at Sao Paolo University in Brazil with the objectives of developing new technologies for timber bridge construction, the analysis and improvement of existing structural systems, and the adaptation of international technologies to local conditions (Calil, 2006). The aim of the program therefore, was to have technology to be able to construct safe and economic timber bridges, with simple and modern techniques, and with good durability like other structural materials. According to Calil (2006), most timber bridges in Brazil are not designed and constructed by technical personnel Engineers and Contractors) specialized in timber structures. This led to expensive, unsafe and low durability timber bridges. The state of degradation of these bridges shows a very negative picture of the use of wood as structural material. It therefore became very important to develop wood technology in the construction and rehabilitation of timber bridges in the country. The same can be said of Ghana, where many non-engineered timber bridges are in deplorable state (Allottey, 1991). After four years of the Sao Paolo program, eleven demonstration timber bridges were in the state of Sao Paolo. The bridges were vehicular bridges with spans ranging between 6m – 23m. The 6-7m span bridges were designed with $30KN/m^2$ design live loads and the 8 – 23m span bridges were designed with $45KN/m^2$ design live loads.

2.8 RESEARCH TREND ON STRENGTH PROPERTIES OF TIMBER

Alik and Badorul (2006) worked on the strength performance of structural-size timber of six species in Malaysia. Their objective was to evaluate the strength performance of full size structural timber of the species. The species worked on include *Dryobalanops beccarii*, *Dryobalanops fusca*, *Dryobalanops oblongifolia*, *Dryobalanops lanceolata*, *Dryobalanops rappa* and *Dryobalanops sumatrensis* which are all found in Sarawak, Malaysia. The Dryobalanops species are all softwoods which are lesser utilized and their properties needed to be established for commercial utilization. The species had not yet been classified under the European standards for the classification of timber, EN338. Samples with nominal dimensions of 50mm by 100mm were used for the tests.

Timber beams of dimensions 50mm by 100mm by 3000mm were used for static bending test. Strength values were determined based on British Standards BS 5820:1979 and BS 373:1957. Forty (40) samples of each species were used for the mechanical tests. Alik and Badorul (2006) also tried to determine the correlation between small clear and structural size timber in terms of their modulus of rupture (bending strength). Table 2.2 below summarizes the results obtained by the authors. The moisture contents of the samples were high because the test was conducted at the green condition of the species (i.e. the samples were not dried before the test).

Table 2.2: Results of Alik and Badorul's test on Dryobalanops species (2006)

Timber Species	No. of samples tested	Structural Size: Modulus of elastic. MOE (GPa)	Bending strength MOR (MPa)	Tensile strength TS (MPa)	Density (g/cm^3)	Moisture content (%)	Small Clear Size: Bending strength MOR (MPa)	Modulus of elasticity MOE (GPa)
D. beccarii	40	19.21	74.2	69.38	0.7	56.61	106.58	14.9
D. fusca	40	17.76	65.14	57.88	0.63	65.25	89.9	12.7
D. oblongifolia	40	18.68	67.58	65.15	0.67	52.16	93.71	13.56
D. lanceolata	40	19.44	69.3	61.45	0.68	54.62	96.19	13.68
D. rappa	40	16.83	61.32	57.65	0.64	53.28	81.76	13.28
D. sumatrensis	40	19.57	71.28	61.07	0.68	42.36	95.49	13.96

Structural size dimension = 50mm x 100mm x 3000mm (for bending test)
50mm x 100mm x 2000mm (for tensile test)
Small clear size dimensions = 20mm x 20mm x 300mm

The results obviously showed a weak correlation between small clear and structural size timber in terms of their modulus of rupture. The conclusion that could be deduced from this study is that the strength values obtained from small clear wood specimens did not work well to be used to correlate the strength of full size structural timber. This is because the structural size members contain possible defects such as knots, checks and cracks which reduce their strength. Small clear sample tend to have higher strength values because of the lack of defects which is not the case in structural timber. It was calculated that the strength ratios of almost defect-free structural size to small clear specimens of the species were 0.75 and 0.77 at green and air-dried conditions of the wood respectively. Their finding was consistent with the study conducted by Alik and Nakai (1997a), who used *Dipterocarp* species. Based on their density, the species could be classified under medium hardwood species.

The moisture content of the species ranged from 42.36% to 65.25% (Table 2.2) with a mean value of 54%. This range of moisture content is considered as green condition since the moisture content had far exceeded the fibre saturation point. Alik and Nakai (1997b) in their study showed that above fibre saturation point, the strength values appeared to be constant as the moisture content increased. In this case, within the green condition, there was no effect on the strength of the timber with an increase or loss of moisture content.

Vries et al (2006) investigated the strength characteristics of round wood and the establishment of a grading system to classify them in the Netherlands. The investigation dealt with the determination of the modulus of elasticity and the bending strength. The wood species concerned was larch (*Larix kaempferi*), grown in The Netherlands with diameters between 80mm and 140mm. The reasons why round timber is not used in construction in The Netherlands which necessitated the investigation included the unavailability of strength values of the species and also connections for engineers; and lack of standards with regard to strength and test methods.

Two Europeans standards, EN 14251:2003 (which deals with test methods for structural timber of round cross-section) and prEN 14544:2005 (which deals with the requirements on strength of round timber) were adapted in their investigations. For the design of the test arrangement, the standard EN 408 for sawn timber was used. This could affect the results obtained by Vries et al since EN408 is specifically for sawn timber.

About 50 specimens were tested under 4 sample groups of different diameters: 100mm, 120mm, 140mm and tapered 140mm (Table 2.3). The authors also tried to compare their results with EN 338 in their analysis and obtained the strength class of larch round timber grown in The Netherlands as C40 (where C denotes Softwood; and 40 denotes the bending strength in N/mm^2). Table 2.3 below presents the results obtained by the authors.

Table 2.3: Characteristic values for larch round timber and strength class according to EN338 (Results of Vries et al work)

	Sample 1 d=100mm	Sample 2 d=120mm	Sample 3 d=140mm	Sample 4 Tapered (140mm)	Total Sample
No. of samples tested	44	54	68	39	205
f_{mk} (N/mm^2)	43	48	31	50	40
$E_{0,mean}$ (N/mm^2)	14600	15860	15300	16175	15463
$E_{0,05}$ (N/mm^2)	9747	11337	10096	10686	10460
ρ_{mean} (Kg/m^3)	575	583	578	572	577
Strength Class	C35	C45	C30	C35	C40

The results in Table 2.3 indicate that the overall strength class for the larch population grown in the Netherlands is C40. Since the characteristic bending strength of 'Sample 3' is lower than that of the 'Total Sample', the population criteria has to be adjusted according to EN 384 otherwise the 'Total sample' has to be assigned C35 strength class. The researchers also noted that the characteristic values of other strength properties (tension, compression, etc.) which are derived from the key values in EN 338 have to be reconsidered and established for round timber.

Round wood poles normally have some curvature which may force the positioning of a pole in the test arrangement to change during loading. This means that, it is not possible to measure the correct cross section dimensions for the calculation of the moment of inertia until after the pole is placed in the test set up. If this is not taken into consideration, results can be greatly affected. However, the researchers took their measurements after the poles had been securely fixed in the set up, which was a good practice and makes their results genuine.

Chen et al (2006) investigated the bending strength and modulus of elasticity of British Columbia coastal Douglas-fir and Hem-fir timber in Canada. Douglas-fir and Hem-fir species are softwoods which belong to the strength class of C16 for General Structural (GS) and C24 for Special Structural (SS) according to BS 5268:1984. The species were sampled in two sizes, i.e. 105mm x 210mm and 105mm x 305mm in cross section for both species. The specimens were also graded into Select Structural (SS), No.1 and No.2 grades according to the Canadian Lumber Grades Authority Standard grading rules. SS is the highest grade, followed by No.1 and No.2 is the least grade.

The bending strength and modulus of elasticity were measured in a proof loading test which was conducted in accordance with the requirements of ASTM D 4761 and Japanese test protocol requirement (the researcher being a Japanese student). Their research was conducted to obtain accurate material property information for the development of refined design procedures which will benefit their construction industry, especially in North America where timber is widely used for residential and light office and industrial buildings. The characteristic mean modulus of elasticity and the characteristic 5th percentile bending strength were derived as a function of member size, species and grade. The results obtained for the SS and No.1 grades were similar so the table below summarizes the results of SS and No.2 grades of the samples for the two species.

Table 2.4: Summary results of Chen et al test on Hem-fir and Douglas-fir

	Hem-fir		Douglas-fir	
	105 x 210mm	105 x 305mm	105 x 210mm	105 x 305mm
	SS	SS	SS	SS
Mean MOE (GPa)	12.27	12.14	13.51	12.27
Bending strength, MOR (MPa)	30.08	26.8	24.13	23.39
	No.2	No.2	No.2	No.2
Mean MOE (GPa)	12.07	12.07	11.24	11.52
Bending strength, MOR (MPa)	25.77	17.16	15.21	15.24

The authors did not specify the sample size for the test which makes evaluating the results difficult but is evident from the table that the strength values of the No.2 grade samples are lower than those of the SS grade samples. This is because lower grades of lumber have more defects in them than in higher grades which reduce their strength.

Solli, 1999 investigated the differences between the local and global modulus of elasticity (MOE) in bending of structural timber. The European test method for the determination of the MOE in bending of structural timber, EN 408:1995 was used in his work.

According to Solli, there have been several discussions whether the local or the global value is the most representative value of the bending stiffness. A number of arguments have been raised in the discussions. Some researchers believe that since the local MOE is the current system being used and works well, there is no need to welcome a new system of global MOE, whose possible consequences are unknown. The local MOE is well known in the European strength class system (EN 338:1995), so with new system of values, designing engineers might be confused. The researcher also argued that the limits of deflection given in the European building regulations are based on design by local MOE. Some researchers, on the other hand argued that the local MOE is not the correct value when the deflection of a floor shall be calculated. The argument is that the local value as described in EN 408:1995 is based on the critical section and therefore cannot be representative for a whole span. They also believe that the test procedure of global MOE is easier and timesaving compared with the corresponding local MOE test procedure. The weight of this argument is obviously dependent on what kind of test equipment is used.

On this background, the Norwegian Institute of Wood Technology (NTI) carried out a project where both the local and the global MOE in bending were measured on 200 pieces of Norway Spruce (Picea abies). In addition to the two MOE values the bending strength, density and moisture content were also measured. The dimensions of the pieces were 48mm x 123mm (100 pieces) and 48mm x 148mm (100 pieces).

All 200 pieces were tested in bending (MOE and bending strength). MOE was determined according to *EN 408:1995* (local MOE) and *prEN 408:1995 rev.* (global MOE). Both the local and global deflections were recorded continuously at a mid-span bending stress interval from 5 to 10 N/mm². The results obtained by Solli are shown in Table 2.5 below:

Table 2.5: Results from the bending test of Solli

	Bending strength (N/mm^2)	MOE_{local} (N/mm^2)	MOE_{global} (N/mm^2)	Density (kg/m^3)	Moisture Content (%)
Maximum	79.1	17359	15822	613.2	14.3
Minimum	13.0	4958	5510	338.3	11.5
Mean	40.4	10708	9780	458.9	13.4
Std. Dev.	12.6	2421	1931	50.3	0.4
COV (%)	31.1	22.6	19.7	11.0	3.2

The results indicate a good correlation between the local and global MOE in bending. Based on Solli's results, the relation can be expressed by the equation:

$$MOE_{LOCAL} = 1.18\ MOE_{GLOBAL} - 856 \qquad R^2 = 0.89 \qquad (2.1)$$

The researcher further determined the relation between the bending strength, f_m, and the MOE_{local}; and the relation between the bending strength, f_m, and the MOE_{global} as follows:

$$f_m = 4.42 \times 10^{-3}\ MOE_{LOCAL} - 5.3 \qquad R^2 = 0.65 \qquad (2.2)$$

$$f_m = 5.39 \times 10^{-3}\ MOE_{GLOBAL} - 9.8 \qquad R^2 = 0.58 \qquad (2.3)$$

where R^2 is the coefficient of determination (see section 3.2.5)

Solli concluded that the MOE_{global} is not as sensitive to inaccurate measurements as MOE_{local} since the global deflection is about ten times the local. The MOE_{local} is in principle based on pure bending deflection whilst MOE_{global} is also influenced by shear deflection. A measurement of the MOE_{global} contains a higher number of possible sources of error such as the initial twisting of test pieces during testing. If the intended use of MOE is to estimate the corresponding bending strength of a piece of timber MOE_{local} is the unrivalled alternative of the two methods. From equations 2.2 and 2.3, it easier to agree with Solli that the local modulus is a better alternative of the two methods because, equation 2.2 gives a better correlation between f_m and MOE_{LOCAL} of 0.65 compared to equation 2.3 with a correlation of 0.58 between f_m and MOE_{GLOBAL}

2.9 LITERATURE ABOUT SELECTED SPECIES FOR THE RESEARCH

Ten tropical timber species belonging to the lesser-known ones were studied. These are: *Albizia ferruginea* (Awiemfosamina), *Amphimas pterocarpoides*(Yaya), *Antiaris toxicaria* (Kyenkyen), *Blighia sapida* (Akye), *Canarium schwienfurthii* (Bediwonua), *Celtis zenkeri* (Esa),*Cola gigantea* (Watapuo), *Petersianthus macrocarpus* (Esia), *Sterculia oblonga* (Ohaa),and *Sterculia rhinopetela* (Wawabima). The species were selected based on their availability and distribution in the forest. Physical features like growth diameter, and length of bole were also considered. Literature on properties of lesser known species in Ghana, including natural durability, density and exploitation status also guided in the selection of the above species for the study.

Albizia ferruginea (Awiemfosamina) belongs to the family of the Mimosaceae. The tree grows to a height of 37 m or more, diameter about 3.0m, with a clear, straight bole of length 20 m. The heartwood colour is variable from mid-brown to dark red-brown, with an attractive appearance. Its leaflets are softly hairy, with long ginger hairs especially around petiole, and a tiny thickened acumen. Its bark is thick, rough and scaly (Hawthorne, 1990). Sapwood colour is distinct from heartwood, pale yellow or straw-coloured, about 50 mm wide. It has interlocked grain, which is sometime variable in direction and has a coarse texture. Its density ranges from 580 to 820 kg/m^3, when seasoned averages about 700 kg/m^3. Its bending strength is reported to be generally lower than European beech but varies with density. Limited tests indicate that it dries with little degrade but very slowly in big sizes. It is extremely resistant to preservative treatment though sapwood is permeable. Sapwood is liable to attack by powder post beetles and the British Building Research Establishment (BBRE) reports heartwood to be highly resistant to termites and very durable. Machining properties are satisfactory but with a tendency to breaking out when machined across the grain, nailing is satisfactory when pre-bored. It is generally used for crates, boxes, mouldings, flooring, light structural work, exterior joinery and wooden houses (Abbiw, 1990).

Amphimas pterocarpoides (Yaya) belongs to the family of the Papilionaceae. The tree grows to a height of about 50 m and attains a girth of up to 3-4 m. It has a straight bole which is cylindrical up to about 25 m with small blunt buttresses. The bark is usually about 5mm thick for a matured tree and it is dark-grey to blackish in colour (Hawthorne, 1990). Irvine (1961) indicated that the distribution of Yaya ranges from Guinea to Cameroon. Hall and Swane (1981) have found the species abundant in the semi-deciduous and evergreen forests of Ghana. It is a light coloured interior species of attractive appearance. It has a density of 770 kg/m^3 at 12-15% moisture content. It is generally used for mouldings, doors, plywood, veneer, furniture, trim, paneling, and interior joinery. It is reported, by the British Building Research Establishment (BBRE), to have medium strength with a low natural durability.

Antiaris toxicaria (Kyenkyen) belongs to the family of the Moraceae. It grows to a height of 40 to 50 m with a clear, straight, cylindrical bole up to 21m and diameter of about 5 m. It is found in West, Central and East Africa. Its colour is white or light yellow-brown with sapwood, which is not visually distinguishable from the heartwood, but may be up to 150 mm thick. It has interlocked grain with a texture of medium to coarse. Its density ranges from 370 to 530 kg/m^3 at 12 % moisture content. It tends to dry rapidly but tends to distort. Twist may be a serious defect and thick material tends to end split. *Antiaris toxicaria* machines well but cutting edges must be kept sharp to prevent tearing during machining operations across the end grain. Nailing is satisfactory with poor bending properties. Both heartwood and sapwood are reported to be very perishable but permeable for preservative treatment. The sapwood is very susceptible to sapstain when green and logs liable to severe attack by ambrosia beetles and may extend to the heartwood. Successful utilization depends on rapid extraction from the forest and protection from degrades by insect and fungal attack including blue stains. It is mainly used for toys, shelving, boxes, tool handles, carving, trim, interior joinery, plywood, sliced and rotary veneer.

Blighia sapida (Akye) is of the family of the Sapindaceae. The tree grows to a height of 25 m tall and 3 m in girth with short and crooked bole up to 15 m with heavy crown. Leaves alternate, pinnate often with more than three pairs of dark-green, obviate-oblong leaflets (Hawthorne, 1990). Ripe fruits are pear-shaped, red or yellow-tinged red, splitting into three parts each with one shinning white, fleshy aril at the base. The arils are edible when fresh and ripe but the unripe or fallen fruit contain poisonous hypoglycin compound. It is native to West Africa but has spread to other parts of tropical America. No data and documentation have been found on it strength properties and durability. However, another species *Blighia* *unijugata* of the same family has been noted to be durable and used for pestles and as building material (Abbiw, 1990).

Canarium schwienfurthii (Bediwonua), belongs to the family of the Burseraceae. The tree grows to a height of 50 m with a straight, cylindrical bole of length 27 m and a diameter of about 4 m. It has very small buttresses. It is widely distributed in East, Central and West Africa. Its colour is pale pinkish-brown. The sapwood is white or straw coloured with thickness up to 100 mm. Its grain is interlocked, sometimes producing a very attractive stripe or roe figure when cut on the quarter. Texture is coarse. It has a density of about 530 kg/m^3 at 12 % moisture content. Sometimes brittleheart is found in large logs. The freshly cut timber has a pleasant scent. It dries rather slowly and fairly well. Tendency to end split and for original shakes to extend. To give a good finish on interlocked faces, cutters should be kept sharp otherwise, a wooly finish is obtained. It has a very poor bending strength. The sapwood is liable to attack by powder-post beetles and reported to be non-resistant to termites in West Africa. Its heartwood is non-durable and extremely resistant to preservative treatment though sapwood is permeable. Because of its severe blunting effect on cutting edges, its uses are restricted. It is, though, suitable for cores of plywood and can be sliced to produce decorative paneling, as it has an attractive figure and may readily be stained.

Celtis zenkeri (Esa) is of the family of the Ulmaceae. It is a lesser-utilised species with a tree which grows to a height of 27 to 36 m. It has a clear bole with a diameter of 3 m with long buttresses. It has leaves with strongly scalariform venation and dense hairs. Bark is often yellow. It is widely distributed in tropical Africa. The heartwood and sapwood is not easily distinguishable. Whitish or clear light yellow when freshly cut, becoming grayish-white on exposure. It has straight grain but frequently irregular with fairly fine and uniform texture. It has a density of 780 kg/m^3 when seasoned. It has high strength properties with medium movement in service. It dries rapidly with little degrade but with slight end-splitting and distortion. It machines well with moderate blunting effect on cutting edges and very susceptible to fungal staining and longhorn beetles. Sapwood is liable to attack by powder-post beetles. Heartwood is perishable and moderately resistant to preservative treatment. *Celtis zenkeri* is a heavy timber having good strength properties except for wood bending properties. And it's useful for floorings.

Cola gigantea (Watapuo) belongs to the family of the Sterculiaceae. The tree grows to a height of 50 m and 5 m in girth with a long straight bole with high buttresses. Irvine (1961) indicated that the species has distribution from Cote de'Ivoire to Nigeria. With a thick bark, rather smooth, ashy grey, coming off in strips and slightly fissured, it exudes gum. The fleshy aril of the seed and the soft fruit-pulp hardening later are both edible. The light grey-brown heartwood is soft, porous, coarse-grained, perishable, and liable to blue stain discolouration. The sapwood is whitish yellow. It is sometimes used for domestic utensils, and makes good fuel (Irvine, 1961).

Petersianthus macrocarpus (Esia) is of the family of the Lecythidaceae. The tree grows to a height of 40 m or more and with a diameter of 2.5 m. It has a straight, clear, cylindrical bole but sometimes with basal thickening. It is native to West Africa. Its heartwood is reddish-brown, often of plain appearance. Sapwood is wide and pale in colour. It has interlocked grain with coarse texture. It has a density of 800 kg/m^3 at 12 % moisture content. The freshly cut timber has a powerful unpleasant odour, which does not persist after drying. It had high strength properties with large movement in service. It dries slowly with a very pronounced tendency to check and split. It machines well with moderate blunting effect on cutting edges. It has been reported by the British Building Research Establishment (BBRE) to be moderately resistant to termites in West Africa. The heartwood is durable and extremely resistant to preservative treatment but the sapwood is permeable. Although it is locally abundant in West Africa, and with a good stem form, it has not been extensively utilized mainly because of the degrade that occurs during drying. It may be suitable for heavy, rough construction work or railway sleepers.

Sterculia oblonga (Ohaa) belong tothe family oaight, clear and cylindrical bole up to 15 to 21 m with a diameter of 2.5 m. It is widely distributed in West Africa. It is creamy-white to light yellowish-brown. The numerous high rays give the timber a striking fleck figure on accurately quarter-sawn stock. The sapwood is pale in colour, not clearly distinct from the heartwood extending in width to about 100 to 200 mm. Its grain is interlocked with a coarse texture. It has a density from 690 to 830 kg/m^3. Freshly sawn timber has a strong, disagreeable odour but this does not persist after the timber has been thoroughly dried. It has good strength properties with medium movement in service. It dries slowly with a marked tendency for surface checking, end-splitting, cupping and shakes. It machines satisfactorily but with a fibrous finish. It has moderate wood bending properties. The heartwood is non-durable but extremely resistant to preservative treatment though sapwood is permeable. It is a timber with good strength properties, but difficult to season and lacking in natural durability.

Sterculia rhinopetela (Wawabima) belongs to the family of the Sterculiaceae. The tree has a height of about 40 m with narrow buttresses, extending up the bole to about 3 m. The bole is straight and cylindrical with length about 21 m. It is mostly found in West Africa. It is variable in colour from pale to deep reddish-brown. The sapwood has a straw-coloured with clearly demarcated heartwood. The grain is sometimes straight but usually interlocked with coarse texture. Its density varies widely from 530 to 1020 kg/m^3. It has high strength properties with large movement in service. It dries very slowly with severe cupping, appreciable checking and end-splitting. It machines satisfactory but with a fibrous finish on end grain. It has moderate wood bending properties. The sapwood is liable to attack by powder-post beetles but heartwood is moderately durable. The heartwood is extremely resistant to preservative treatment and the sapwood is moderately resistant. It is mainly used for plywood, veneer-rotary, exterior structures, flooring, fittings and joinery.

The features of the selected species discussed above have been summarized in Table 2.6 below:

Table 2.6: Summary of Features of the 10 Selected Species

Species		Average Density at (12-15)% MC (Kg/m^3)	Growth height (m)	Girth (m)	Bole* characteristics and length (m)	Prescribed felling diameter (cm)	Colour	
Botanical Name	Local Name						Heartwood	Sapwood
Albizia ferruginea	Awiemfosamina	700	30 - 45	3	Clear, up to 20m	90	Dark red-brown	Pale yellow
Amphimas pterocarpoides	Yaya	770	45- 50	3 – 4	Straight cylindrical up to 25m	90	Yellowish-brown	
Antiaris toxicaria	Kyenkyen	530	40 – 50	4.5	Cylindrical up to 21m	110	Light yellow-brown	White
Blighia sapida	Akye		25	3	Short, crooked up to 16m	90	Brown-orange	White
Canarium schweinfurthii	Bediwonua	530	48	3.5	Straight, cylindrical from 10 – 30m	110	Pale-pinkish brown	White
Celtis zenkeri	Esa-Kokoo	780	40	3	Straight up to 20m	70	Clear light yellow	Yellowish-white
Cola gigantea	Watapuo		50	5	Straight	90	Grey-brown	Whitish-yellow
Petersianthus macrocarpus	Esia	800	40	2.5	Straight, clear and cylindrical	70	Reddish-brown	Yellow-white
Sterculia oblonga	Ohaa	830	35	2.5	Clear cylindrical, fairly straight. 15m	70	Creamy white	Pale yellow
Sterculia rhinopetala	Wawabima	530 - 1020	40	4	Straight cylindrical up to 21m	70	Dark red-brown	Pale

* Bole is the trunk of a tree which is usually converted to timber

2.10 NATURAL DURABILITY OF WOOD

The natural durability of wood is its ability to resist the attacks of foreign organisms including fungi, insects and marine borers (Panshin and De Zeeuw, 1980). A number of factors account for this such as the presence of extractives, and a moisture content outside of the natural moisture limits of the agent of destruction.

Wood consists of sapwood and heartwood. The sapwood is constituted by a part of living cells and conducts water and mineral salts. It is frequently attacked by insects due to the presence of soluble reserve substances in the parenchyma cells. These stored carbohydrates which serve as nutrient sources can be a factor promoting attack by fungi and insects. The sapwood undergoes a number of changes to be transformed progressively into the heartwood. Some of these changes consist in increase of acidity, formation of extractives such as tannins, which gives it a specific colour, and formation of gums, resins, tyloses and usually decrease in moisture content. These transformations and processes therefore make the heartwood often more durable than the sapwood and sometimes slightly harder. The presence of these extractives in sufficient amounts prevents or minimizes the severity of attack by destructive organisms because of their toxicity.

An indication of the effect of extractives on the durability of heartwood in some species is the early decomposition of the extractive-free sapwood from a piece of lumber. In some instances, the lower resistance of sapwood may be due to its greater permeability (Kollman and Cote, 1984). Moisture content clearly below fibre saturation point prevents or minimizes the attack by these organisms, particularly the decay fungi, because they need sufficient and easily available moisture to facilitate metabolism. Furthermore, the heartwood's lower rate of diffusion, the blocking of cell cavities by gums, resins, tyloses in the vessels and tylosoids in the resin canals adversely affect the balance between air and water necessary for the growth of fungi (Kollman and Cote, 1984).

The natural resistance of wood to deterioration that can be ascribed to reasons other than the toxicity of its impregnating substances, are the woody cell walls. These consist of highly complex, insoluble polymers of high molecular weight; these substances must be altered by enzymes produced by the attacking organisms into simpler products before they can be assimilated. Wood lignification creates a physical barrier to enzymatic attack on the polysaccharides. Therefore only those organisms that possess enzymes capable of destroying the lignin or at least of altering its protective association with the polysaccharides are capable of decaying wood. The structure of cellulose with crystalline and amorphous regions also restricts the action of depolymerizing enzymes. They can initially only affect the non-crystalline portions. Therefore, cellulose can sometimes provide some resistance to fungal and bacterial degradation.

Durability of wood also varies within and between trees. Although no known wood is entirely immune to the attack by degrading organisms, a number of wood species possess superior resistance. It must be kept in mind; however, that timber resistant to fungal attack may or may not be durable when subjected to attack by insects or marine borers. Furthermore, the durability of a given wood species may fluctuate between wide extremes (Kollman and Cote, 1984).

2.11 CONCLUSION

The review of related literature on the topic above gave a lot of insight into timber and granted the researcher a lot of knowledge on how to carry out the research activities. Helpful information on research works conducted on various timber species both in Ghana and abroad especially those of lesser known species aided in the selection of the ten (10) species for the research.

The review of research papers on strength properties of timber and previous laboratory tests conducted in similar topics guided in determining the type of tests that were conducted on the ten (10) species in order to achieve the objectives of the study. Test procedures, methods and analysis of results to be employed in chapter 3 of this thesis were based on approved test standards as used in researches reviewed above. Shortcomings and limitations of previous works also helped in the selection of methods used in this research.

It was observed that the use of small clear size specimen (eg. 20mm x 20mm x 300mm) gave higher mechanical properties than structural size specimen (eg. 50mm x 100mm x 3000mm). This is as a result of the presence of defects that weaken and subsequently cause variability in the strength properties of timber.

CHAPTER THREE

LABORATORY TESTS AND RESULTS

3.1 INTRODUCTION

Laboratory tests have been conducted to determine the mechanical properties of the selected timber species. The results obtained were analysed and used to grade species in terms of their strengths. The species include *Albizia ferruginea* (Awiemfosamina), ***Sterculia rhinopetala*** (Wawabima), ***Blighia** sapida* (Akye), *Canarium schweinfurthii* (Bediwonua), ***Petersianthus macrocarpus*** (Esia), *Sterculia oblonga* (Ohaa), *Cola gigantea* (Watapuo), ***Celtis zenkeri*** (Esa), ***Antiaris** toxicaria* (Kyenkyen) and ***Amphimas** pterocarpoides* (Lati). **Strength tests were conducted on the** ten (10) selected species. Four species with the overall **best strength properties were selected** for further bending test for a more detailed **analysis in order to establish their material** properties.

The tests were conducted at the Civil Engineering Laboratory, KNUST to **determine the following properties of** the 10 selected lesser known species:

- Bending strength
- Modulus of elasticity
- Compressive strength
- Shear Modulus
- Moisture content
- Density

The tensile strength test was conducted at the Bern University of Applied Sciences (BFH) Laboratory in Switzerland.

The code adopted for the tests was the European standard EN 408. The European Standard specifies test methods for determining the above properties of structural timber. In addition, the determination of dimensions, moisture content, and density of test pieces are specified.

3.1.1 Modulus of Elasticity (MOE)

Elasticity is the ability of a material to return to its previous shape after stress is released. In many materials, the relation between applied stress and the resulting strain is directly proportional (up to a certain limit), and a graph representing those two quantities is a straight line. The slope of this line is known as Young's Modulus or the Modulus of Elasticity (Timoshenko, 1976). The EN 408 specifies two methods or forms of determining modulus of elasticity; the local and global. The local modulus of elasticity is in principle based on pure bending deflection whilst the global modulus of elasticity is influenced by shear deflection (Solli, 1999). When measuring the global modulus of elasticity, the total deflection will be a combination of bending and shear deflection. The contributory effect of the shear deflection makes a fundamental difference between the global and local modulus of elasticity (Bostrom and Holmquist, 1999; Solli 1999).

The global modulus is not as sensitive to inaccurate measurements as the local modulus since the global deflection is about ten times the local. A measurement of the global modulus contains a higher number of possible sources of error. Because of the size of the total deflection the consequences of an error will normally be relatively small. If the intended use of MOE is to estimate the corresponding bending strength of a piece of timber, the local modulus is the unrivalled alternative of the two methods. This is of special importance concerning bending type strength grading machines.

3.1.1.1 Determination of local modulus of elasticity (EN 408)

The local modulus of elasticity is determined with a test piece having a minimum length of 19 times the depth of the section. The test piece is symmetrically loaded in bending at two points over a span of 18 times the depth as shown in Figure 3.1. If the test piece and equipment do not permit these conditions to be achieved exactly, the distance between the load points and the supports may be changed by an amount not greater than 1.5 times the piece depth, and the span and the test piece length may be changed by an amount not greater than three times the piece depth, while maintaining the symmetry of the test. The test piece is set up simply supported. Small steel plates of length not greater than one-half of the depth of the test piece are inserted between the piece and the loading heads to minimize local indentations. Lateral restraints provided at the supports to prevent buckling. The restraints are provided such that they permit the piece to deflect without significant frictional resistance. Load is applied at constant rate. The rate of movement of the loading piston is ensured to be not greater than $0.003h$ mm/s. The maximum load applied does not exceed $0.4\ F_{max}$

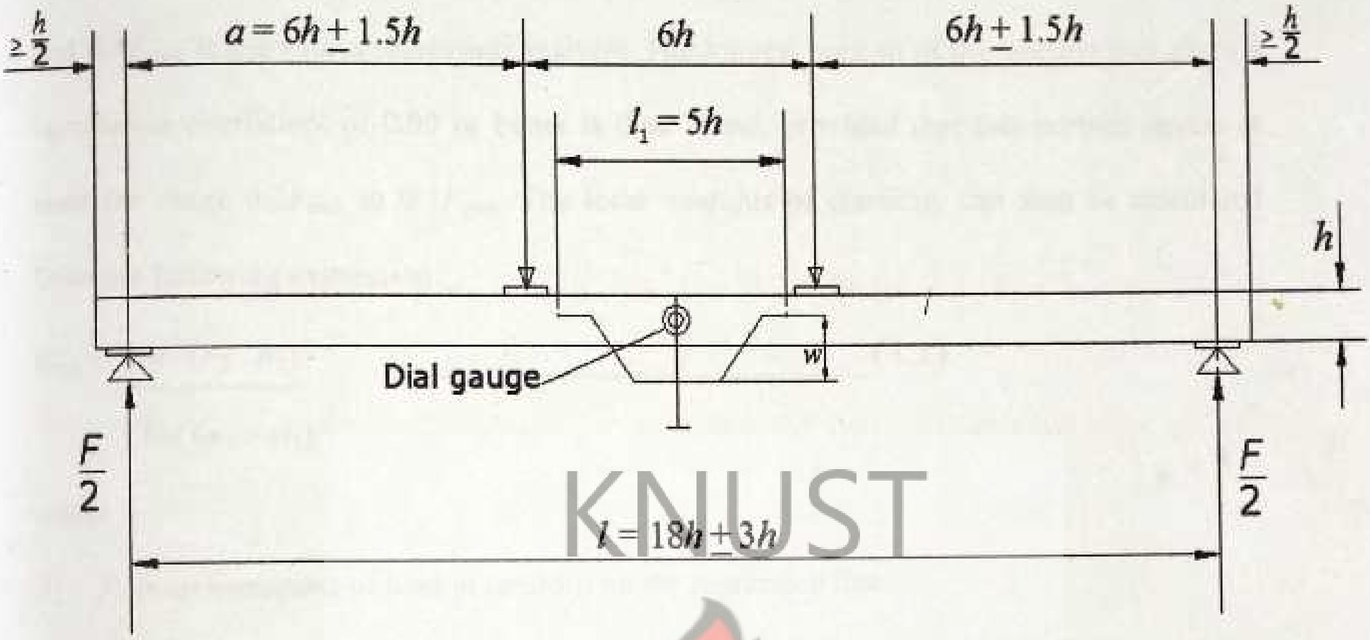


Figure 3.1a: Test arrangement for measuring local modulus of elasticity in bending

Figure 3.1b: Test arrangement for measuring local modulus of elasticity in bending

The deformation w is the average of measurements on both faces of the neutral axis, and is measured at the centre of a central gauge length of five times the depth of the section.

The data is used to plot a load/deformation graph. A section of the graph between $0.1F_{max}$ and $0.4F_{max}$ is used for a regression analysis. The longest portion of this section that gives a correlation coefficient of 0.99 or better is then found, provided that this portion covers at least the range $0.2F_{max}$ to $0.3F_{max}$. The local modulus of elasticity can then be calculated from the following expression:

$$E_{m,l} = \frac{a l_1^2 (F_2 - F_1)}{16I(w_2 - w_1)} \qquad (3.1)$$

where

$F_2 - F_1$ is an increment of load in newtons on the regression line

$w_2 - w_1$ is the increment of deformation in millimeters corresponding to $F_2 - F_1$ (Figure 3.2)

a is the distance between a loading position and the nearest support in a bending test;

l_1 is the gauge length for the determination of modulus of elasticity

I is the second moment of area in millimeter to the fourth power

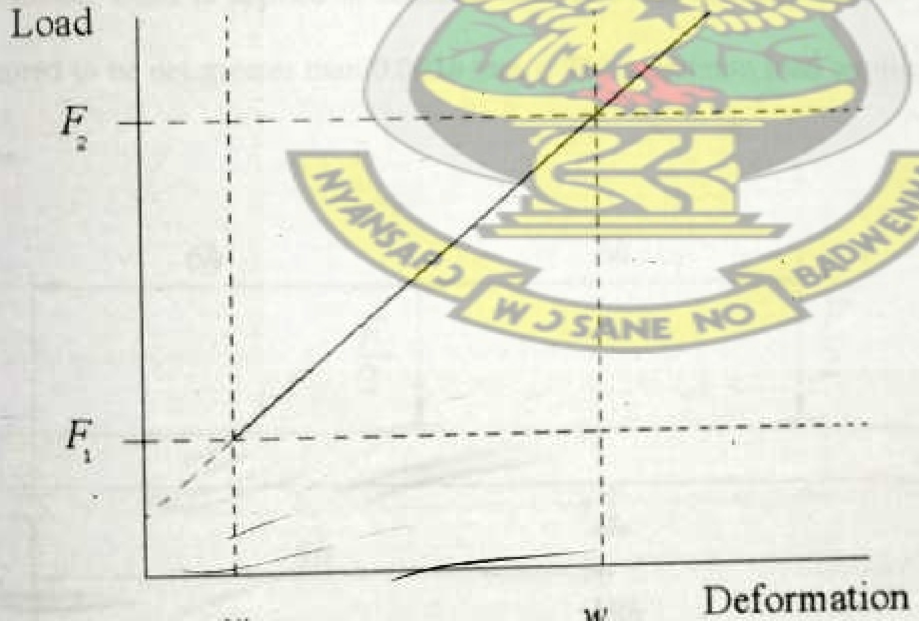


Figure 3.2: Load-deformation graph within the range of elastic deformation

3.1.1.2 Determination of global modulus of elasticity (EN 408)

The global modulus of elasticity is determined with a test piece having a minimum length of 19 times the depth of the section. The test piece is symmetrically loaded in bending at two points over a span of 18 times the depth as shown in Figure 3.3. If the test piece and equipment do not permit these conditions to be achieved exactly, the distance between the load points and the supports may be changed by an amount not greater than 1.5 times the piece depth, and the span and the test piece length may be changed by an amount not greater than three times the piece depth, while maintaining the symmetry of the test.

The test piece is set up simply supported. Small steel plates of length not greater than half of the depth of the test piece are inserted between the piece and the loading heads to minimize local indentations. Lateral restraints provided at the supports to prevent buckling. The restraints are provided such that they permit the piece to deflect without significant frictional resistance. Load is applied at constant rate. The rate of movement of the loading piston is ensured to be not greater than $0.003h$ mm/s. The maximum load applied does not exceed 0.4 F_{max}

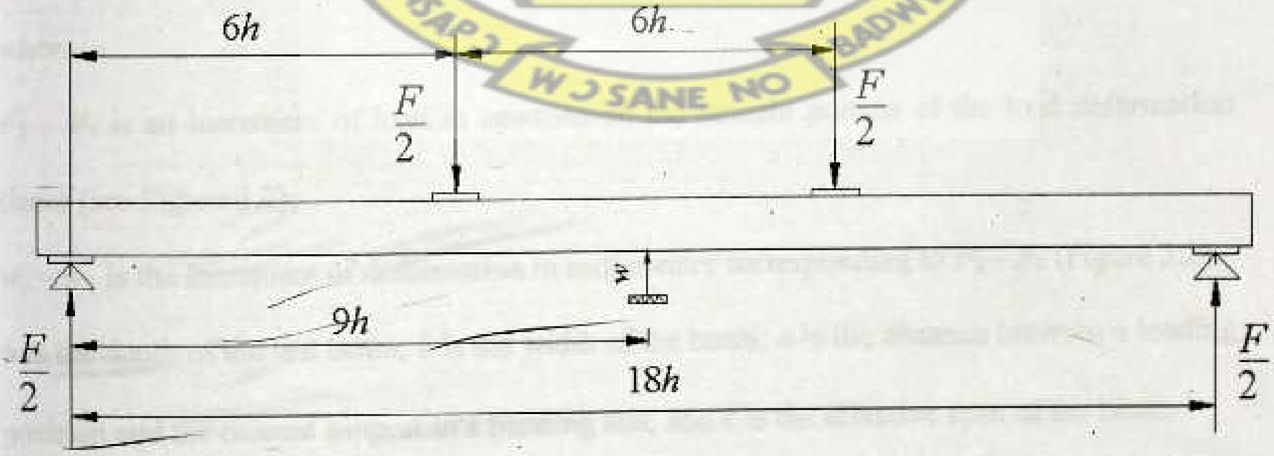


Figure 3.3a: Test arrangement for measuring global modulus of elasticity in bending

Figure 3.3b: Test arrangement for measuring global modulus of elasticity in bending

The deformation, w, is measured at the centre of the span. The global modulus of elasticity can then be calculated from the following expression:

$$E_{m,g} = \frac{\ell^3 (F_2 - F_1)}{bh^3 (w_2 - w_1)} \left[\left(\frac{3a}{4\ell} \right) - \left(\frac{a}{\ell} \right)^3 \right] \quad \text{——} (3.2)$$

where

$F_2 - F_1$ is an increment of load in newtons on the straight portion of the load deformation curve (see Figure 3.2);

$w_2 - w_1$ is the increment of deformation in millimeters corresponding to $F_2 - F_1$ (Figure 3.2)

h is the depth of the test beam; b is the width of the beam; a is the distance between a loading position and the nearest support in a bending test; and ℓ is the effective span of the beam.

3.1.2 Shear Modulus

The shear modulus is the elastic modulus used for the deformation which takes place when a force is applied parallel to one face of the object while the opposite face is held fixed by another equal force. It is a measure of the ability of a material to resist transverse deformations and is a valid index of elastic behaviour only for small deformations, after which the material is able to return to its original configuration.

When an object, like a piece of timber (Figure 3.4) of height **L** and cross section **A** experiences a force **F** parallel to one face, the sheared face will move a distance **Δx**. The shear stress is defined as the magnitude of the force per unit cross-sectional area of the face being sheared (**F/A**). The shear strain is defined as **Δx/L**.

The shear Modulus G is defined as the ratio of the stress to the strain.

$$G = \frac{\text{shear stress}}{\text{shear strain}} = \frac{F/A}{\Delta x/L} = \frac{FL}{A\Delta x} \qquad (3.3)$$

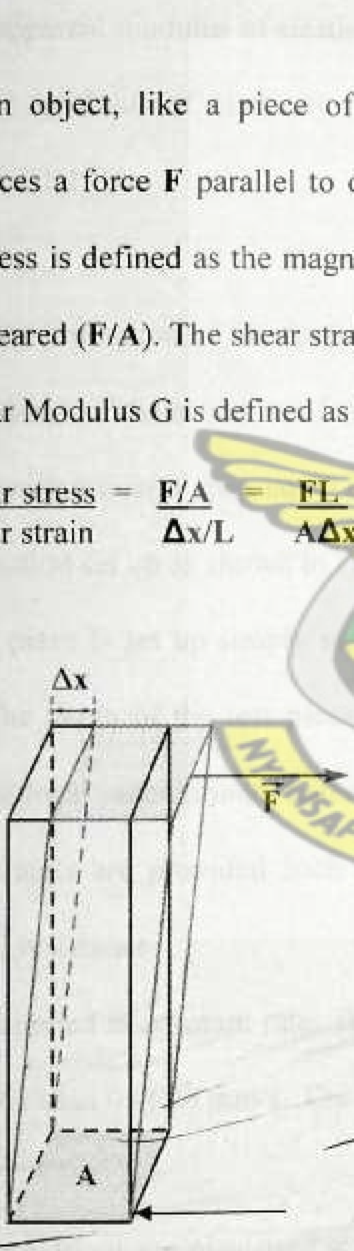


Figure 3.4: A piece of timber undergoing shear deformation

The bigger the shear modulus the more rigid the material, since for the same change in horizontal distance (strain) you will need a bigger force (stress). This is why the shear modulus is sometimes called the modulus of rigidity.

3.1.2.1 Determination of Shear Modulus – Single span Method (EN 408)

This method involves the determination of the local modulus of elasticity in bending, $E_{m,l}$ and the apparent modulus of elasticity, $E_{m,app}$ for the same length of test piece.

The local modulus of elasticity in bending is determined the same way as described in section 3.1.1.1

For the determination of the apparent modulus of elasticity, the same test piece used for the determination of the modulus of elasticity in bending is used. The test piece is loaded in the centre bending over a span equal to the gauge length used in the local modulus of elasticity determination set up as shown in Figure 3.1.

The test piece is set up simply supported. Small steel plates of length not greater than one-half of the depth of the test piece are inserted between the piece and the loading heads to minimize local indentations. Lateral restraints provided at the supports to prevent buckling. The restraints are provided such that they permit the piece to deflect without significant frictional resistance.

Load is applied at constant rate. The rate of movement of the loading piston is ensured to be not greater than $0.002h$ mm/s. The maximum load applied does not exceed $0.4\ F_{max}$

The deformations are measured at the centre of the span. The apparent modulus of elasticity $E_{m,app}$ can then be calculated from the following expression:

$$E_{m,app} = \frac{l_1^3(F_2 - F_1)}{48I\,(w_2 - w_1)} \qquad (3.4)$$

where

$F_2 - F_1$ is an increment of load in newtons on the straight line portion of the load deformation curve;

$w_2 - w_1$ is the increment of deformation in millimeters corresponding to $F_2 - F_1$ (Figure 3.2)

l_1 is the gauge length for the determination of modulus of elasticity

Calculation of Shear modulus, G

The shear modulus G is given by the equation:

$$G = \frac{k_G h^2}{l_1^3\left(\dfrac{1}{E_{m,app}} - \dfrac{1}{E_{m,\ell}}\right)} \qquad (3.5)$$

where

k_G = 1,2 for rectangular or square cross sections (Figure 3.5)

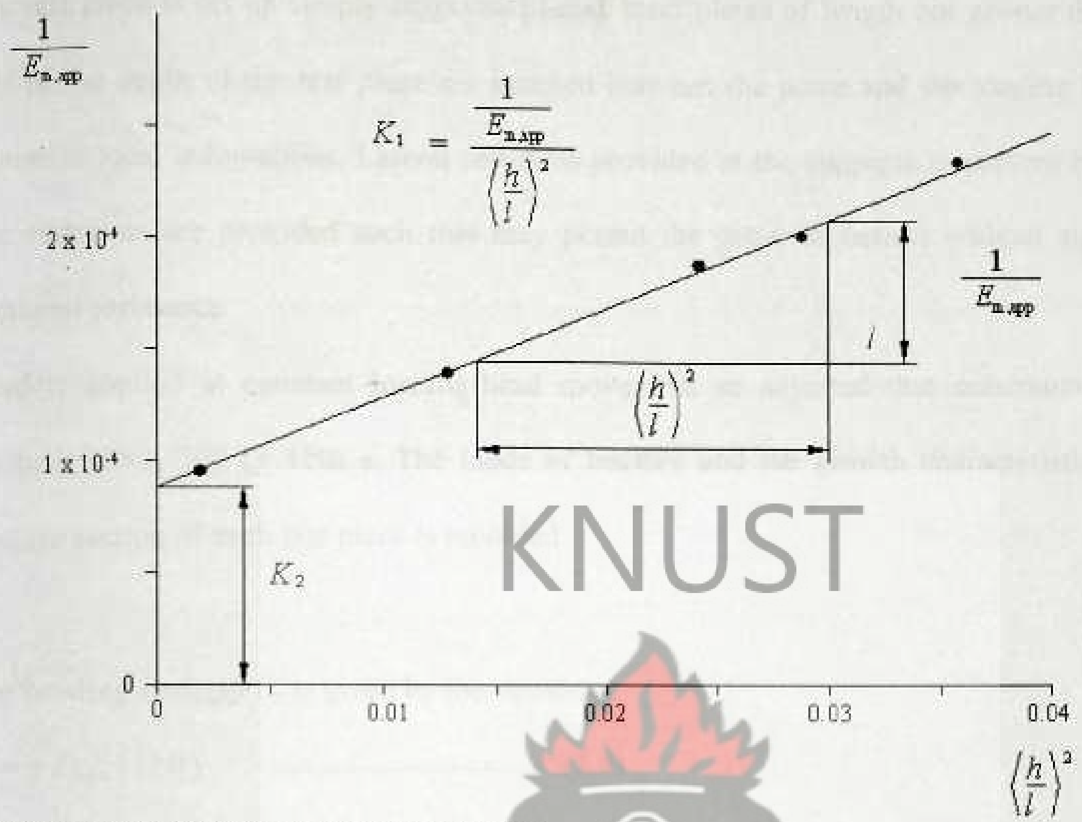


Figure 3.5: Determination of Shear Modulus

3.1.3 Determination of bending strength

The bending strength of a material refers to the material's ability to withstand an applied stress which subjects it to bending without failure.

The bending strength is determined with a test piece having a minimum length of 19 times the depth of the section. The test piece is symmetrically loaded in bending at two points over a span of 18 times the depth as shown in Figure 3.1. If the test piece and equipment do not permit these conditions to be achieved exactly, the distance between the load points and the supports may be changed by an amount not greater than 1.5 times the piece depth, and the span ~~and the~~ test piece length may be changed by an amount not greater than three times the piece depth, while maintaining the symmetry of the test.

The test piece is set up simply supported. Small steel plates of length not greater than one-half of the depth of the test piece are inserted between the piece and the loading heads to minimize local indentations. Lateral restraints provided at the supports to prevent buckling. The restraints are provided such that they permit the piece to deflect without significant frictional resistance.

Load is applied at constant loading-head movement so adjusted that maximum load is reached within 300 (± 120) s. The mode of fracture and the growth characteristics at the fracture section of each test piece is recorded.

The bending strength f_m is given by the equation;

$$f_m = a\,F_{max} / (2W) \qquad (3.6)$$

Where W is the section modulus

3.1.5 Determination of tensile strength parallel to the grain

Tensile stress is the stress state caused by an applied load that tends to elongate the material in the axis of the applied load, in other words the stress caused by pulling the material.

The test is conducted with a test piece having sufficient length to provide a test length clear of the testing machine's grips of at least nine times the larger cross-sectional dimension.

The test piece is loaded using gripping devices which will permit as far as possible the application of a tensile load without inducing bending. Load is applied at a constant loading-head movement so adjusted that maximum load is reached within 300 (± 120) s.

The tensile strength $f_{t,0}$ is given by the equation;

$$f_{t,0} = F_{max} / A \qquad (3.7)$$

Where F_{max} is the failure load, and A is the cross sectional area in square millimetres

3.1.6 Determination of compressive strength parallel to grain

Compressive stress (or compression) is the stress state caused by an applied load that acts to reduce the length of the material in the axis of the applied load, in other words the stress state caused by squeezing the material.

The test is conducted with a test piece having a length six times the smaller cross-sectional dimension. The end surfaces are prepared to ensure that they are plane and parallel to one another and perpendicular to the axis of the piece.

The test piece is loaded concentrically using spherically seated loading-heads, which permit the application of a compressive load without inducing bending. After an initial load has been applied, the loading-heads are locked to prevent angular movement. Load is applied at a constant loading-head movement so adjusted that maximum load is reached within 300 ($\pm$ 120) s. The compressive strength $f_{c,0}$ is given by the equation;

$$f_{c,0} = F_{max} / A \qquad (3.8)$$

Where F_{max} is the failure load, and A is the cross sectional area in square millimeters

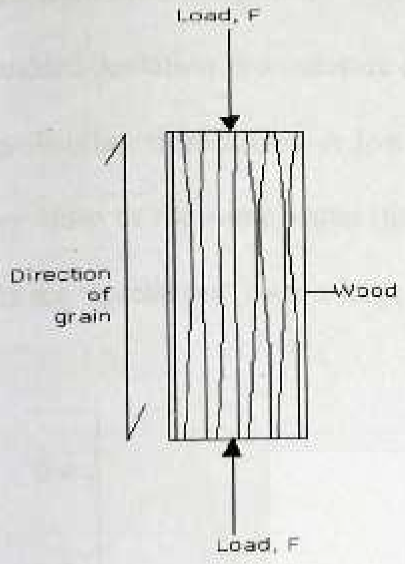


Figure 3.6: Piece of wood under compressive load, F, applied parallel to the direction of grain

3.2 STATISTICAL TERMS USED IN ANALYSIS

Statistics is a mathematical science pertaining to the collection, analysis, interpretation, and presentation of data (Lincoln, 1986). Statistical methods were used to describe or summarize the data collected. Descriptive statistics which is the statistical method used to summarize data, either numerically or graphically, to describe a sample was used in the analysis. The basic numerical descriptors include the mean, standard deviation and coefficient of variation.

3.2.1 The Mean (Average)

The mean is the arithmetic average of a set of values, or distribution; however, the mean is not necessarily the same as the middle value (median), or the most likely (mode). For a data set, the mean is the sum of the observations divided by the number of observations.

3.2.2 Standard Deviation

Standard deviation is a measure of the variability or dispersion of a population, a data set, or a probability distribution. A low standard deviation indicates that the data points tend to be very close to the same value (the mean), while a high standard deviation indicates that the data are 'spread out' over a large range of values.

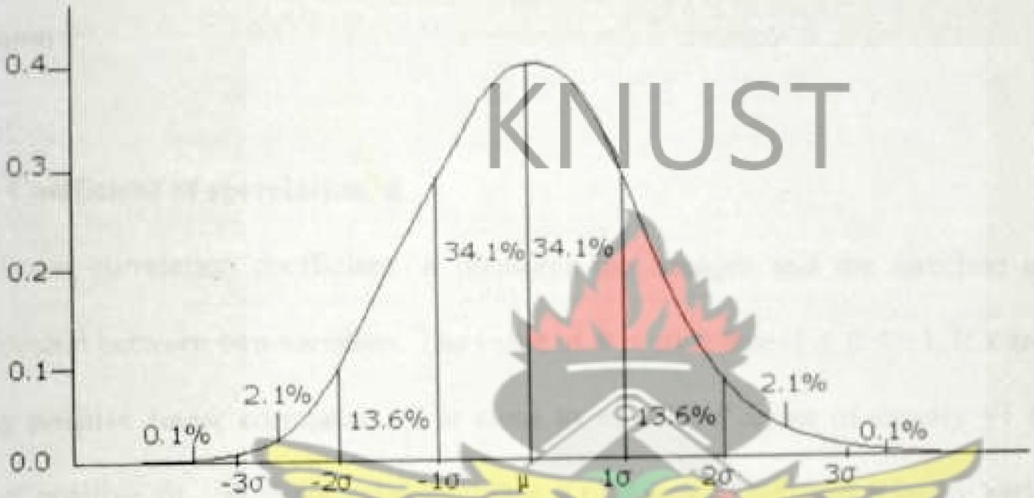


Figure 3.7: A plot of a normal distribution (or bell curve). Each colored band has a width of one standard deviation

3.2.3 Coefficient of variation

The coefficient of variation (CoV) is a normalized measure of dispersion of a probability distribution. It is defined as the ratio of the standard deviation, σ, to the mean, μ:

$$\text{CoV} = \frac{\sigma}{\mu} \qquad (3.9)$$

This is only defined for non-zero mean, and is most useful for variables that are always positive. The coefficient of variation is computed only for data measured on a ratio scale. As an example, if a group of temperatures are analyzed, the standard deviation does not depend on whether the Kelvin or Celsius scale is used. However, the mean temperature of the data set would be different in each scale and thus the coefficient of variation would be different. So the coefficient of variation does not have any meaning for data on an interval. The coefficient of variation is useful because the standard deviation of data must always be understood in the context of the mean of the data. The coefficient of variation is a dimensionless number, so when comparing between data sets with different units, or wildly different means, the coefficient of variation is used for comparison instead of the standard deviation.

3.2.4 Coefficient of correlation, R

The linear correlation coefficient, R measures the strength and the direction of a linear relationship between two variables. The value of R is such that $-1 \leq R \leq +1$. If x and y have a strong positive linear correlation, R is close to +1. An R value of exactly +1 indicates a perfect positive fit. Positive values indicate a relationship between x and y variables such that as values for x increase, values for y also increase. If x and y have a strong negative linear correlation, R is close to -1. An R value of exactly -1 indicates a perfect negative fit. Negative values indicate a relationship between x and y such that as values for x increase, values for y decrease. If there is no linear correlation or a weak linear correlation, R is close to 0. A value near zero means that there is a random, nonlinear relationship. A correlation greater than 0.8 is generally described as strong, whereas a correlation less than 0.5 is generally described as weak.

3.2.5 Coefficient of determination, R^2

In statistics, the coefficient of determination, R^2 is used in the context of statistical models whose main purpose is the prediction of future outcomes on the basis of other related information. It is the proportion of variability in a data set that is accounted for by the statistical model. It provides a measure of how well future outcomes are likely to be predicted by the model.

There are several different definitions of R^2 which are only sometimes equivalent. One class of such cases includes that of linear regression. In this case, R^2 is simply the square of the sample correlation coefficient, R, between the outcomes and their predicted values, or in the case of simple linear regression, between the outcome and the values being used for prediction. In such cases, the values vary from 0 to 1. An R^2 of 1.0 indicates that the regression line perfectly fits the data. For example, if $R = 0.922$, then $R^2 = 0.850$, which means that 85% of the total variation in y can be explained by the linear relationship between x and y (as described by the regression equation). The other 15% of the total variation in y remains unexplained.

3.2.6 Characteristic strength

The characteristic strength is the value below which the strength lies in only a small number of cases. It is determined from the test results using statistical principles, and is normally defined as the value below which not more than 5% of the test results fall. This is also known as the 5th percentile value and it is the value upon which design is based. The characteristic values are determined as follows:

$f_k = f_m - 1.64 \times S$ ——————— (3.10)

Where f_k = characteristic strength

f_m = mean strength

S = standard deviation

3.3 BENDING STRENGTH TEST

3.3.1 Test Material

Specimen from all the 10 different species were taken and tested in bending. The species with the least number of beams tested was *Alb. ferruginea* (Awiemfosamina) with only seven (7) beams tested. The species with the highest number of samples tested is *Celtis zenkeri* (Esa) with 16 beams tested. Logs of the species had an average diameter of 60mm especially for Sterculia *rhinopetala* and Blighia *sapida*. Species like Celtis zenkeri and Cola *gigantea* had average diameter of 120mm and 130mm respectively. Albizia *ferruginea* logs had an average diameter of 40mm which was relatively small and so many beams could not be obtained for the test.

The beams were all tested with the same dimensions. The dimensions of the beams initially were 50 x 150 mm in cross section with an effective span of 3000mm. Due to the experience of lateral torsional buckling during the testing as a result of the small transverse width of 50mm relative to the depth of 150mm, the depth of the beams were reduced to 120mm and the support system was also changed from a round metal support to a wooden frame which restrained the beam from falling off the support and also from buckling. The new dimensions thus were 50 x 120 mm in cross section with an effective span of 2500mm (Table 3.1). The span of the beams, 3000mm was also reduced to 2500mm in order to conform to the EN 408 specification of a minimum span of 18 times the depth of test beams (Figure 3.1a).

Table 3.1: Number of Beams Tested and their dimensions for the 10 species

Species		Number of Beams tested	Test Dimension (mm)
Botanical Name	Local Name		
Albizia ferruginea	Awiemfosamina	7	50 x 120 x 2500
Blighia sapida	Akye	11	50 x 120 x 2500
Canarium schweinfurthii	Bediwonua	12	50 x 120 x 2500
Celtis zenkeri	Esa	16	50 x 120 x 2500
Petersianthus macrocarpus	Esia	8	50 x 120 x 2500
Sterculia oblonga	Ohaa	10	50 x 120 x 2500
Sterculia rhinopetala	Wawabima	10	50 x 120 x 2500
Cola gigantean	Watapuo	10	50 x 120 x 2500
Antiaris toxicaria	Kyenkyen	11	50 x 120 x 2500
Amphimas pterocarpoides	Lati	12	50 x 120 x 2500

3.3.2 Apparatus

The apparatus for the testing included the loading machine (which comprises the hydraulic pump, loading piston, pressure gauge and hose), dial gauges, 2 metal pieces and fittings for measuring the relative deflection of the beam at the centroid, digital camera, weighing balance, moisture meter and an oven for determining moisture content.

3.3.3 Test Standard

The code adopted for the test was the European standard EN 408. The temperature within which the tests were done ranged from 28 – 31^{0}C. The relative humidity was about 80%.

3.3.4 Loading Criteria

Cyclic loading, which is the repeated application of loads, was used in the test procedure. This is because many structures, including bridges are usually subjected to repeated loading due to their usage. It has been found that structural components subjected to repeated loads may fail even though the associated stress levels are well below the yield strength (Bedford and Liechti, 2004). Basically, a small amount of damage is produced each time a repeated load is applied. Although the amount of damage done in each repetition, or cycle, is insufficient to cause failure, damage can accumulate and eventually result in failure. The resistance of material to such failures is therefore determined by subjecting them to cyclic loading in bending experiments.

3.3.5 Procedure

The test beam was fitted with two straight metal pieces to measure the relative deflection of the beam by means of dial gauges. The dial gauges were fitted on both sides of the beam to measure the local modulus of elasticity in bending. The specimen was then simply supported in the set-up (Figure 3.8). A dial gauge to measure the deflections for the global modulus of elasticity was placed at the bottom (mid span) of the beam i.e. the point of maximum deflection. Lateral restraints were provided by the supports to prevent lateral torsional buckling. The restraint permitted the specimen to deflect without significant frictional resistance.

The test beam was symmetrically loaded in bending at two points over an effective span of 2500mm as shown in Figure 3.8a. The load was applied by means of a hydraulic pump at a constant rate. Dial gauge readings were taken for both the local and the global modulus determination after every 2 KN load. The load was increased at multiples of 2 KN up to 6 or 8 KN (about 0.4 of maximum expected failure load) and then reduced back to zero. This is repeated and the beam is loaded to failure at the third time of loading.

The failure load and dial gauge readings were recorded for the computation of the bending strength and deflections.

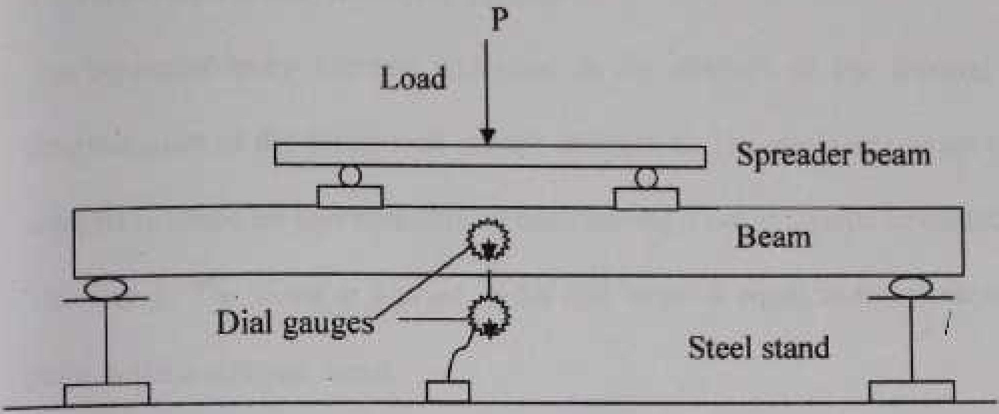


Figure 3.8a: Test set up

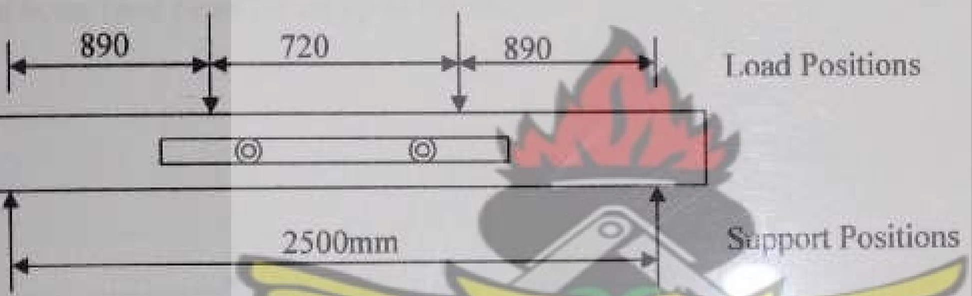


Figure 3.8b: Load and support positions

Figure 3.9: The test set up

3.4 THEORETICAL DEFLECTIONS

The conjugate-beam method was used in the analysis of the flexural behaviour and the determination of the theoretical central deflection. The conjugate-beam method of structural analysis is based on two theorems in determining a beam's slope or deflection.

Theorem 1: The slope at a point in the real beam is equal to the shear at the corresponding point in the conjugate beam.

Theorem 2: The displacement (deflection) of a point in the real beam is equal to the moment at the corresponding point in the conjugate beam.

The test beam (real beam) is set up as below:

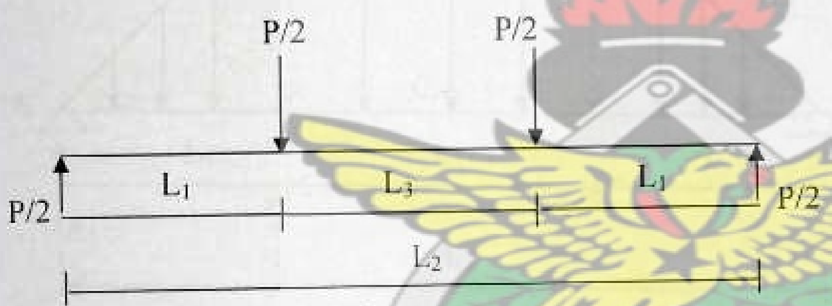


Figure 3.10: Test beam loaded

The bending moment, M diagram is drawn for the beam

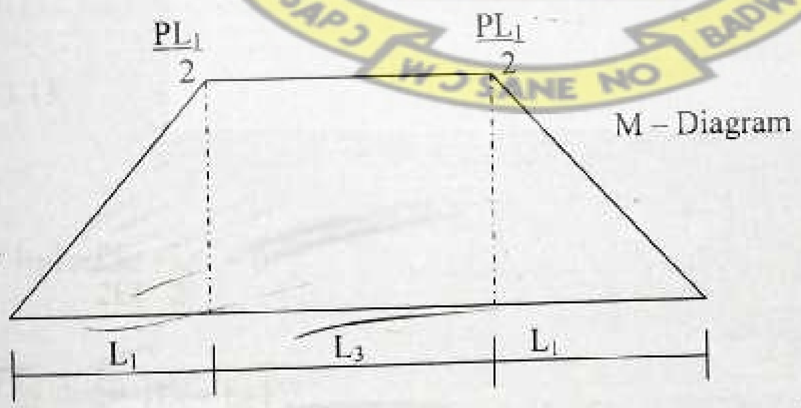


Figure 3.11: Bending Moment Diagram for the beam being loaded in Figure 3.9

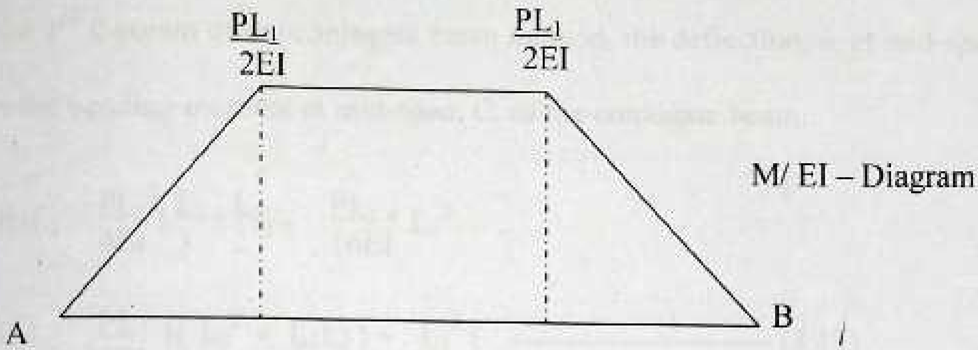


Figure 3.12: M/EI – Diagram for the beam

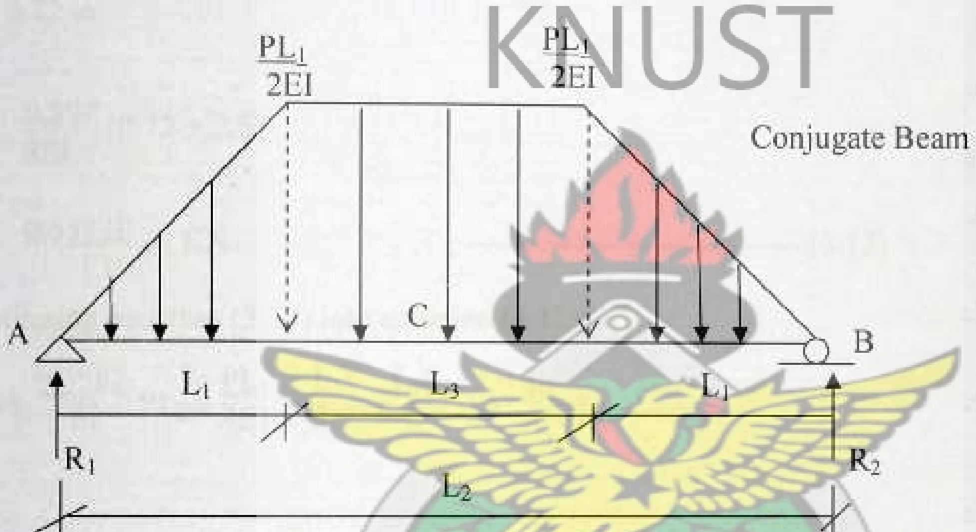


Figure 3.13: The conjugate beam loaded with the M/EI – Diagram

The beam can be analyzed by taking moments about point B,

From Figure 3.13,

$\sum MB = 0$

$$R_1L_2 - \tfrac{1}{2}[L_3 + L_2] * \frac{PL_1}{2EI} * \frac{L_2}{2} = 0$$

$$R_1 L_2 = {}^1/_2 \frac{PL_1}{2EI} * \frac{L_2}{2} [L_3 + L_2]$$

$$R_1 = \frac{PL_1}{8EI}[L_3 + L_2] \quad \text{------------------------} \quad (3.11)$$

From the 2nd theorem of the conjugate beam method, the deflection, δ, at mid-span of the real beam = the bending moment at mid-span, C, of the conjugate beam.

$$\delta = \tfrac{1}{2} R_1 L_2 - \frac{PL_1^{\,2}}{4EI}\left[\frac{L_1}{3} + \frac{L_3}{2}\right] - \frac{PL_1}{16EI} * L_3^{\,2}$$

$$\delta = \tfrac{1}{2} R_1 L_2 - \frac{PL_1}{4EI}\left[\left(\frac{L_1^{\,2}}{3} + \frac{L_1 L_3}{2}\right) + \frac{L_3^{\,2}}{4}\right] \text{ ------------------------- (3.12)}$$

From the test set up, L_1, L_2 and L_3 have the following dimensions:

$L_1 = 0.89$m
$L_2 = 2.5$ m
$L_3 = 0.72$ m

$$R_1 = \frac{0.89P}{8EI}[0.72 + 2.5]$$

$$R_1 = \frac{0.3582P}{EI} \quad \text{KN} \text{ --------------------------------------- (3.13)}$$

Substituting equation (3.13) into equation (3.12)

$$\delta = \tfrac{1}{2}\,\frac{0.3582}{EI}\,PL_2 - \frac{PL_1}{4EI}\left[\left(\frac{L_1^{\,2}}{3} + \frac{L_1 L_3}{2}\right) + \frac{L_3^{\,2}}{4}\right]$$

$$\delta = \tfrac{1}{2}\,\frac{0.3582 \text{ X } 2.5P}{2\,EI} - \frac{0.89P}{4EI}\left[\left(\frac{0.89^2}{3} + \frac{0.89 \times 72}{2}\right) + \frac{0.72^2}{4}\right]$$

$$\delta = \tfrac{1}{2}\,\frac{0.4478P}{EI} - \frac{0.2225P}{EI}[0.2640 + 0.3204 + 0.1296]$$

$$\delta = \frac{0.4478}{EI}P - \frac{0.2225}{EI}P$$

$$\delta = \frac{0.2889 \times 10^9}{EI}P \text{ (mm)}$$

$$\rightarrow \quad P = \frac{1}{0.2889 \times 10^9} EI\delta$$

$$\mathbf{P = 3.4613\,EI\delta} \text{ --------------------------------- (3.14)}$$

Where E is the local modulus of elasticity

3.5 RESULTS AND INTERPRETATION

3.5.1 Results

The results of the bending tests have been determined and the table that follows gives the summary of the results. The details of the laboratory readings and the calculated modulus of elasticity (both local and global) and the bending strength of each of the beams can be found in Appendix A.

There are also the results of other material properties of the beams. These include the density and moisture content of the beams.

3.5.2 Discussion of typical load-deflection curve

Figure 3.14 shows the load-deflection behaviour of *Albizia ferruginea* (Awiemfosamina) – AF10 (Figure 3.14). The beam is loaded initially from 0 to 6KN at intervals of 2KN. "L1" denotes the first loading (load-deformation) path and "U1" denotes the path taken (on the load deformation curve) as the beam is unloaded at the same 2KN rate from 6KN to 0. It was observed that the first unloading "U1" does not follow the original loading path "L1" i.e. even though the load on the beam had been completely removed, the beam does not return to its original horizontal position thereby leaving a permanent deformation of 0.08mm in the case of AF10 (local modulus). The beam is loaded again from 0 to 6KN at the rate of 2KN for a second time to study the behaviour of the beams under repetitive loads which is the case in the service of bridge. The behaviour indicates that the material is partially elastic with permanent strain since the beams returns partially to its original shape during unloading. "L2" denotes the path taken in the 2nd loading. The beam is then unloaded at the same rate from 6KN to 0. The unloading path is denoted by "U2" and this time the beam returns to the position before the 2nd loading (i.e. 0.08mm deflection). The beam is then loaded to failure in a third time of loading denoted by the path "L3". Load-deflection curves of the other species have been presented at Appendix C, they show similar behaviour as described above and shown in Figure 3.14a.

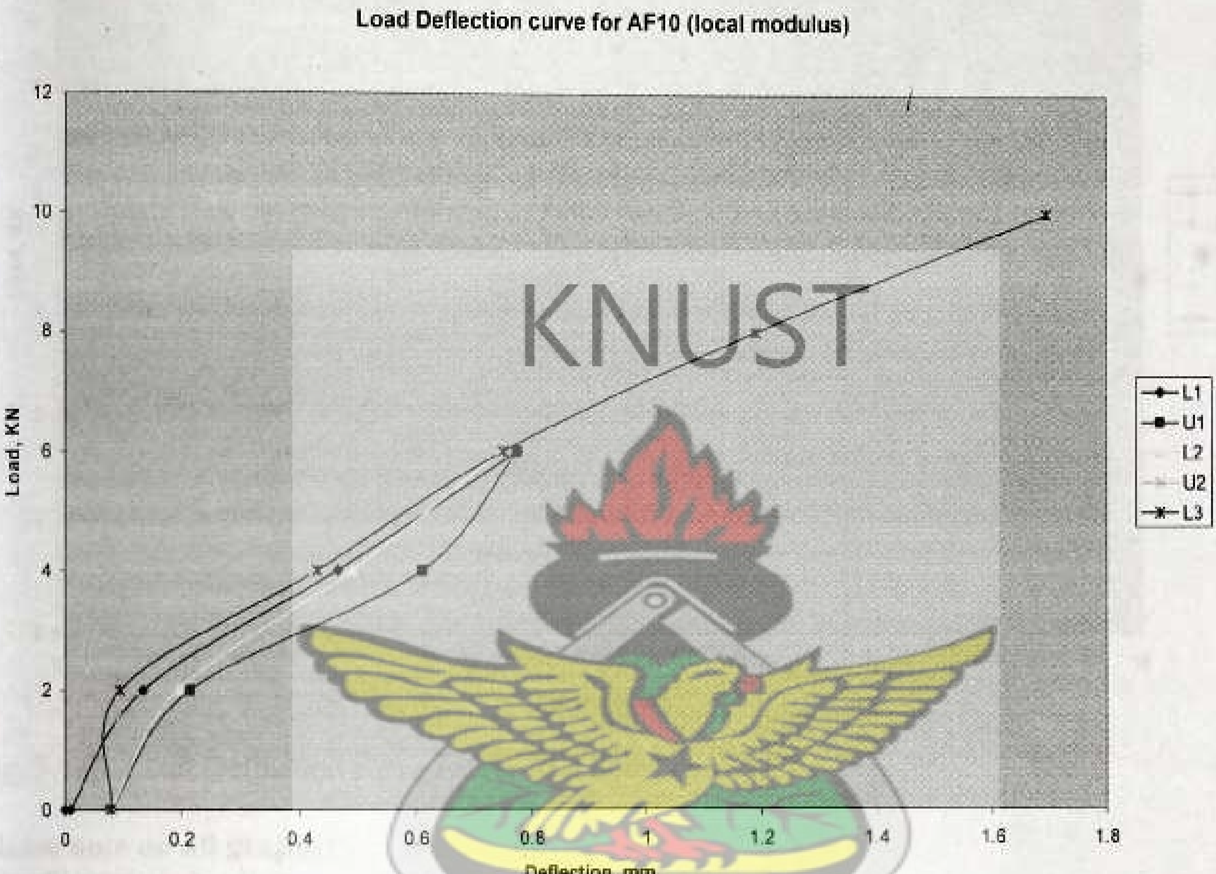


Fig. 3.14a: Load Deflection curve for AF10 (Local modulus)

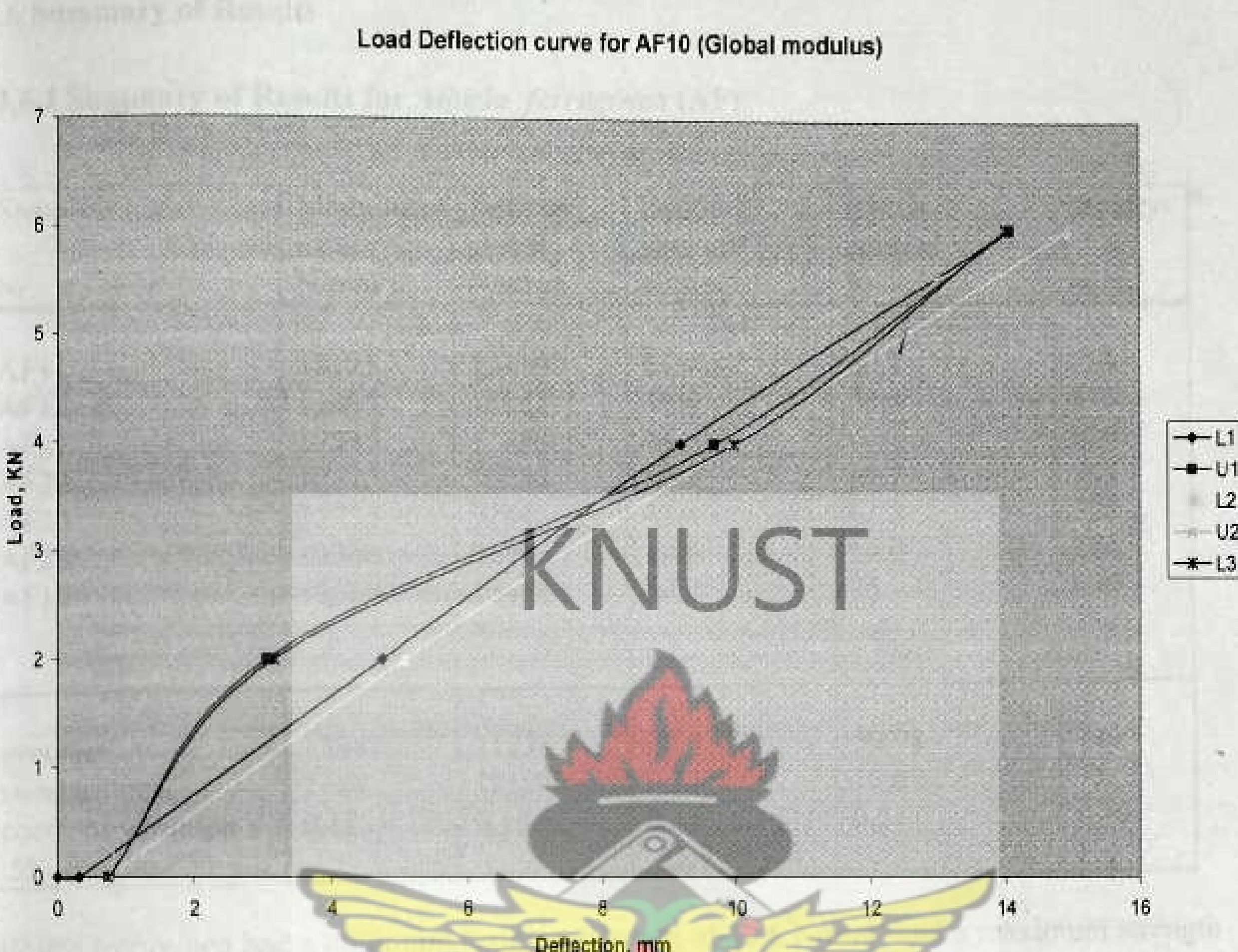


Fig. 3.14b: Load Deflection curve for AF10 (Global modulus)

Please note on all graphs:
L1 – First cycle loading
U1 – First cycle unloading
L2 – Second cycle loading
U2 – Second cycle unloading
L3 – Loading to failure

3.6 Summary of Results

3.6.1 Summary of Results for *Albizia ferruginea* (AF)

Sample Nr.	Modulus of elasticity local N/mm²	global N/mm²	Bending strength N/mm²	moisture content %	density kg/m³
AF11	18125	12419	61.9	40.7	729
AF12	11403	13082	40.6	58.8	870
AF21	13930	11350	44		629
AF22	12325	10014	54.2	55.7	762
AF20	12235	9144	47.4	61.5	728
AF23	12346	9191	40.6	54.2	774
AF10	16566	13468	60.9	35	690
average	13847	11238	49.9	50.98	740
standard dev.	2544	1819	9.11	10.63	75
coeff. of variation	0.18	0.16	0.18	0.21	0.10
5%-fractile			31.7		

Albizia ferruginea had a minimum bending strength of 40.6 N/mm² and a maximum strength value of 61.9 N/mm² with an average of 49.9 N/mm². It also has a 5th percentile bending strength of 31.7 N/mm². This is due to the small number of samples tested and the difference in the range of values obtained. It has an average density of 740 Kg/m³ and an average local and global modulus of elasticity of 13847 N/mm² and 11238 N/mm² respectively. These values compare very well with EN 338 strength class D30 and so Albizia ferruginea is predicted to belong to EN 338 strength class D30.

3.6.1.1 Mode of Failure

All the beams failed in tension with a break in the centre of the beams. The breaks were usually small cracks or in some cases small splits in the centre. In the case of AF23 there was a big split at the tension zone.

The load-deformation curve shows a permanent deformation of 0.08mm and 0.8mm in AF10 (local modulus) and AF10 (global modulus) respectively.

Figure 3.15: AF23 after failure

3.6.2 Summary of Results for *Amphimas pterocarpoides*(AP)

Sample Nr.	Modulus of elasticity local N/mm^2	Modulus of elasticity global N/mm^2	Bending strength N/mm^2	moisture content %	density kg/m^3
AP24	19248	15887	69.3	-	762
AP110	16339	14468	77	7	771
AP19	16096	13185	68.1	-	731
AP11	18079	15279	69.3	-	780
AP17	11342	13586	45.4	15.1	744
AP12	11273	13796	61.6	17.6	829
AP16	15194	13419	68.1	-	758
AP14	15470	14394	71.5	17.9	800
AP18	16465	14500	64.4	15.7	775
AP111	17692	15383	58.4	15.4	813
AP13	14870	13141	64.4	16	762
AP15	15075	13607	45.4	17.1	744
average	15595	14220	63.6	16.4	772
standard dev.	2397	917	9.73	1.12	29
coeff. of variation	0.15	0.06	0.15	0.07	0.04
5%-fractile			46.1		

Amphimas pterocarpoides obtained maximum bending strength of 77 N/mm^2 and a minimum bending strength of 45.4 N/mm^2 with an average of 63.6 N/mm^2. It however obtained a 5th percentile bending strength of 46.1 N/mm^2. It has an average density of 772 Kg/m^3 at moisture content of 16%. It compares well with EN 338 strength class D40. It is therefore predicted to belong to EN 338 strength class D40.

3.6.2.1 Mode of Failure

The beams failed in tension. The load-deflection curve shows an elastic deformation with permanent deformation of 0.07mm in AP16 (local modulus) and 0.9mm in AP16 (global modulus).

3.6.3 Summary of Results for *Antiaris toxicaria* (AT)

Sample Nr.	Modulus of elasticity local N/mm^2	Modulus of elasticity global N/mm^2	Bending strength N/mm^2	moisture content %	density kg/m^3
AT23	10954	9836	38.5	-	430
AT17	10116	9027	38.5	-	401
AT27	8550	9786	38.5	-	430
AT26	8680	8663	36.5	16	452
AT21	10428	9249	44.5	16.9	457
AT24	11540	8834	42.9	15.2	444
AT12	9172	8080	43.6	-	421
AT19	9604	8286	42.9	-	419
AT22	10199	9439	36.5	16.8	455
AT18	7817	6811	31.3	15.9	442
AT14	9370	9088	29.2	15.8	439
average	9675	8827	38.4	16.1	436
standard dev.	1105	868	4.97	0.64	17
coeff. of variation	0.11	0.10	0.13	0.04	0.04
5%-fractile			29.0		

Antiaris toxicaria had a maximum bending strength of 44.5 N/mm^2 and a minimum bending strength of 29.2 N/mm^2 with an average of 38.4 N/mm^2. It however obtained a 5th percentile bending strength of 29.0 N/mm^2. It has an average density of 436 Kg/m^3 at moisture content of 16%. It is the species with the lowest density and compares well with EN 338 softwoods of strength class C30 due to its density. Its bending strength does not measure up to the minimum bending strength of 30 N/mm^2 for hardwoods. It is therefore predicted to belong to EN 338 strength class C30.

3.6.3.1 Mode of Failure

All the beams failed in tension with some of them breaking completely into two at the centre. From the load-deflection curve for AT12 (global modulus), a permanent deformation of 1mm was recorded at the initial loading and an elastic deformation occurred upon subsequent loading. The AT12 (local modulus) curve showed an elastic deformation with a permanent deformation of 0.23mm.

Figure 3.16: Failure of AT21

3.6.4 Summary of Results for *Blighia sapida* (BS)

Sample Nr.	Modulus of elasticity local N/mm^2	global N/mm^2	Bending strength N/mm^2	moisture content %	density kg/m^3
BS12	13549	11467	74.5	27.8	870
BS24	14323	12963	67.7	21.2	896
BS11	12530	12520	47.4	26.3	869
BS21	10788	11564	62.1	33.1	887
BS22	21081	13205	67.7	33.8	992
BS23	11530	12387	67.7	49.2	909
BS25	9888	10817	42.2	26.5	791
BS26	10655	10103	50.1	18.1	851
BS27	9834	11494	64.4	39.4	927
BS28		12100	63.3	20	954
BS29		14237	67.9	18.5	946
average	12686	12078	61.4	28.5	899
standard dev.	3517	1161	10.2	9.7	55
coeff. of variation	0.28	0.10	0.17	0.34	0.06
5%-fractile			42.0		

Blighia sapida had a maximum bending strength of 74.5 N/mm^2 and a minimum bending strength of 42.2 N/mm^2 with an average of 61.4 N/mm^2. It however obtained a 5th percentile bending strength of 42.0 N/mm^2. It has an average density of 899 Kg/m^3 at moisture content of 28%. It compares well with EN 338 strength class D40. It is therefore predicted to belong to EN 338 strength class D40.

3.6.4.1 Mode of Failure

This species had defects like knots in them. The mode of failure was greatly affected by the knot positions. The beams failed by breaking or cracking at the position of the knots. Most of the beams failed in tension with cracks occurring along the grains of the beams.

Figure 3.17: BS11 after failure

3.6.5 Summary of Results for *Cola gigantea* (CG)

Sample Nr.	Modulus of elasticity local N/mm²	global N/mm²	Bending strength N/mm²	moisture content %	density kg/m³
CG13	10175	10426	42.9	16.8	698
CG14	9209	8897	56.3	17.6	669
CG15	8593	7916	29.7		676
CG21	9724	9600	50.1	21.5	584
CG23	7425	8431	35.8	16.8	800
CG26	11264	9366	50.1	17.3	775
CG22	7344	8364	37.1	15.5	679
CG12	11176	10937	50.9	18.6	575
CG11	11183	10512	50.9	33.1	613
CG16	16093	12337	52.8	17.2	644
average	10219	9679	45.7	19.4	671
standard dev.	2526	1375	8.8	5.4	74
coeff. of variation	0.25	0.14	0.19	0.28	0.11
5%-fractile			29.0		

Cola gigantea had a maximum bending strength of 56.3 N/mm² and a minimum bending strength of 29.7 N/mm² with an average of 45.7 N/mm². It however obtained a 5th percentile bending strength of 29.0 N/mm². It has an average density of 671 Kg/m³ at moisture content of 19%. It has a relatively low density and compares well with EN 338 softwoods of strength class C30 due to its density. Its bending strength also does not measure up to the minimum bending strength of 30 N/mm² for hardwoods. It is therefore predicted to belong to EN 338 strength class C30.

3.6.5.1 Mode of Failure

The beams failed generally in tension during bending. For some of the beams, there was a split away of some section of the beam within the tension zone. For CG26, there was a bearing failure in the compression zone and the beam continued to split or deform under sustained loading after failure. Compression failure occurred in CG12.

Figure 3.18: CG26 at failure

3.6.6 Summary of Results for *Canarium schweinfurthii* (CS)

Sample Nr.	Modulus of elasticity local N/mm²	Modulus of elasticity global N/mm²	Bending strength N/mm²	moisture content %	density kg/m³
CS12	8777	8269	40.7		448
CS14	11004	9637	41.5	38.2	529
CS16	10340	9506	41.7	32.7	534
CS11	11468	9514	41.2		505
CS13	9839	11685	46.1	27.5	451
CS15	13248	9569	47.5	29.7	510
CS110	10023	9134	46.1	27.1	495
CS111	8562	8515	46.3	26.1	460
CS10	9902	8561	40.4	37.2	465
CS17	10021	9257	51.2	31.7	496
CS19	10287	8989	40.9	19	458
CS116				35.2	505
average	10316	9331	43.9	30.4	488
standard dev.	1281	911	3.6	5.8	30
coeff. of variation	0.12	0.10	0.08	0.19	0.06
5%-fractile			37.4		

Canarium schweinfurthii had a maximum bending strength of 51.2 N/mm² and a minimum bending strength of 40.4 N/mm² with an average of 43.9 N/mm². It however obtained a 5th percentile bending strength of 37.4 N/mm². It has an average density of 488 Kg/m³ at moisture content of 30%. Its density is low and compares well with EN 338 softwoods of strength class C35. It is therefore predicted to belong to EN 338 strength class C35.

3.6.6.1 Mode of Failure

The beams were very ductile. They generally buckled during loading before breaking in the tension zone. Some of the beams e.g. CS12 and CS11 could not break in tension but rather deflected excessively with buckling and suddenly flew off the supports. The curve shows an elastic deformation with permanent deformations of 0.18mm and 1mm for CS17 (Global modulus and CS17 (local modulus) respectively.

Figure 3.19: CS110 at failure

3.6.7 Summary of Results for *Celtis zenkeri* (CZ)

Sample Nr.	Modulus of elasticity local N/mm^2	Modulus of elasticity global N/mm^2	Bending strength N/mm^2	moisture content %	density kg/m^3
CZ11	17455	15537	75.9		828
CZ12	18043	13730	69	14.9	815
CZ13	17740	15282	41.4	24.6	840
CZ14	17512	14032	42.2	25.3	816
CZ21	23716	15849	42.2	40.2	859
CZ22	17180	13811	41.4	35.7	815
CZ15	17682	15961	71.8	15.7	860
CZ16	18924	14515	72.9	29.5	826
CZ17	19550	14165	57.2	34.5	813
CZ18	14949	13172	78.7	25	808
CZ19	15068	14726	71.5	39.1	884
CZ110	17135	15314	78.7	22.9	889
CZ111	18358	14484	77.4	38.5	841
CZ112	16515	12488	77.4	30.3	779
CZ113	13825	12780	77.4	32.5	804
CZ114	15092	12524	77.4	26.6	791
average	17422	14273	65.8	29.0	829
standard dev.	2292	1143	15.2	7.9	31
coeff. of variation	0.13	0.08	0.23	0.27	0.04
5%-fractile			39.9		

Celtis zenkeri had a maximum bending strength of 78.7 N/mm^2 and a minimum bending strength of 41.4 N/mm^2 with an average of 65.8 N/mm^2. It however obtained a 5^{th} percentile bending strength of 39.9 N/mm^2. The low 5^{th} percentile bending strength relative to the average value is due to the wide range of values between the minimum and maximum bending strength (standard deviation). It has an average density of 829 Kg/m^3 at moisture content of 29%. Its density and bending strength compares well with EN 388 strength class D35. It is therefore predicted to belong to EN 338 strength class D35.

3.6.7.1 Mode of Failure

Most of the beams failed in tension with splitting along the grain of the beam after breaking. A few exhibited failure in bearing. CZ414 and CZ112 for example failed in tension with the beam splitting along the grains of the beam from the tension zone. The splitting usually occurred with a loud noise. There was also a sign of compression failure in CZ15.

The load-deflection curves show an elastic behaviour of the specimen. Permanent deformations of 0.1mm in CZ16 (local modulus) and 1.2mm in CZ16 (global modulus) were recorded.

Figure 3.20: CZ112 showing large splits at failure

3.6.8 Summary of Results for *Petersianthus macrocarpus* (PM)

Sample Nr.	Modulus of elasticity local N/mm²	global N/mm²	Bending stress N/mm²	moisture content %	density kg/m³
PM25	10164	12843	74.4	18.4	784
PM24	14016	12075	71.8	20.5	810
PM11	11666	9663	58.6	25.7	885
PM14	10310	8118	65.9	37.2	972
PM22	10458	9549	50.1	27.2	787
PM23	13340	10923	64.4	35	876
PM12	16071	9949	51.3	35	883
PM13	10146	10831	51	31	872
Average	12021	10494	60.9	28.8	859
standard dev.	2223	1504	9.6	6.98	63
coeff. of variation	0.18	0.14	0.16	0.24	0.07
5%-fractile			41.7		

Petersianthus macrocarpus had a maximum bending strength of 74.4 N/mm² and a minimum bending strength of 50.1 N/mm² with an average of 60.9 N/mm². It however obtained a 5th percentile bending strength of 41.7 N/mm². The low 5th percentile bending strength relative to the average value is due to the wide range of values between the minimum and maximum bending strengths (standard deviation). It has an average density of 829 Kg/m³ at moisture content of 28%. Its density and bending strength compares well with EN 388 strength class D40. It is therefore predicted to belong to EN 338 strength class D40.

3.6.8.1 Mode of Failure

Some of the beams looked weak in compression during loading and so failed in compression. PM22 failed at a knot in the tension zone. There were irregularities or defects like knots in some of the beams which affected their mode of failure depending on the position of the knots. The load-deflection curves show an elastic deformation in the specimen with

3.6.10: Summary of Results for *Sterculia rhinopetala* (SR)

Sample Nr.	Modulus of elasticity local N/mm^2	Modulus of elasticity global N/mm^2	Bending strength N/mm^2	moisture content %	density kg/m^3
SR12	18082	13614	79.6	38.8	956
SR11	16194	13772	72.4	47	992
SR13	13179	11973	79.6	52.6	1024
SR14	11282	10797	57.9	45.8	816
SR15	14582	14554	101.3	53.9	1111
SR16	21151	12928	78.1	38.8	953
SR17	15767	13702	91.5	32.6	1030
SR18	16509	12608	70.3	50.7	1017
SR19	17009	15462	93	56	1079
SR110		14412	93	19.8	1092
Average	15973	13382	81.7	43.6	1007
standard dev.	2839	1356	13.1	11.3	86
coeff. Of variation	0.18	0.10	0.16	0.26	0.09
5%-fractile			56.8		

Sterculia rhinopetala had a maximum bending strength of 101.3 N/mm^2 and a minimum bending strength of 57.9 N/mm^2 with an average of 81.0 N/mm^2. It however obtained a 5th percentile bending strength of 56.8 N/mm^2. The 5th percentile bending strength was lower than the minimum bending strength value due to the few number of beams tested, and the wide range of values between the minimum and maximum bending strengths (standard deviation). It has an average density of 1007 Kg/m^3 at a moisture content of 43%. Its density and bending strength compares very well with EN 388 strength class D50. It is therefore predicted to belong to EN 338 strength class D50.

3.6.10.1 Mode of Failure

The beams generally failed in compression. This particular species had a lot of moisture in them so it was observed that moisture (liquid) drained down from some of the beams at the permanent deformations of 0.1mm in PM23 (local modulus) and 1.5mm in PM23 (global modulus).

Figure 3.21: PM22 at failure

3.6.9: Summary of Results for *Sterculia oblonga* (SO)

Sample Nr.	Modulus of elasticity local N/mm^2	Modulus of elasticity global N/mm^2	failure stress N/mm^2	moisture content %	density kg/m^3
SO11	16370	14120	88	26.7	887
SO12	14216	10954	55.1	35.2	745
SO23		14377	60.9	42.7	851
SO24	15008	10374	67.7	22.2	737
SO21	19800	15556	74.5	35.9	907
SO22	14441	12750	74.5	32.4	875
SO25	12865	11574	70.3	39	795
SO26	21742	15113	78.7	16.7	813
SO13	16823	12859	65.7	38.8	814
SO14		12362	64.4	35.8	787
Average	16408	13004	70.0	32.5	821
standard dev.	3008	1756	9.4	8.20	58
coeff. of variation	0.18	0.14	0.13	0.25	0.07
5%-fractile			52.1		

Sterculia oblonga had a maximum bending strength of 88 N/mm^2 and a minimum bending strength of 55.1 N/mm^2 with an average of 70.0 N/mm^2. It however obtained a 5^{th} percentile bending strength of 52.1 N/mm^2. The low 5^{th} percentile bending strength relative to the average value is due to the number of beams tested and the wide range of values between the minimum and maximum bending strengths (standard deviation). It has an average density of 821 Kg/m^3 at moisture content of 32%. Its density and bending strength compares very well with EN 388 strength class D50. It is therefore predicted to belong to EN 338 strength class D50.

3.6.9.1 Mode of Failure

Most of the beams failed in tension. For SO12, bearing failure occurred at the point of application of the load before breaking in the tension zone. There was an excessive deflection in SO21. It could not break. It therefore failed through deflection. The load-deflection curve shows an elastic deformation for SO21. Permanent deformations of 0.09mm and 0.8mm were recorded for SO21 (local modulus) and SO21 (global modulus) respectively.

Figure 3.22: SO21 at failure

point of application of the loads suggesting bearing failure as shown in the picture. Load-deflection curve shows an elastic behaviour of specimen. Permanent deformations of 0.05mm and 0.9mm were recorded in SR17 (local modulus) and SR17 (global) respectively.

Figure 3.23: SR33 at failure with water dripping from the specimen

Table 3.2: Predominant failure modes of the 10 species

Species	Predominant failure mode
AF	Tension failure with cracks and splitting in centre
BS	Tension failure with cracks occurring along grains
CS	Tension failure with buckling of beams
CZ	Tension failure with splitting along grains. Splitting occurred with loud noise.
PM	Compression and tension failure
SO	Tension failure with excessive deflection in some beams
SR	Bearing failure and tension failure in some beams
CG	Tension failure with cracks and splitting
AT	Tension failure
AP	Tension failure with cracks and splitting in centre

Table 3.3a: Summary of Results of the ten (10) species

Species	Modulus of elasticity				Bending strength			Moisture Content	Density	Proposed strength class
	local		global							
	average N/mm^2	Std. Dev. N/mm^2	average N/mm^2	Std. Dev. N/mm^2	average N/mm^2	Std. Dev. N/mm^2	5%-fractile N/mm^2	average %	average kg/m^3	(EN338)
AF	13847	2544	11238	1819	49.9	9.1	31.7	51.0	740	D30
BS	12686	3517	12078	1161	61.4	10.2	42.0	28.5	899	D40
CS	10316	1281	9331	911	44.0	3.6	37.4	30.4	488	C35
CZ	17422	2292	14273	1143	65.8	15.2	39.9	29.0	829	D35
PM	12021	2223	10494	1504	60.9	9.6	41.7	28.8	859	D40
SO	16408	3008	13004	1756	70.0	9.4	52.1	32.5	821	D50
SR	15973	2839	13382	1356	81.7	13.1	56.8	43.6	1007	D50
CG	10219	2526	9679	1375	45.7	8.8	29.0	19.4	671	C30
AT	9675	1105	8827	868	38.4	5.0	29.0	16.1	436	C30
AP	15595	2397	14220	917	63.6	9.7	46.1	16.4	772	D40

NOTE:

AF = *Albizia ferruginea* (Awiemfosamina)
BS = *Blighia sapida* (Akye)
CS = *Canarium schweinfurthii* (Bediwonua)
CZ = *Celtis zenkeri* (Esa)
PM = *Petersianthus macrocarpus* (Esia)
SO = *Sterculia oblonga* (Ohaa)
SR = *Sterculia rhinopetala* (Wawabima)
CG = *Cola gigantea* (Watapuo)
AT = *Antiaris toxicaria* (Kyenkyen)
AP = *Amphimas pterocarpoides* (Yaya)

Std. Dev = Standard Deviation

Table 3.3b: Comparison of Results of the ten (10) species

Species	Modulus of elasticity				Failure Load, P_{ult} (KN)	Deflections		
	local		global			Experimental Deflection, δ_{ult} (mm)	Theoretical Deflection, δ'_{ult} (mm)	Permissible design deflection, mm (0.003*span) from BS 5268:1984
	average N/mm^2	Stand. Dev. N/mm^2	Average N/mm^2	Stand. Dev N/mm^2				
AF	13847	2544	11238	1819	14.7	1.44	3.7	7.5
BS	12686	3517	12078	1161	17.6	1.69	4	7.5
CS	10316	1281	9331	911	17.2	1.47	2.6	7.5
CZ	17422	2292	14273	1143	18.6	1.02	4.6	7.5
PM	12021	2223	10494	1504	17	2.65	5.2	7.5
SO	16408	3008	13004	1756	20.2	2.5	4.7	7.5
SR	15973	2839	13382	1356	22.8	1.57	5.8	7.5
CG	10219	2526	9679	1375	12.6	3.9	4.5	7.5
AT	9675	1105	8827	868	10.7	1.43	3.3	7.5
AP	15595	2397	14220	917	17	1.3	3.9	7.5

NOTE:

AF = *Albizia ferruginea* (Awiemfosamina)
BS = *Blighia sapida* (Akye)
CS = *Canarium schweinfurthii* (Bediwonua)
CZ = *Celtis zenkeri* (Esa)
PM = *Petersianthus macrocarpus* (Esia)
SO = *Sterculia oblonga* (Ohaa)
SR = *Sterculia rhinopetala* (Wawabima)
CG = *Cola gigantea* (Watapuo)
AT = *Antiaris toxicaria* (Kyenkyen)
AP = *Amphimas pterocarpoides* (Lati)

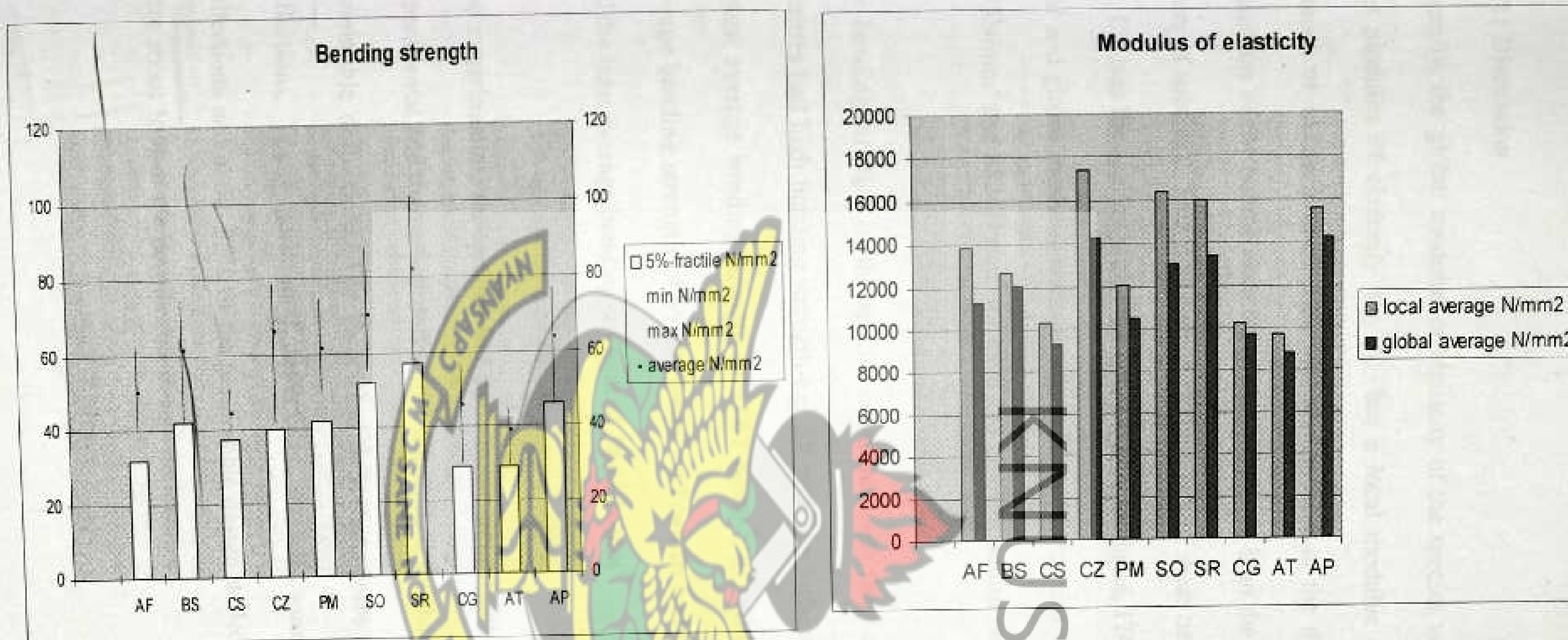


Figure 3.24: Comparison of results of the 10 species

3.6.11 Discussion

Generally, the global modulus of elasticity of the species were found to be lower than the local modulus of elasticity e.g. AF has a local modulus of 13847 N/mm^2 but a global modulus of 11238 N/mm^2. This was also evident in the deflection of beams. The global deflections of the beams were about 10 times more than the local deflections (see modes of failure of species). This is as a result of the effect of shear deformation in the global modulus test. CZ was the species with the highest elastic modulus, 17422N/mm^2 and 14273N/mm^2 for local and global modulus respectively. AT was the species with the least elastic modulus of 9675N/mm^2 and 8827N/mm^2 for local and global modulus respectively.

The bending strength of the species correlated with their densities. The species with high densities had high bending strength e.g. SR with the highest density of 1007 Kg/m^3 had the highest average bending strength of 81.67 N/mm^2, SO with a density of 821Kg/m^3 had an average bending strength of 69.98 N/mm^2. AT which had the lowest density of 435 Kg/m^3 had the least average bending strength of 38.45 N/mm^2.

The experimental, theoretical and permissible deflections have been compared. Both the experimental and theoretical deflections determined for the species were less than the design permissible deflections. The theoretical deflections were however twice the experimental deflections. The experimental deflections obtained were about 45% of the theoretical deflections and about 25% of the permissible design deflections. The theoretical deflections were about 56% of the permissible design deflections.

SR was found to be the species with the best strength properties with the bending strength and elastic modulus being paramount in rating the species. The rating of the results, durability characteristics and the availability of the species in the forest were used to select the best 4 species out of the 10 for further bending test.

3.7 FURTHER BENDING TEST

From the results above four species have been selected for further bending tests. The further bending test was conducted to study the behaviour of the species under unfavourable conditions such as the presence of defects such as knots and shakes which are likely to be the situation in the use of timber for construction. To ensure this the dimensions of the specimen were increased from 50 x 120 x 2500mm to 100 x 200 x 3000mm. It was also done to obtain more data on the mechanical properties of the selected species so that a relatively more credible classification of the species can be made in terms of the tests conducted. The species were *Canarium schweinfurthii* (Bediwonua), *Sterculia rhinopetala* (Wawabima), *Albizia ferruginea* (Awiemfosamina) and *Blighia sapida* (Akye)

These species were selected not only on the basis of the above results but also on their durability characteristics and availability in the forest. Quartey et al (2008) conducted durability tests on the same species in a related research at the Faculty of Renewable Natural Resources, KNUST. From the results, Albizia *ferruginea* was found to be the most durable species. Both Stereulia *rhinopetala* ~~and~~ Blighia *sapida* were found to have good natural durability characteristics. Canarium *schweinfurthii* was found in a later analysis of Quartey's work to have poor natural durability. It however showed good structural properties in terms of bending strength in the first series of tests. It therefore became necessary to conduct the further tests on the three species with good durability properties.

3.7.1: Summary of Results for Albizia ferruginea (AF)

Sample Nr.	Modulus of elasticity local N/mm²	Modulus of elasticity global N/mm²	Bending strength N/mm²	Shear-Modulus N/mm²	moisture content %	density kg/m³
AF41	13655	10024	23.2	155	93	981
AF42	14763	8816	25.3	149	54.3	972
AF43	12460	10221	28.0	184	74.3	845
AF45	9879	9725	21.4	129	55.7	907
AF31	14010	11539	43.3	156	91.5	823
AF32	11466	9603	22.1	153	50.6	821
AF33	13423	11153	31.6	146		922
AF34	11295	9168	39.0	163	44.2	779
Average	12619	10031	29.2	154	66.2	881
Standard dev.	1642	931	8.2	15	20.0	75
Coeff. of variation	0.13	0.09	0.28	0.10	0.30	0.09

Due to the increase in the dimensions of the specimen from 50 x 120mm (in the first series of test) to 100 x 200mm cross sections in the further bending test, it was realized that the average bending strength of 'Awiemfosamina' reduced from 49.9 N/mm² to 29.2 N/mm². This means that the previous classification of D30 from the first series of test does not hold. It is predicted to belong to C14 class. This can be accounted for by the presence of defects such as knots and shakes in the beams during the test. The beams used for the first series of test were clear of such knots and shakes since they were smaller in dimension. It is also evident in the results of the modulus of elasticity. The local modulus of elasticity reduced from 13847 N/mm² (in the first series of test) to 12619 N/mm².

3.7.1.1 Mode of Failure

All the beams failed in tension with a break in the centre of the beams. Some of the breaks were associated with noise. Some of the failure occurred along existing cracks along the grains of the beams. Some failures occurred at or near areas of recognized weaknesses and defects such as knots, spots, holes, etc. The load-deflection curve indicates a permanent deformation of 0.58mm and 1.8mmn in AF42 (local modulus) and AF42 (global modulus) respectively.

Figure 3.25: AF32 at failure

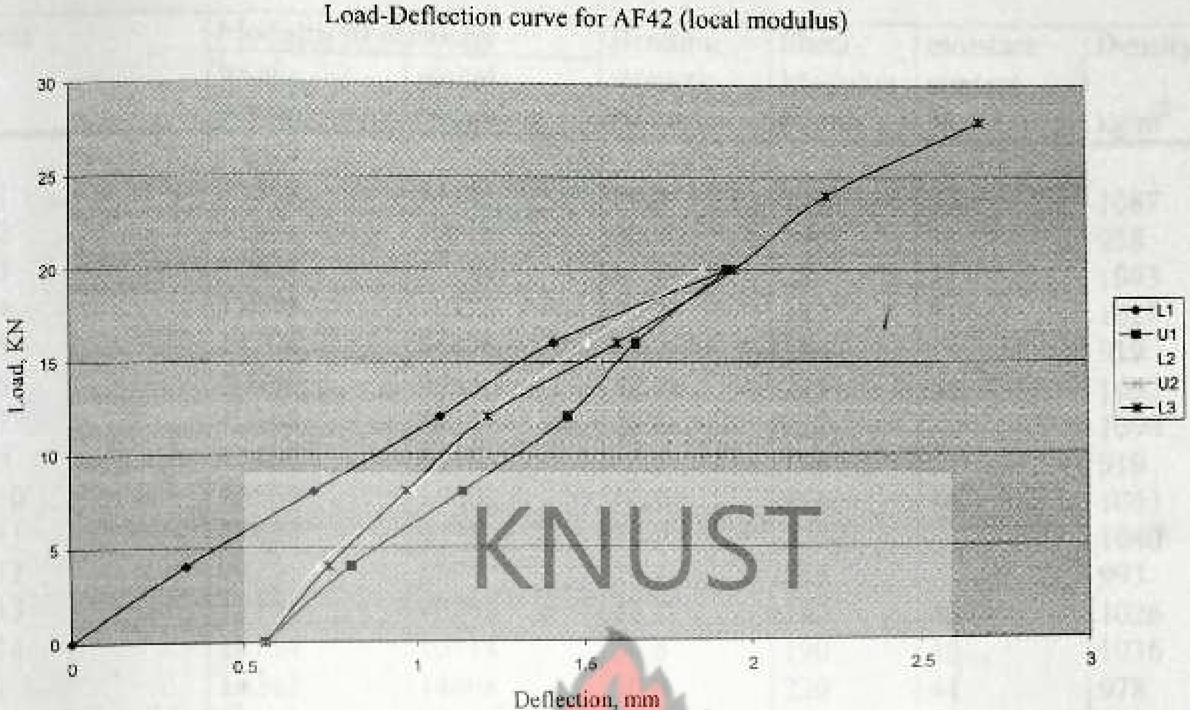


Fig. 3.26a: Load Deflection curve for AF42 (Local modulus)

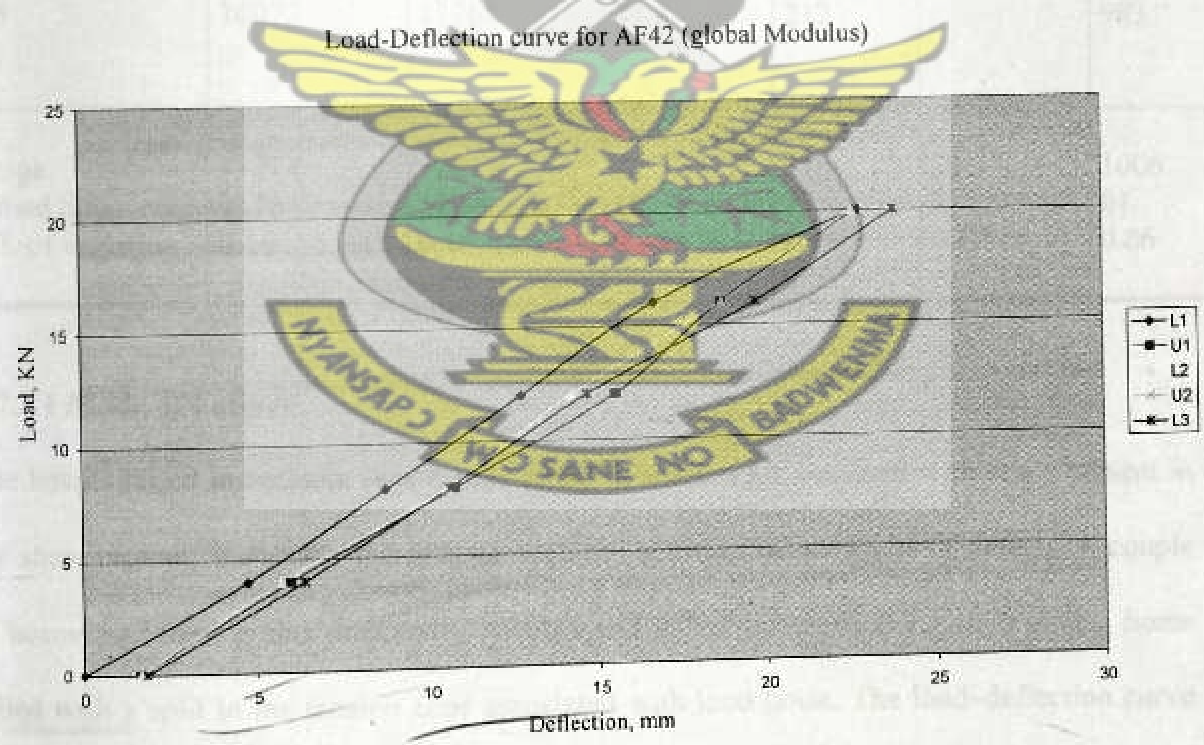


Fig. 3.26b: Load Deflection curve for AF42 (Global modulus)

3.7.2:Summary of Results for Blighia sapida (BS)

Sample Nr.	Modulus of elasticity local N/mm²	Modulus of elasticity global N/mm²	Bending strength N/mm²	Shear-Modulus N/mm²	moisture content %	Density kg/m³
BS31	13528	11385	49.6	247	37	1087
BS32	11821	10515	30.3	196	33	958
BS33	15874	12486	61.5	280	31	1093
BS34	11015	10773	27.9	182	31	1027
BS35	14104	11476	42.2	180		919
BS36	10940	9958	32.1	213	31	1062
BS37	10368	8472	28.8	222		1090
BS39	11400	8410	33.4	196		919
BS310	13564	10601	42.9	216	46	1043
BS311	13422	9435	34.0	223		1040
BS312	9829	8894	29.1	186		993
BS313	12391	10282	33.7	182	46	1026
BS314	14754	10718	34.8	190	40	1036
BS41	14242	14408	50.0	220	44	978
BS42	12900	9827	39.4	246	34	898
BS43	17126	12659	55.2	205	48	942
BS44	15579	14208	44.7	192	36	1022
BS45	16073	11263	43.8	212		983
Average	13274	10876	40	210	38	1006
Standard dev.	2116	1724	10	27	7	61
Coeff. of variation	0.16	0.16	0.25	0.13	0.17	0.06

3.7.2.1 Mode of Failure

The beams failed in tension. Few beams failed in shear. This occurred with knots present in the shear regions. Generally the failures occurred at points of weakness or defects. A couple of beams had their grains diagonally oriented and failure occurred along such grains. Some failed with a split in the tension zone associated with loud noise. The load-deflection curve shows an elastic deformation in BS31 (local modulus). The global modulus curve shows a permanent deformation of 0.08mm.

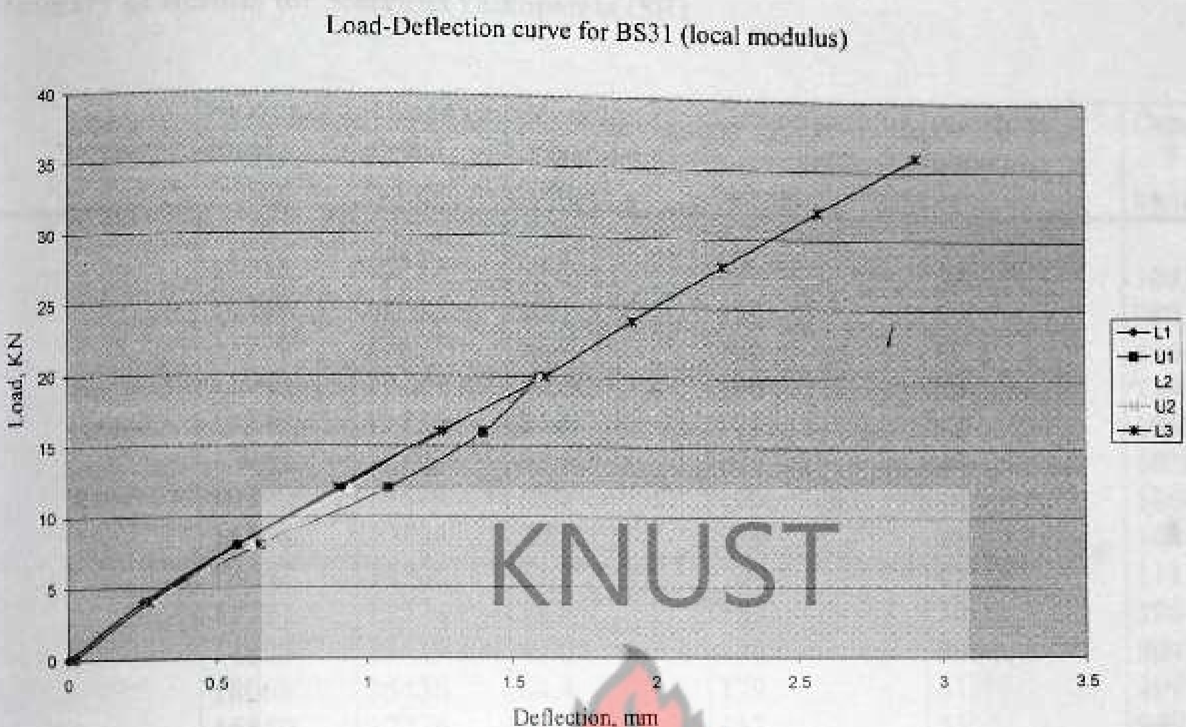


Fig. 3.27a: Load Deflection curve for BS31 (Local modulus)

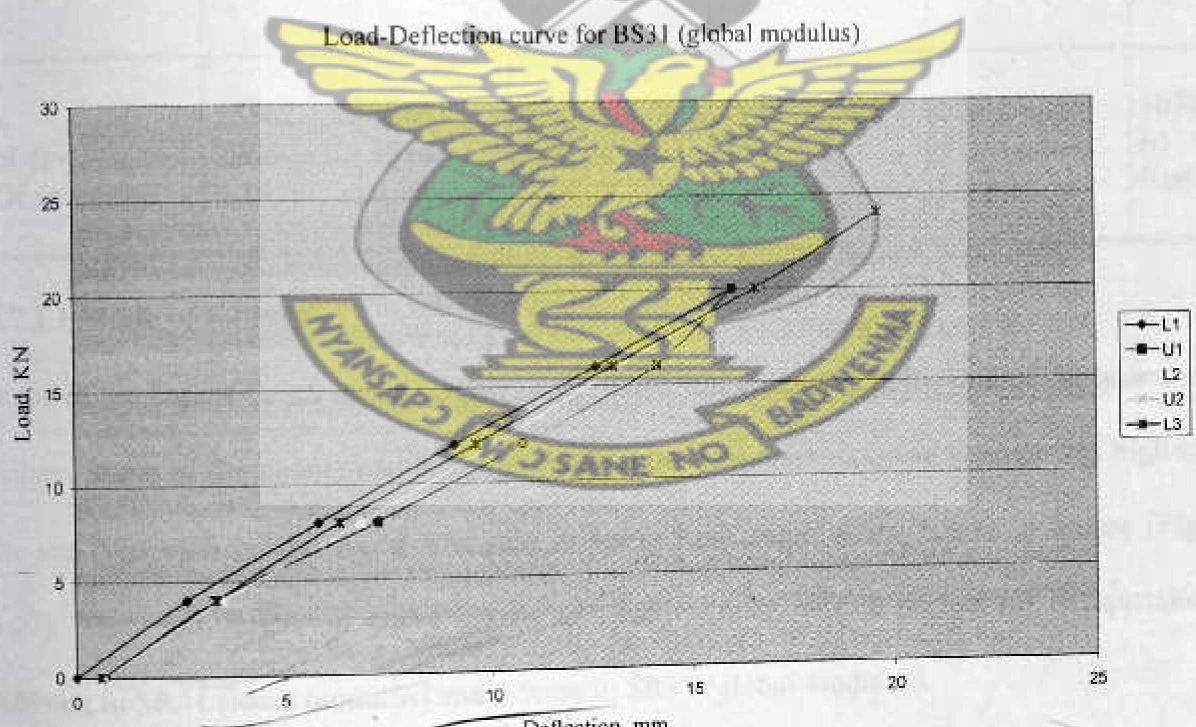


Fig. 3.27b: Load Deflection curve for BS31 (global modulus)

3.7.3: Summary of Results for Sterculia rhinopetala (SR)

Sample Nr.	Modulus of elasticity local N/mm²	global N/mm²	Bending strength N/mm²	Shear-modulus N/mm²	moisture content %	Density kg/m³
SR31	15018	12257	67.3	171	59	1033.5
SR32	16349	13045	66.7	154	59	980.2
SR33	15976	13607	62.9	166	62	1064.4
SR34	11955	10179	49.6	190	55	1141.5
SR35	14549	13135	84.6	171	42	1058.6
SR36	12967	12892	63.4	175	45	1028
SR37	10399	12479	63.4	190	48	1060.3
SR38	16378	12494	70.5	166	48	1084.3
SR39	18642	14506	63.2	188	47	1111.4
SR310	12771	11574	68.9	183	50	1084.5
SR41	14328	11419	47.0	170	58	904.8
SR42	18068	15456	64.4	179	41	1075
SR43	15880	12376	58.2	157	55	949.1
SR44	22083	14203	63.2	154	56	977.6
SR45	18070	13599	55.3	167	56	1019.2
Average	15562	12881	63.2	172	52	1038
Standard dev.	2966	1316	8.9	12	7	64
Coeff. Of variation	0.19	0.10	0.14	0.07	0.13	0.06

3.7.3.1 Mode of Failure

Some of the beams failed in the compression. There was excessive deflection of the beams at failure. Some of the beams had bearing failure at the supports. Moisture content was high so the sap (water) from the wood was seen dripping from some of the beams at failure (Fig. 3.27). The load-deflection curve shows an elastic curve with a permanent deformation 0.09mm in SR31 (local modulus) and 0.9mm in SR31 (global modulus).

Fig. 3.28: SR33 at failure with moisture dripping off

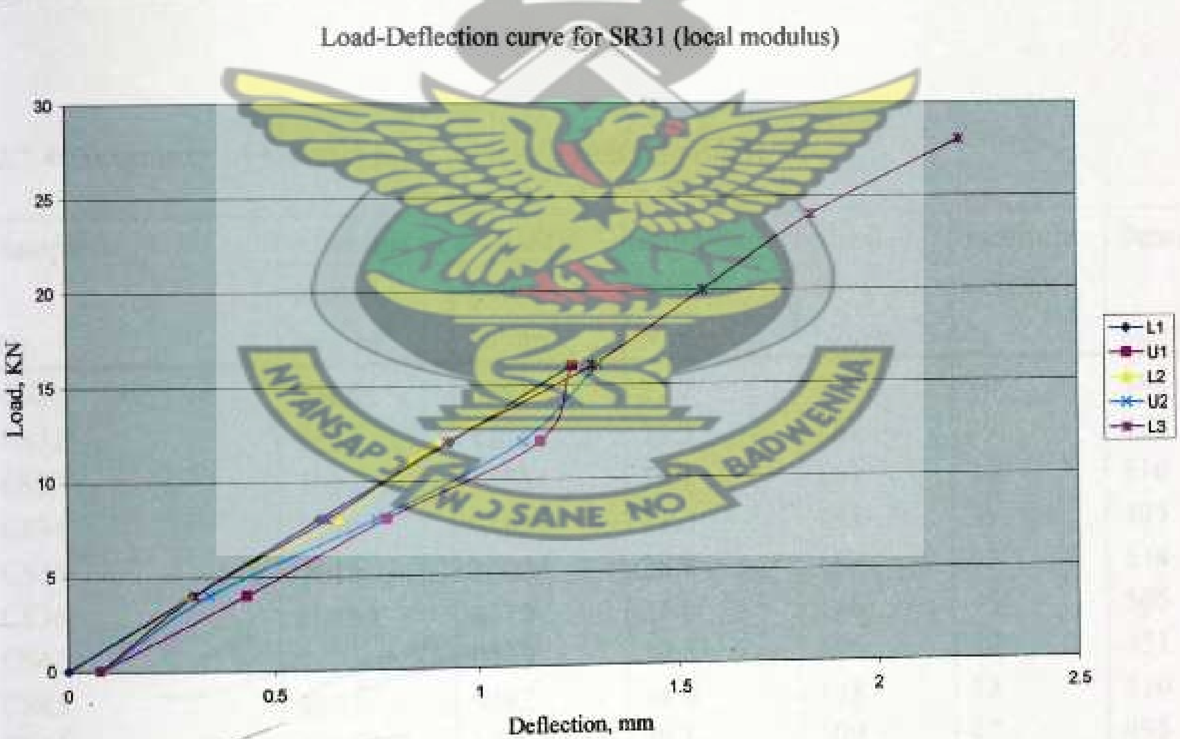


Fig. 3.29a: Load Deflection curve for SR31 (Local modulus)

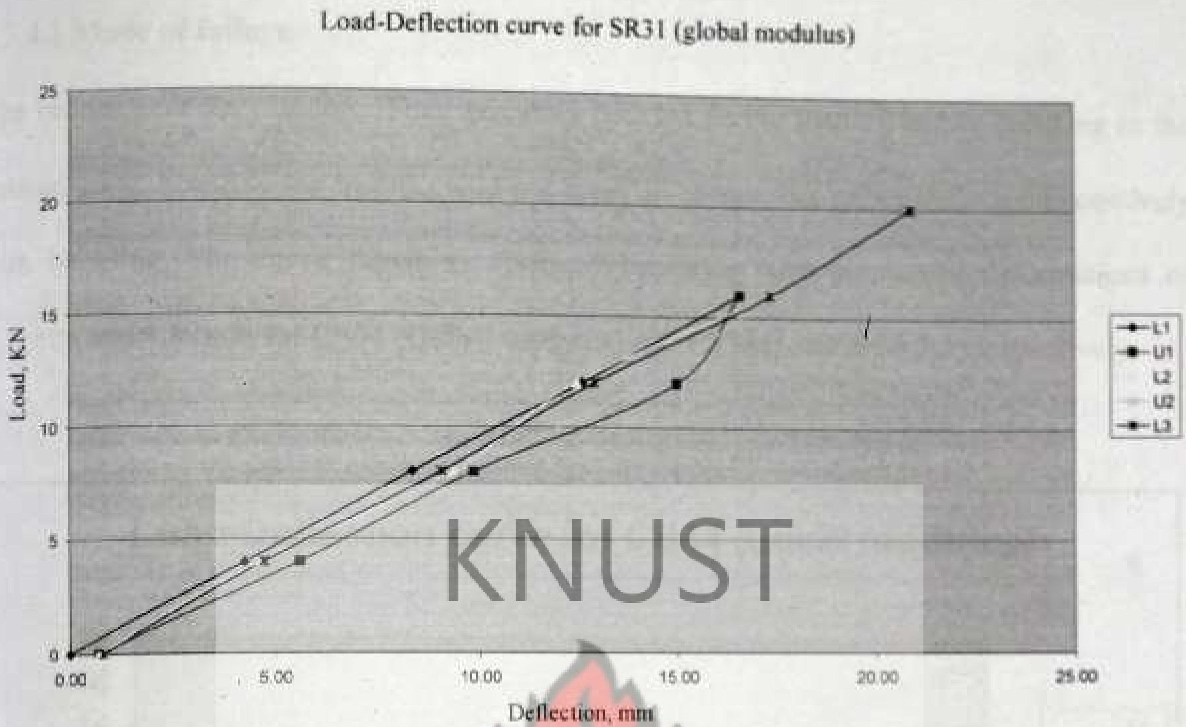


Fig. 3.29b: Load Deflection curve for SR31 (global modulus)

3.7.4: Summary of Results for Canarium sweinfurthii (CS)

Sample Nr.	Modulus of elasticity local N/mm²	Modulus of elasticity global N/mm²	Bending strength N/mm²	Shear modulus N/mm²	moisture content %	density kg/m³
CS31	7366	6691	22.7		39	448
CS32	8223	8775	23.2	185	39	529
CS33	7817	10737	30.1	164	39	510
CS34	8770	9455	21.0	155	35	495
CS35	9613	10024	28.8	187	42	534
CS36	10454	8779	34.6	158	45	505
CS41	7203	8249	12.5	136	48	451
CS43	8873	9382	34.0	128	38	510
CS45	13605	10794	20.1	109	47	495
Average	9209	9103	25.2	153	41	497
Standard dev.	1987	1286	7.2	27	4	30
Coeff. Of variation	0.22	0.14	0.29	0.18	0.11	0.06

3.7.4.1 Mode of failure

The beams were very ductile. They generally buckled during loading before breaking in the tension zone. Some of the beams could not break in tension but rather deflected excessively with buckling. The curve shows an elastic deformation with permanent deformations of 0.11mm and 1.83mm for CS34 (Global modulus) and CS34 (local modulus) respectively.

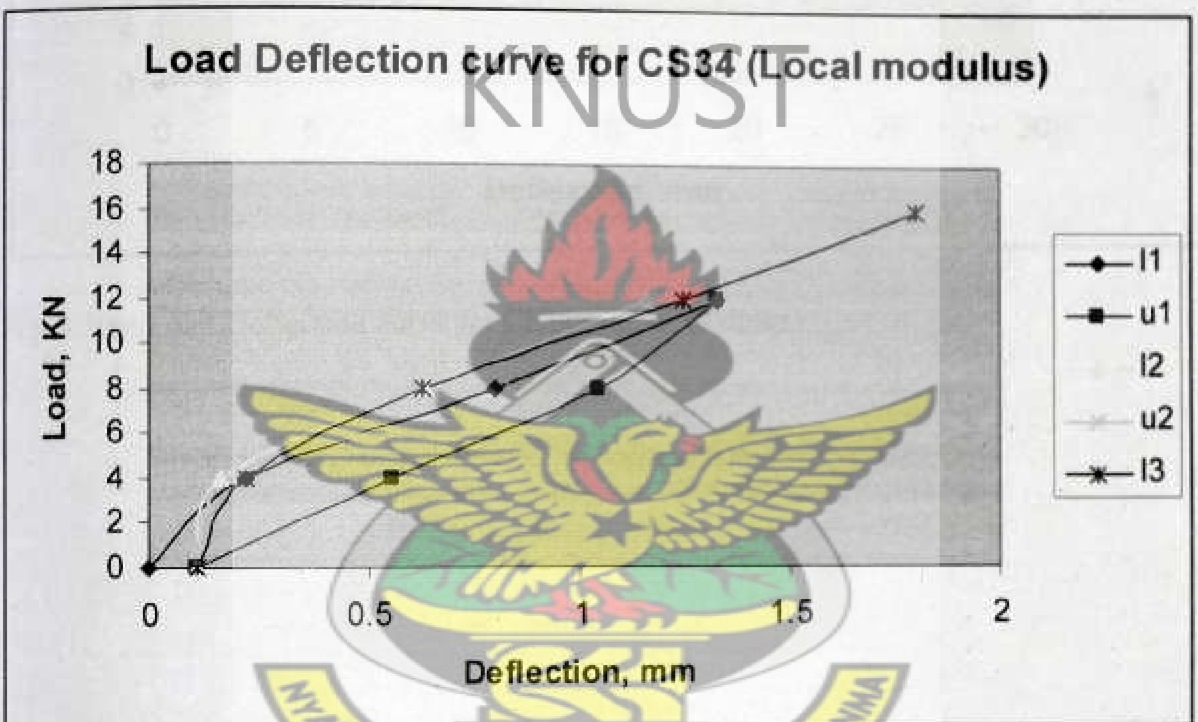


Fig. 3.30a: Load Deflection curve for CS34 (Local modulus)

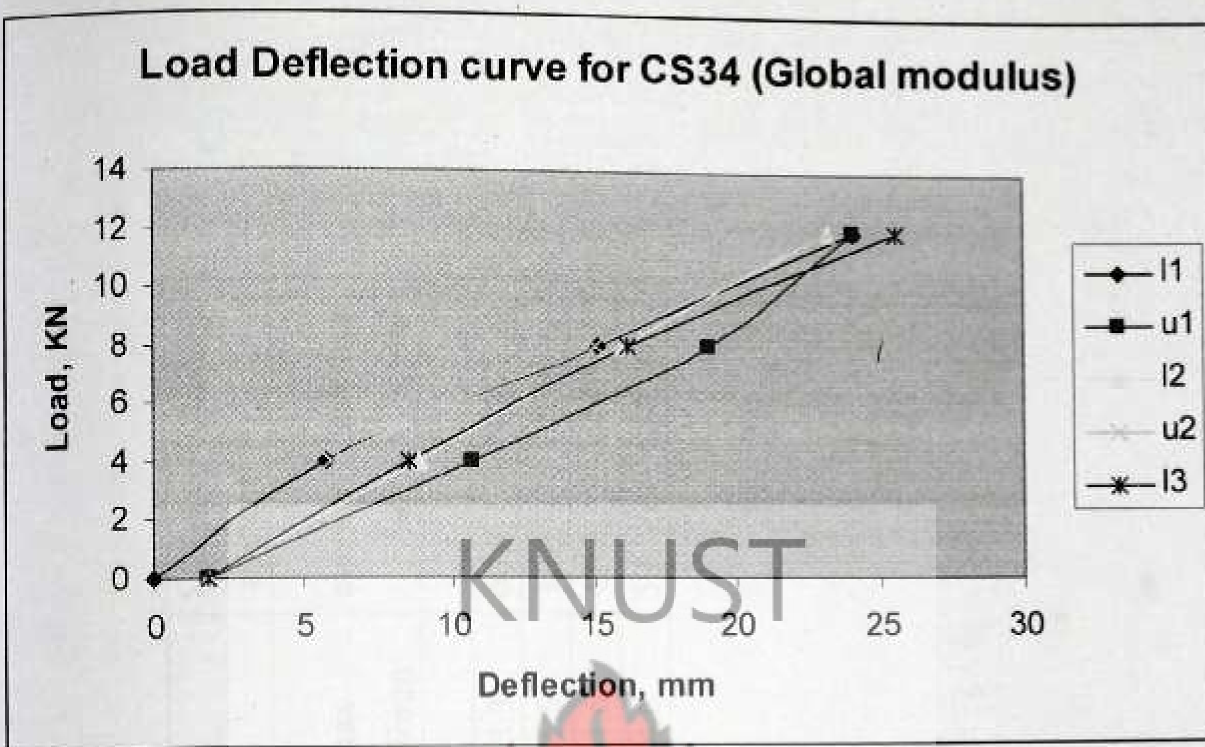


Fig. 3.30b: Load Deflection curve for CS34 (global modulus)

Table 3.14: Summary of bending test results of the 4 species

Species	Modulus of elasticity		Bending strength				Moisture content	Density
	local	global						
	average N/mm^2	average N/mm^2	average N/mm^2	5%-fractile N/mm^2	min N/mm^2	max N/mm^2	average %	average kg/m^3
AF	12619	10031	29.2	14.0	21.4	43.3	66.2	881
BS	13274	10876	39.6	19.5	27.9	61.5	38.1	1006
CS	9209	9103	25.2	11.5	12.5	34.6	41.2	497
SR	15562	12881	63.2	47.6	47.0	84.6	52.1	1038

NOTE:

AF=*Albizia ferruginea* (Awiemfosamina)

BS = *Blighia sapida* (Akye)

CS = *Canarium schweinfurthii* (Bediwonua)

SR = *Sterculia rhinopetala* (Wawabima)

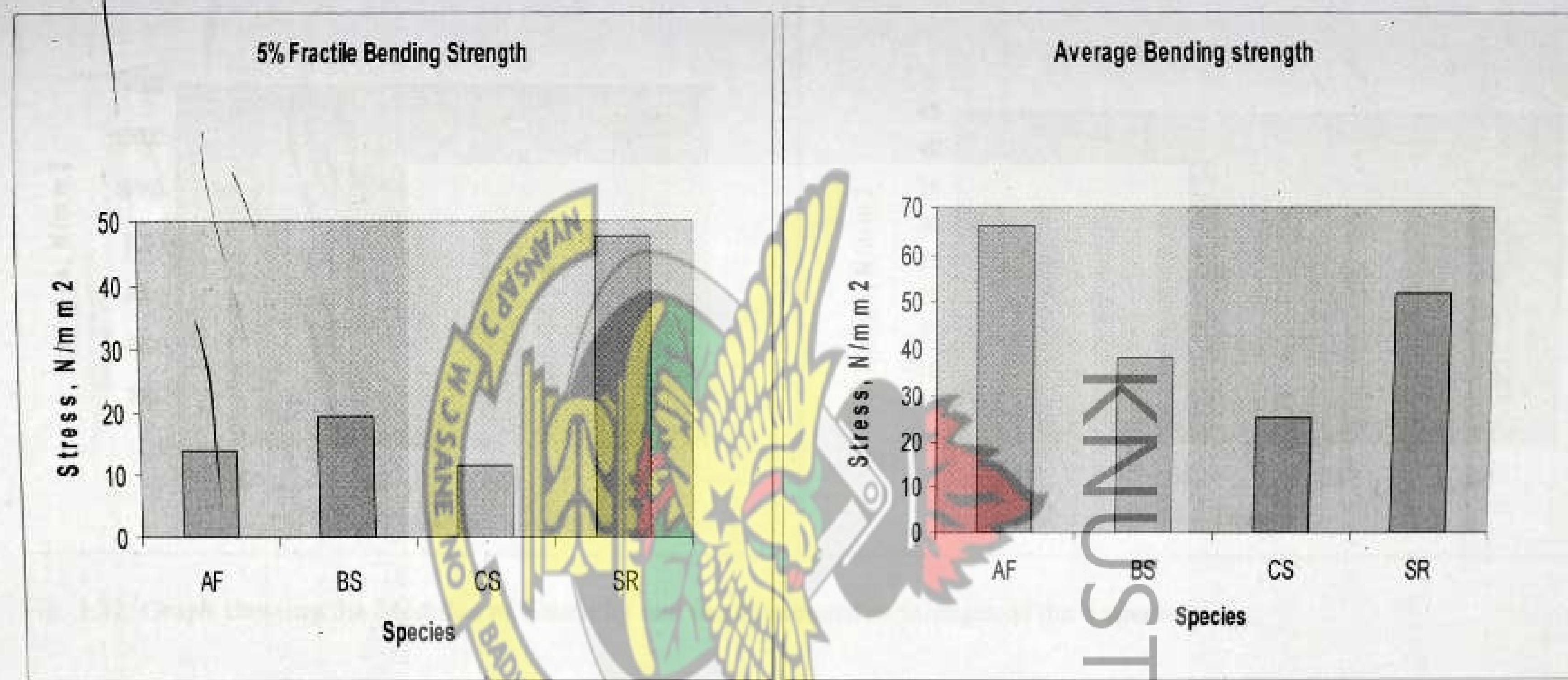


Fig. 3.31: Graphs comparing the 5th Percentile and Average Bending stresses of the 4 species

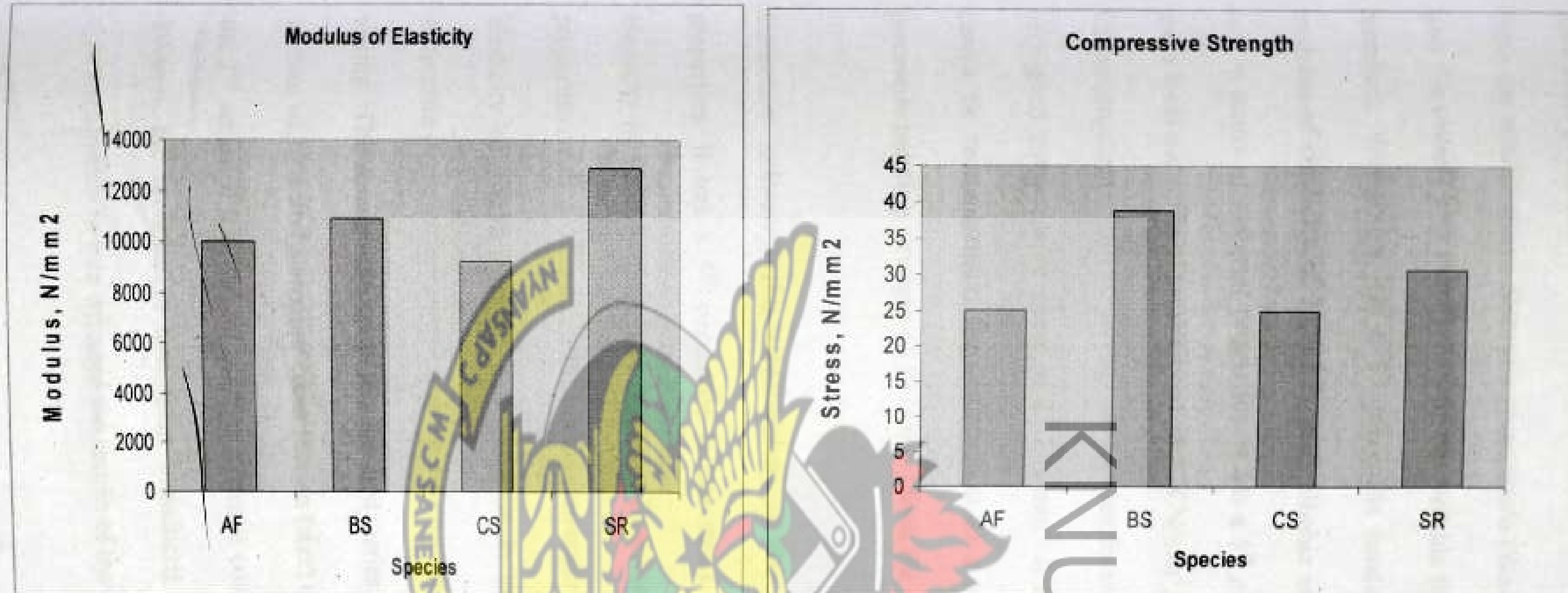


Fig. 3.32: Graph showing the Modulus of Elasticity and the Compressive Strength of the 4 species

3.6.2 Interpretation

From the results above *Sterculia rhinopetala* (Wawabima) has been found to be the species with the overall best material properties suitable for consideration in the design of structural members. Wawabima has a 5th percentile bending strength of 47.6 N/mm^2 and a local modulus of elasticity of 15,562 N/mm^2. Blighia sapida (Akye) was also found to be the 2nd best in terms of material properties. It has a 5th percentile bending strength of 19.5 N/mm^2 and a local modulus of elasticity of 13,274 N/mm^2. It can also be used as a structural member for construction. *Albizia ferruginea* (Awiemfosamina) produced some good results such as very good compressive stress of 25.2 N/mm^2. It is also very high in durability. However it cannot be recommended for construction as a structural member because it has low 5th percentile bending strength of 14 N/mm^2 from the results.

Canarium schweinfurthii (Bediwonua) is the species with relatively poor structural properties. It has a 5th percentile failure stress of 11.5 N/mm^2 and a local modulus of elasticity of 9209 N/mm^2. The results of Bediwonua in the first series of test showed better properties with a 5th percentile bending strength of 37.4 N/mm2 with a local modulus of elasticity of 10316 N/mm^2. The variation in the two sets of results is as result of the difference in moisture content and the dimensions of the test specimen used for each series of testing. The specimens used for the first series of testing had a lower average moisture content of 30% and a smaller cross section (size) of 50mm x 120mm. The specimens used for the 2nd series of test had an average moisture content of 41% and cross section of 100mm x 200mm. the increase in dimension introduced defects such as cracks and knots in the specimen which in turn affected the results of the 2nd series of testing.

The load-deflection curves of all the species show elastic deformation behaviour in all the species with permanent deformations. This means the species are not perfectly elastic i.e. the beams do not return to their original positions after loading and unloading during the test. An exception was BS31 (*Blighia sapida*) whose load-deflection curve for local modulus showed a perfectly elastic deformation. The results also indicate that, the deflection values for local modulus were lower compared to the global modulus deflections. This was due to the effect of shear deformation in the beams that occurred during the testing of the specimen which affected the global modulus deflections. In most cases, the local modulus deflections were about 10% of the global modulus deflections.

Figure 3.31 demonstrates the differences in the values of the 5th percentile and average bending strengths of the species. The 5th percentile values depend on the number of specimen tested and the standard deviation of the values obtained in the test. The differences are therefore due to the different number of specimen tested for each species and the deviation of their strength values. It is therefore important that 5th percentile values are used in design instead of average values since some specimen's values fall below the average.

3.8 COMPRESSIVE STRENGTH TEST

3.8.1 Material and Apparatus for the test

Specimens of the wood species of dimension 50 x 50 x 300mm were used for the test. This is because the code adopted for the test, EN 408 specifies that the test piece should have a length of six times the smaller cross-sectional dimension and the end surfaces should be prepared to ensure that they are plane and parallel to one another, and perpendicular to the axis of the piece. The testing machine used allows specimen height of 300mm and hence cross-section of 50mm.

The apparatus used include a compressive test Pressing Machine with a gauge which reads the load at failure, and a moisture meter to check moisture content.

3.8.2 Test Procedure

The specimens were marked for identification. The specimen was then placed and mounted in the Pressing machine set up (shown in Figure 3.33). Care is taken to make sure the specimen is placed at the central axis of the point of application of the compressive load. One end of the specimen acts as a base and is seated on a plate fixed in the machine. When the machine is set and turned on, a loading piston was released from the top which gradually touched the top end of the specimen. The intensity of the load applied increased with time. The gauge read the load applied at any time. This continued until failure was reached, where the gauge of the machine stops reading even with continual loading. The failure load is then recorded. The compressive stress was then determined using the failure load and the cross sectional area of the specimen. i.e. $\sigma_c = F/A$ where, σ_c is the compressive stress, F is the failure load and A is the cross sectional area of the specimen.

Figure 3.33: Test set up for compression test

Table 3.5: Compressive Stress Results for 40 samples per species for the 4 species

	Species			
	BS	SR	AF	CS
No. of samples	40	40	40	40
Average Compressive stress (N/mm^2)	46.54	39.36	31.49	35.41
Standard Dev.	4.62	5.21	3.76	5.49
Coeff. of Variation	0.10	0.13	0.12	0.16
5th Percentile Comp. stress (N/mm^2)	38.78	30.60	25.17	26.17

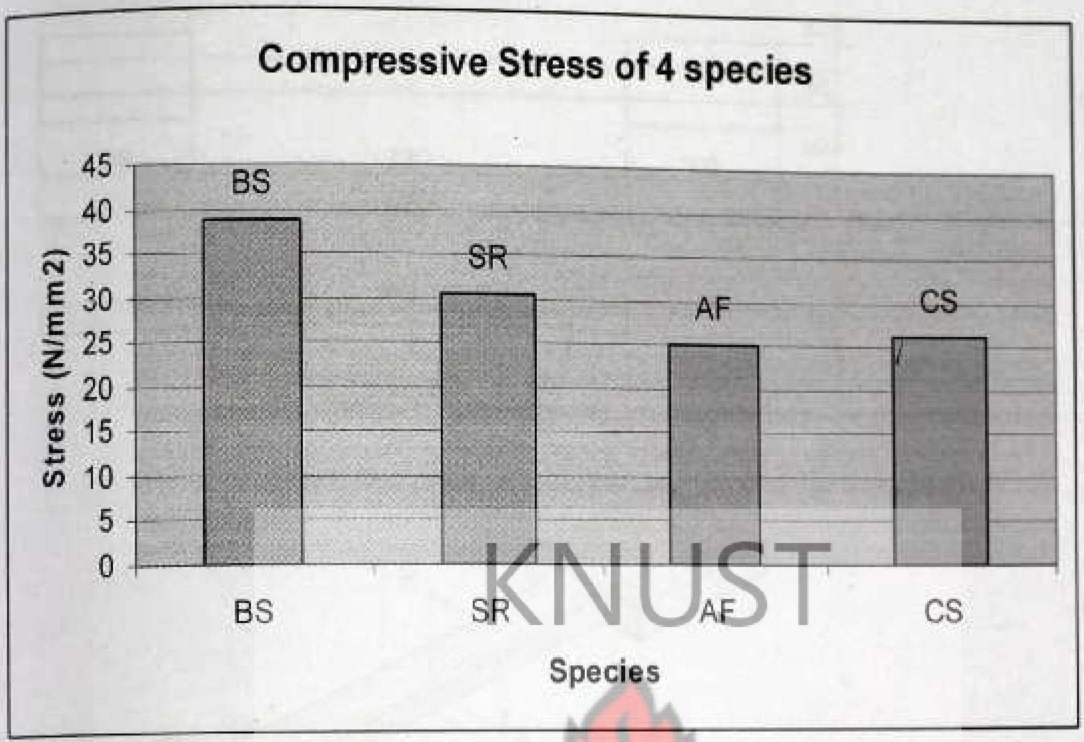


Figure 3.34: Comparison of compressive stress of 4 species

3.9 TENSILE STRENGTH TEST

Tensile strength test was conducted in the Strength Test Laboratory of the Bern University of Applied Science (BFH), Biel in Switzerland. The standard or code adopted for the test was DIN 52188 which is used in the testing of wood; determination of ultimate tensile stress parallel to grain.

3.9.1 Test Material

Specimens of dimension 25 x 25 x 500 mm of the species were sent to Switzerland for the test. The specimens were then machined into the form as required by the test standard (shown in the figure).

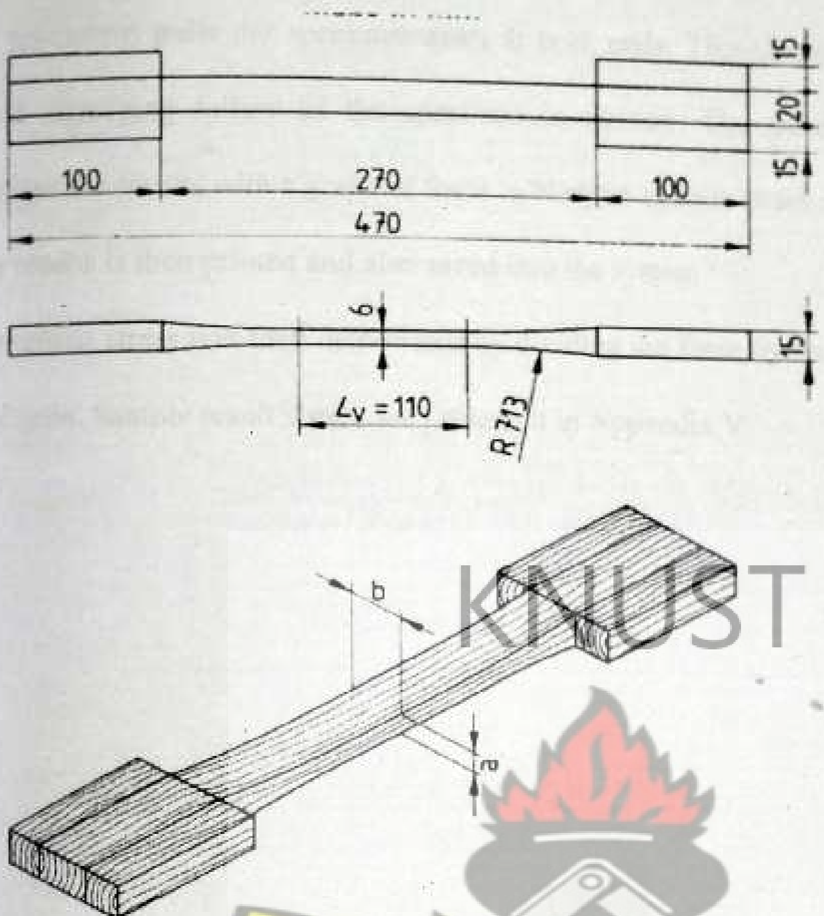


Figure 3.35: Test specimen machined to test standard dimensions

3.9.2 Test Apparatus and Procedure

Tensile Strength Test Machine with its computer and printer set up, Electronic outside Caliper, and a weighing balance.

The dimensions of the test specimen are taken including its weight for their density determination. The specimen is then put in the machine which is anchored rightly at the top and bottom within the set up. Information on the species is then fed into an attached computer unit of the machine. The dimensions of the specimen are also input into the computer. When the load is applied, the pistons of the machine (which acts as the anchors of the specimen) pulls the specimen apart at both ends. This continues until a sharp noise is heard signifying failure of the specimen in tension. The monitor of the computer unit displays the results with a graph of force in Newton against strain in mm plotted.

The results is then printed and also saved into the system.

The tensile stress was then determined by dividing the force by the cross sectional area of the specimen. Sample result sheets are presented in Appendix V.

Table 3.16: Summary of Results of the 4 species

Species	Modulus of elasticity		Bending strength		Moisture content	Density	Compressive Strength		Tensile strength
	local	global							
	average N/mm^2	average N/mm^2	average N/mm^2	5%-fractile N/mm^2	average %	average kg/m^3	average N/mm2	5%-fractile N/mm^2	5%-fractile N/mm2
AF	12619	10031	29.2	14.0	66.2	881	31.5	25.2	22
BS	13274	10876	39.6	19.5	38.1	1006	46.5	38.8	31
SR	15562	12881	63.2	47.6	52.1	1038	39.4	30.6	27
CS	9209	9103	25.2	11.5	41.2	497	35.4	24.9	25

NOTE:

AF = *Albizia ferruginea* (Awiemfosamina)

BS = *Blighia sapida* (Akye)

SR = *Sterculia rhinopetala* (Wawabima)

CS = *Canarium schweinfurthii* (Bediwonua)

3.10 Comparison of Results to EN 338

From the results of all the tests conducted on the various species, Sterculia rhinopetala (Wawabima) and Blighia sapida (Akye) were found to be the best in term of their mechanical properties. Wawabima was found to be the overall best with its outstanding bending stress values. Albizia ferruginea (Awiemfosamina) was found to be the best in terms of durability properties because of the components of its extracts (Quartey, 2008). However, its bending strength was very low. Wawabima was therefore recommended to be used for construction as main structural elements and Akye was recommended to be used as supporting elements such as cladding because of its high natural durability. The strength values of wawabima have therefore been compared with strength values of EN 338 Classes D40 and D50 hardwoods (shown in the Table 3.6 below).

Table 3.6: Table comparing the Design Values of EN338 Classes D40 and D50 to Wawabima

	EN338		Wawabima
	D40	D50	(5th Percentile)
Bending, $f_{m,d}$ (N/mm^2)	40	50	47.6
Tension parallel to grain, $f_{t,0,d}$ (N/mm^2)	24	30	27.0
Compression parallel to grain, $f_{c,0,d}$ (N/mm^2)	26	29	30.6
Tension perpendicular to grain, $f_{t,90,d}$ (N/mm^2)	0.6	0.6	0.6
Compression perpendicular to grain, $f_{c,90,d}$ (N/mm^2)	8.8	9.7	9.3
Shear (N/mm^2)	3.8	5.3	4.2
E-Modulus parallel to grain (N/mm^2)	11000	14000	12880.0
E-Modulus perpendicular to grain (N/mm^2)	750	930	840.0
Shear Modulus (N/mm^2)	700	880	790.0
Density (Kg/m^3)	590	650	926.0

The characteristic values of the Tension perpendicular to grain, $f_{t,90,d}$, and Compression perpendicular to the grain, $f_{c,90,d}$, were obtained by the following relation according to EN 384:2004, which is used in the determination of characteristic values of mechanical properties and density:

$$f_{v,90,d} = \min \begin{cases} 0.6 \\ 0.0015\rho_k \end{cases}$$

$$f_{c,90,d} = 0.015\,\rho_k$$

where ρ_k is the characteristic density of test species

From EN338 Classification of hardwoods, *Sterculia rhinopetala* (Wawabima) is found to have design values above Class D40. It has values close to EN338 Class D50. From Table 3.6 above, *Sterculia rhinopetala* (Wawabima) can be said to belong to EN338 Class D40.

3.11 Conclusion

Bending, compression and tensile strength tests were conducted on all the ten (10) selected species to determine their mechanical properties. Results obtained were used to classify the various species in to their proposed strength classes according to the EN 338. Results of the first series of tests were used to select 4 species with the best strength properties (together with other factors such as their durability properties and availability of the species in the forest) for further strength tests for more detailed study.

The results indicated that *Sterculia rhinopetala* (Wawabima) was the species with the overall best properties and compares well with EN 338 class D40. It therefore belongs to strength class D40 and qualifies to be used as a structural timber e.g. in the construction of bridges. The species with the least strength properties was *Antiaris toxicaria* (Kyenkyen). It had the lowest density of 436 Kg/m^3 and a 5th percentile bending strength of 29 N/mm^2. It therefore belongs to the class of C30 which is a class of softwoods according to the EN 338.

Wawabima was recommended to be used for the construction of prototype pedestrian bridge to demonstrate its use in construction.

CHAPTER FOUR

LOCAL WOOD PROCESSING TECHNOLOGY

4.1 INTRODUCTION

The practical realization of light bridges built with lesser-utilized timber species will be successful if the bridges are built out of local materials and also by local artisans (Wuethrich, 2008). The use of local materials such as connectors and the capacity of the existing technology of local carpentries and sawmills have been assessed in this aspect of the research.

Baiden et al (2005) examined the key barriers inhibiting the use and potential of timber for housing construction, where questionnaire were issued to timber processing firms. The authors found out that though the raw materials (timber) are available, a significant proportion is processed for the export market and those delivered to the local market are inadequate or largely unsuitable for construction. The authors also cited technical barriers such as ineffective treatment of timber and the absence of skilled tradesmen as part of the key barriers inhibiting the use of timber for housing construction.

The aim of this aspect of the research is to enable the local wood processing industry to improve their existing technologies in support of an increasing construction of timber bridges. The capacity of sawmills ~~to~~ process timber into appropriate structural sizes for bridge construction were assessed. The skills of carpenters and carpentries readiness and capacity to work with lesser known species in the construction bridges were also assessed.

The following areas were therefore researched in the chapter:

- Existing timber bridges in the region of Kumasi
- Technologies of existing sawmills in the region of Kumasi
- Technologies of existing carpentries in the region of Kumasi

4.2 EXISTING TIMBER BRIDGES AND THEIR STATE

The results are based on the research of seven bridges in the region of Kumasi. The superstructures of three of the bridges were made of composite steel-timber structural members. Four of the bridges had their superstructure made entirely of timber. The four timber bridges have been researched and recorded systematically.

Those four have been researched according to the following methodology:

1. General information to the bridge; where the name, location, year of construction and the institution responsible for repair and maintenance of the bridge is recorded.
2. Illustration of the bridge
3. Technical description of the bridge. This is where technical data on the bridge are recorded. The data include the type of bridge, span of bridge, width of bridge, height of bridge, nature of river bed and banks, foundations, superstructure, road bed, fastenings, preservation and painting of the bridge.

Measures that may improve durability and reduce maintenance. Based on the information and observations made, measures that may improve the durability of the bridge and hence reduce the rate of maintenance are recommended.

4.2.1 The "Kaase" Timber Bridge

Bridge Name: Kaase

Location: Kaase, Kumasi

Year of Construction: 1991

Institution responsible for repair and maintenance: Kumasi Metropolitan Assembly

Technical description of the bridge

Usage type	Heavy traffic
Span	7.9m
Width	8.0m
Height	1.3m
Banks	Concrete
Foundation	Concrete
Superstructure	Timber: Afina (*Strombosia glaucescens*), Dahoma , Kusia
Roadbed	Asphalt
Fastenings	Thread bolts and nuts, nails

Element	Condition	Action Recommended
Abutments	Good	None
Piers / Piles	Good	None
Wing wall	Good	Painting of wing wall
Main beams	Good	None
Cross beams	Good	None
Deck	Good	None
Handrails / Posts	Good	Painting of handrails / posts

Preservative	Good	Replacement of decayed braces
Paintwork	Good	None
Foundations	Good	None
River bed	Good	None
Banks	Concreted, Good	None
Approach paths	Asphalted, Good	None

4.2.1.1 Measures to improve durability and reduce maintenance of the bridge

The timber members used to brace the Afina *(Strombosia glaucescens)* piles showed signs of decay - to prevent further decay, the timber members need to be preserved.

The asphalt on the road has deteriorated - it needs resurfacing to prevent direct impact on timber decking from heavy duty trucks that ply the road. This can cause the decking to fail. In addition, exposure to periodic sunshine and rainfall will lead to expansion and contraction of the decking due to the continual intake and loss of moisture (water) in the wood members. If this is not done it can lead to gradual deterioration of the decking and hence the failure of the timber members.

4.2.2 The "Anloga-Sobolo" Timber Bridge

Bridge Name: Anloga-Sobolo

Location: Anloga, Kumasi

Year of Construction: 2005

Institution Responsible for repair and maintenance: Community

Figure 4.2a: Anloga-Sobolo bridge 1

Figure 4.2b: Anloga-Sobolo bridge 2

Technical description of the bridge

Usage type	Pedestrian
Span	26m
Width	1.3m
Height	1.25m
Banks	Natural
Foundation	Concrete
Superstructure	Timber (dimensions 2x4 inches, 2x6 inches, 4x6 inches)
Load Capacity	2.97KN on 2" x 4" joist members (Minimum predicted loading on bridge is 3.9KN)
Roadbed	Timber planks
Fastenings	Thread bolts and nuts, nails

Element	**Condition**	**Action Recommended**
Abutments	Good	None
Piers/Piles	Good	None
Wing wall	None	None
Main beams	Good	None
Cross beams	Severe deflection and warping	Preservation of lumber
Deck	Safe	None
Handrails/Posts	Good	Provide more midrails
Preservative	None	No information
Paintwork	None	None
Foundations	Good	None
River bed	Good	Excavate silt and refuse
Banks	Siltation,erosions, refuse erosion	Plastering with concrete
Approach paths	Refuse dump	Stop dumping refuse

4.2.2.1 Measures to improve durability and reduce maintenance of the bridge

It was found that unseasoned timber members were used in the construction– it is therefore recommended that wood used in bridge construction be seasoned and preservatives applied to ensure durability of the bridge.

Loads were not assessed in the design of the bridge so it appeared the bridge had exceeded its load limit hence an excessive deflection was evident. The loading capacity of the 2" x 4" joist members is estimated to be 2.97KN but the members are predicted to be carrying a minimum load of 3.9KN. This means that the bridge's load capacity has been exceeded by 1.0KN. Therefore before construction, proper assessment of loads should be done to enable the selection appropriate size of members and connectors to be used.

Additional guardrails should be provided to reduce the risk of pedestrians (especially children) falling off the bridge into the river.

Nails were not galvanized so had rusted. The use of galvanized bolts instead of nails could bring an improvement.

Refuse were being dumped at the bridge site. Refuse dumps serve as incubations for bio-deteriorators, which may attack timber members of the bridge. The dumping of refuse in the surroundings of the bridge should be avoided.

Desilting should also be done to allow the free flow of the river to prevent stagnation of water which causes the water level to rise and can cause flooding of the banks.

4.2.3 The "Susuankyi" Timber Bridge

Bridge Name: Susuankyi

Location: Susuanso, Kumasi

Year of Construction: 2004

Institution Responsible for repair and maintenance: Community

Figure 4.3a: Susuankyi bridge 1

Figure 4.3b: Susuankyi bridge 2

Technical description of the bridge

Usage type	Pedestrian
Span	15.9m
Width	1.4m
Height	1.8m
Banks	Natural
Foundation	Concrete
Superstructure	Timber (dimensions 1x4 inches, 2x4 inches, 2x8 inches)
Load Capacity	2.76KN on 2" x 4" joist members (Minimum predicted loading on bridge is 2.9KN)
Roadbed	Timber planks
Fastenings	Nails

Element	Condition	Action Recommended
Abutments	None	None
Piers/Piles	None	None
Wing wall	None	None

Main beams	Fairly safe	None
Cross beams	A lot deflection and warping	Preservation of lumber
Deck	Poor	None
Handrails/Posts	Good	Provide more midrails
Preservative	None	Non information
Paintwork	None	None
Foundations	None	None
River bed	Siltation, erosions, refuse	Excavate silt and refuse
Banks	overtaken by refuse, erosion	Plastering with concrete
Approach paths	heavily dump with refuse	Stop dumping refuse

4.2.3.1 Measures that may improve durability and reduce maintenance

The timber members used were mostly made of Wawa species and therefore not durable. It is recommended that the Wawa members in the bridge be seasoned and preservatives applied to ensure the bridges safety and durability.

Loads were not assessed so it appeared the bridge had exceeded its load capacity hence excessive deflections. The loading capacity of the 2" x 4" joist members is estimated to be 2.76KN but the members are predicted to be carrying a minimum load of 2.9KN. This means that the bridge's load capacity has been exceeded by 1.2KN. Therefore, before construction proper assessment of loads must be done to commensurate the size of members to be used.

Only ~~nails~~ were used and they were not galvanized so had rusted. Galvanized bolts should have been used where nails were used. The use of bolts instead of nails will bring an improvement.

The approaches of the bridge are used as refuse dump so the soil at the embankment is largely humus and so cannot render any formidable foundation for the bridge. For the proper functioning of the bridge the soil should be engineered.

4.2.4 The "Atonso-Ahinsan" Timber Bridge

Bridge Name: Atonso- Ahinsan

Location: Atonso, Kumasi

Year of Construction: 2004

Institution Responsible for repair and maintenance: Kumasi Metropolitan Assembly

Figure 4.4a: Atonso- Ahinsan bridge 1

Figure 4.4b: Atonso- Ahinsan bridge 2

Technical description of the bridge

Usage type	Pedestrian
Span	33.6m
Width	1.35m
Height	1.3m
Banks	Natural
Foundation	Concrete
Superstructure	Steel and timber (dimensions 2x6 inches, 2x8 inches, 4x4 inches)
Load Capacity	6.4KN on 2" x 6" joist members (Minimum predicted loading on bridge is 4.05KN)

Roadbed	Timber planks
Fastenings	Thread bolts and nuts, nails, metal plates

Element	Condition	Action Recommended
Abutments	Good	None
Piers/Piles	Good	None
Wing wall	None	None
Main beams	Good	None
Cross beams	Good	None
Deck	Good	None
Handrails/Posts	Good	None
Preservative	Not known	None
Paintwork	Good	None
Foundations	Good	None
River bed	Siltation, erosions, refuse	Excavate silt and refuse
Banks	overtaken by refuse, erosion	Plastering with concrete
Approach paths	heavily dump with refuse	Stop dumping refuse

4.2.4.1 Measures that may improve durability and reduce maintenance

Refuse had been dumped at the bridge site and also in the river. Refuse dumps serve as incubations for bio-deteriorators, which may attack timber members of the bridge - refuse dumps in the surroundings of a wooden bridge should be avoided.

The approaches to the bridge are always muddy any time it rained. The drainage at the site should be improved to avoid stagnation of water.

The loading capacity of the 2" x 6" joist members is estimated to be 6.4KN but the members are predicted to be carrying a minimum load of 4.05KN. This means that the bridge's load capacity exceeds the minimum predicted load currently on being sustained by the bridge.

4.2.5 Conclusion on overview of non-engineered bridges

It was found that the existing timber bridges in the region of Kumasi had a little or no engineering design in their construction. It was always a group of carpenters in a community who come together to put up a structure which will enable the people to cross the river from one point to another. This was clearly evident in the nature of the bridges.

There were excessive deflections and in the case of the Susankyi Bridge, the bridge was almost collapsing. However, the builders demonstrated that they had some ideas in bridge design which was evident in the design of the Anloga-Sobolo Bridge, built as a truss. This means that local builders can build properly engineered bridges with supervision from engineers. The loading capacity of the members was less than the actual loads on the bridges which makes it dangerous for usage. Loads expected on the bridges should be assessed before the selection of bridge members in order to choose the right member sizes. The "Anloga-Sobolo" and the "Susuankyi" Bridges were constructed with 2" x 4" (50mm x 100mm) joist members which were inadequate. The Ahinsan-Atonsu Bridge was constructed with 2" x 6" (50mm x 150mm) members so its loading capacity exceeded the expected load on it. It is therefore recommended that such bridges are constructed with 2" x 6" joists instead of 2" x 4" joists.

The dumping of refuse near streams and bridge sites was a usual practice. Almost all bridge sites visited were refuse dumps. The continual deposition has resulted in a heap of rubbish which makes it difficult to even construct foundations of these bridges on firm soil. The bridge foundations were hanging in the decomposed refuse (humus). It is therefore advisable that excavation of the garbage (refuse) be done before the foundations of future engineered bridges are constructed on such bridge sites.

4.3 TECHNOLOGY OF EXISTING SAWMILLS

The technology of existing sawmills were ascertained to determine the readiness of the Sawmills to process other secondary (lesser known) timber species into required structural sizes for the construction of bridges. Reviving the timber construction industry in the use of lesser known species for construction require an assessment of the state of the Sawmill industries and their readiness to prepare these species into lumber. The research sought to look at the capacity of the Sawmills in terms of their machinery, labour, production levels, type of species currently working with, etc. Twelve sawmills in Kumasi were visited out of which four had closed down due to operational difficulties. Eight of the Sawmills responded to the questionnaire administered and also granted interviews. The Sawmills include Kumi and Company Limited, Modern Wood Technology Limited, Naja David Veneer & Plywood Limited, Logs and Lumber Limited, Sunstex Company Limited, Logwood Industries Limited, AG Timber Limited and Ridge Timber Company Limited. Most of them produce lumber and other products mainly for export.

The Sawmills' operation and pricing are largely regulated by the Timber Industry Development Division of the Forestry Commission of Ghana. The norms and regulations for sawmill operation in Ghana and the states of the Sawmills have been outlined below.

4.3.1. Norms and Regulations for Sawmill Operation

The Timber Industry Development Division (TIDD) of the Forestry commission has norms and regulations for sawmill operation (TIDD, 1998) and these are basically as follows:

(i) One must have the necessary machinery before one can apply for a permit to operate a Sawmill.

(ii) Sawmills are categorized based on their size and finished products.

(iii) Sawmills can either apply for a working area to obtain the logs from or buy from loggers or do both.

(iv) Application for concessions goes through various offices from the district level to the national level and when it is granted, a quota (number of trees to cut in a year, species, size etc) is stated which the loggers must adhere strictly to.

(v) A sawmill operating in a working area must fulfill social responsibilities to the community in which the concession is located and this is usually negotiated with the people in the community.

(vi) When sawmill is in operation, the forestry division:

- Inspects all work done on timber from felling of the trees to the finished products.
- Has check points at various points to check source and destination of logs being transported and to arrest illegal transporters.

- Have officials visiting sawmills to inspect production procedures and finished products.
- Have officials prepare certificates for buyers who are exporting the products.
- Ensures royalties are paid on the concessions given. A greater portion goes to the stool lands and the rest goes to government.

(vii) Smaller mills produce timber for the local market but sawmills which produce largely for export are required to produce 20% for the local market and are expected to submit documents to prove this.

The grades of timber produced in the Sawmills are the FAS (i.e. First and Second), No.1 C & S (i.e. No.1 Common & Select) and No.2 C & S (i.e. No.2 Common & Select). This grading is based on the general appearance, nature of stains, wormholes, pin knots and cracks in the timber.

The FAS grade is a combination of the first and second selection after the lumber is sawn. These are usually straight members without defects. The No.1 common & select is the next grade after FAS, made up of straight members with minimal defects. They are selected after the first and second sorting. The No.2 common & select is the third grade made up of straight boards with visible defects. Their selection comes after the No.1 common & select had been sorted.

4.3.2 Outcome of Survey

Twelve sawmills were visited in the Kumasi Metropolis. Eight of the Sawmills were operational and four had closed down. The outcome of the survey is based on the response of the eight sawmills who granted interview and responded to the questionnaire which were sent to them.

Table 4.1: Type of machinery used by Sawmills

Machinery	No. of Sawmills	Percentage out of 8 (%)
Cutting/processing machines	8	100
Kiln Dryer	2	12.5
Moving machines	3	37.5

Table 4.1 shows that all the 8 sawmills have the various cutting and processing machinery such as Band Mills, Cross cut saws, Edgers, Planners, Rippers etc. Three of the Sawmills visited which represent 37.5% have moving machines such as Cranes, lifts, and forklifts for moving and transporting heavy materials. Only two (25%) of the sawmills have a Kiln Dryer. The others either air dry or kiln-dry at the Sawmills with Kiln Dryers. The lack of Kiln Dryers in most the Sawmills affect their production capacity since they have to wait for longer periods to air-dry or pay huge sums to wait for kiln-drying their wood before delivery to clients.

Table 4.2: Years of usage of machinery

Years of Machines	Frequency	Percent (%)
< 1 year	0	0
1-5 yrs	0	0
5-10 yrs	2	25
>10 yrs	6	75
Total	8	100

Table 4.3: Rate of Breakdown of Machinery

Breakdown of Machines	Frequency	Percent (%)
Very often (daily/weekly)	1	12.5
often (monthly)	1	12.5
Not often (3-6 months)	1	12.5
Hardly (yearly or more)	5	62.5
Total	8	100

Seventy-five percent (75%) of the Sawmills have used their machinery for over 10 years and 25% of them had used their machinery between 5 and 10 years. None of the Sawmills visited had a processing machine purchased within the last five years (Table 4.2). The Sawmills also indicated that most of their machines were purchased brand new except for some brands which were only used ones. Five of the Sawmills (62.5%) indicated that their machines hardly breakdown (yearly or over) and only one Sawmill indicated that their machines breakdown very often (daily or weekly) causing delays in meeting contracts (Table 4.3). This explains why most of the machines had been in use for over 10 years without replacement (Table 4.2).

Table 4.4: Availability of Spare parts for repairs

Spare parts	Frequency	Percentage
Available (locally)	7	87.5
Available (imported)	1	12.5
Unavailable	0	0
Total	8	100

Seven of the Sawmills (87.5%) indicated that spare parts for repairs and maintenance of their machinery are available locally. All the Sawmills also mentioned that they have a maintenance department who are constantly checking on the machines. The routine maintenance and the availability of spare parts keep the machines running without severe and often breakdowns.

Table 4.5: Processing of lumber for local market

Lumber for local market	Frequency	Percent (%)
Yes	8	100
No	0	0
Total	8	100

Reasons	Frequency	Percentage (%)
Easy transport	0	0
Inexpensive handling	2	25
TIDD regulation	5	62.5
Ready market	1	12.5
Total	8	100

All the eight Sawmills (100%) responded that they produce lumber for the local market. They gave reasons as indicated above. 62.5% of them said they produce for the local market because it is regulation from the Timber Industry Development Division of the Forestry Commission. Twenty-five percent (25%) said they process for the local market because it easy and inexpensive in handling products to the local market compared to the export market. One Sawmill indicated that the local market is a readily available market even if there are no foreign contracts. The results above indicate that the Sawmills are sending lumber to the local market because it is a regulation by the Government (TIDD). It is a regulation by the TIDD that at least 20% of production should be sent to the local market (the Sawmills can export up to 80% of their produce).

Table 4.6: Processing of lumber for export

Lumber for export	Frequency	Percent (%)
Yes	8	100
No	0	0
Total	8	100

Reasons	Frequency	Percentage out of 8 (%)
Foreign exchange	5	62.5
Profit	1	12.5
Expensive for local market	3	37.5
Local market unavailable	3	37.5

All the Sawmills (100%) again indicated that they produce lumber for export. They gave reasons for producing for the export market. Five of the Sawmills (62.5%) produce for the export market because of the foreign exchange and 37.5% of the Sawmills said their products are expensive for the local market. This is because their production cost is high and they spend a lot of money maintaining their machines for production. The local market is also not ready to pay for such high cost of products so the Sawmills export for foreign exchange in order to make profit.

Table 4.7: Treatment of wood for local market

Treat local wood	Frequency	Percentage
Yes	1	12.5
No	7	87.5
Total	8	100

Seven (87.5%) out of the 8 Sawmills indicated that they do not treat the wood sent to the local market. No preservation of any form is done before the wood is sold to local clients.

This is because of the cost of preservation or drying since the local clients (Carpenters) are not ready to pay for the cost of treatment.

Table 4.8: Sizes of lumber produced for the local market

Sizes (mm x mm)	No. of firms producing size	Percentage
50 x 50	8	100
50 x 100	8	100
50 x 150	7	87.5
50 x 300	6	75
75 x 75	7	87.5
75 x 100	6	75
75 x 150	5	62.5
100 x 100	2	25
100 x 150*	1	12.5
150 x 150*	0	0
150 x 200*	0	0

* Structural sizes for construction of bridges up to 15m spans

All the sawmills (100%) indicated that they produce 50mm x 50mm and 50mm x 100mm sizes of lumber for the local market. Only one company (12.5%) produce 100mm x 150mm lumber size for the local market and none of the firms currently produce 150mm x 150mm and 150mm x 200mm lumber sizes for the local market. The Sawmills attributed this to the demand of the local market. There is no demand for structural size lumber such as 100mm x 150mm, 150mm x 150mm and 150mm x 200mm so the Sawmills do not produce them.

All the Sawmills visited produce ~~all the 3~~ grades of lumber in the market (FAS, No. 1 C&S, No. 2 C&S). The least grade, No. 2 C&S and the surplus from their sorting is what they usually send to the local market through retailers because that is what the local carpenter can afford. The FAS and No.1 C&S are expensive and are always exported.

The Sawmills indicated their interest in processing other secondary or lesser known species into the appropriate structural sizes (such as 100mm x 150mm, 150mm x 150mm, and 150mm x 200mm) if the demand is available and customers are ready to pay. The Sawmills therefore have the capacity to produce for the construction industry but patronage by local clients or market is low because of the cost.

4.4 TECHNOLOGY OF EXISTING CARPENTRIES

Several Carpentry shops and Carpenters were visited in Kumasi to ascertain their capacity in working with the lesser known species. Fifty (50) Carpenters were visited in the Kumasi metropolis but 45 of them granted interview and responded to questionnaire that were sent.

4.4.1 Outcome of survey

The outcome of the survey are summarized in the tables below:

Table 4.9: Type of tools and equipments used by Carpenters

Type of Equipment	Frequency	Percent (%)
Simple hand tools	27	60
Machines	0	0
Hand tools & Machines	18	40
Total	45	100

Sixty percent (60%) of the carpenters use simple hand tools such as hammers, chisels, spirit levels, planes etc. in their workshops while 40% of the respondents indicated that they use both simple hand tools and machines such as Planners, Cross cut saws, Table saws, Circular saws etc. This means that Carpenters are more familiar with the use of simple hand tools than the use of machines.

Table 4.10: Rate of repairs or replacement of tools and machines

Repairs and Replacement	Frequency	Percent (%)
Very often (daily/weekly)	0	0
often (monthly)	13	29
Not often (every 3-6 months)	23	51
Hardly (yearly or more)	9	20
Total	45	100

Twenty-three (51%) of the respondents indicated that their machines or tools do not breakdown often. They usually repair or replace them every 3 – 6 months. Thirteen (13) Carpenters (29%) indicated that their machines breakdown often. They repair or replace some hand tools monthly. However, 20% said they hardly repair machines or replace tools. They do such yearly.

Table 4.11: Availability of spare parts for repair of machinery

Availability of Spare parts	Frequency	Percent (%)
Available (locally)	42	93.3
Available (imported)	0	0.0
Unavailable	3	6.7
Total	45	100

Forty-two Carpenters (93%) indicated that spare parts for the repairs of machinery are available in the local market. All the carpenters also said that they do not have maintenance departments in their workshops but usually employ the services of Engineers at Suame Magazine, Kumasi when breakdowns occur.

Table 4.12: Grades of lumber in the local market

Know Grading	Frequency	Percent (%)
Yes	23	51
No	22	49
Total	45	100
If Yes, Grades	Frequency	Percent (%)
FAS	0	0
No.1 C&S	9	39
No.2 C&S	14	61
Total	23	100

Fifty-one percent (51%) of the Carpenters know about the grading and different grades of wood. 22 of the respondents, representing 49% do not know anything about the various grades of wood. They however said they have their own way selecting and grading wood for their works. Out of the 23 carpenters who know about the various grades, 61% uses the No. 2 C&S (which is the least grade) and 39% uses the No.1 C&S. According to the respondents, the FAS (i.e. First And Second) grade is not available in the local market because the Sawmills export them always. Thus, the FAS can only be obtained from the export market and is very expensive.

Table 4.13a: Obtain lumber from Sawmills

Obtain Lumber from Sawmill	Frequency	Percent (%)
Yes	9	20
No	36	80
Total	45	100

Table 4.13b: Reasons for using Sawmill lumber

Reasons If Yes	Frequency	Percentage
Guarantee of wood quality & grade	4	44.4
Wood is easy to work with		0.0
Client's preference	2	22.3
Others sources are Illegal	3	33.3
Proximity to Sawmill		0.0
Total	9	100

Table 4.13c: Reasons for not using Sawmill lumber

Reasons If No	Frequency	Percentage
Too expensive	29	80.6
Sawmills are far	2	5.6
Bureaucracy & Security checks	4	11
Difficulty in transport	1	2.8
Total	36	100

Tables 4.13a, b and c shows whether the carpentries obtain their wood from Sawmills or not and the reasons behind their choice. Eighty percent (80%) of the carpenters do not obtain their wood from sawmill while 9 (20%) of them obtain their wood from sawmill (Table 4.13a).

Out of the 9 respondents who obtain their lumber from sawmill, 44% choose to pay for the sawmill wood because of the guarantee of the wood grade and quality. This gives them good finish and so they are able to sell their products at higher prices. 33% of the 9 said it is illegal to obtain lumber from other sources such as chain saw operators (bush cuts) and 22% obtain their wood from sawmill because some of their clients prefer sawmill wood. The clients insist that they use only wood from sawmill and such clients are prepared to pay for higher cost.

Twenty-nine (80.6%) out of the 36 respondents who do not obtain their wood from the Sawmills, said wood from sawmill is too expensive so they cannot buy them (Table 4.13c). Eleven percent (11%) of the respondents do not buy wood from sawmills because of the bureaucracies at the Sawmill and several security checks. Before you obtain wood from a Sawmill, you will have to place an order and make an advanced payment and provide proof of registration of your firm. The procedure is cumbersome and so carpenters get discouraged from buying wood from sawmills.

Table 4.14: Construction of timber bridges

Constructed bridge	Frequency	Percentage
Yes	4	9
No	41	91
Total	45	100
If No	Frequency	Percentage
Don't have skills	20	48.8
Never got opportunity	19	46.3
Don't have tools	2	4.9
Total	41	100

Ninety-one percent (91%) of the respondents had never constructed a timber bridge in their career. Only 4 out of the 45 respondents had constructed a timber bridge before. However the bridges they constructed were between 3-5m in span and 1.0 -1.5m in width.

Out of the 41 respondents who said they had never constructed a timber bridge, Twenty (48.8%) do not have the skills to construct bridges. Nineteen (46.3%) of the respondents never got the opportunity to construct timber bridges. They do have the skills to construct bridges but they never got an offer to construct bridges. This means that there are a number of carpenters who can build bridges with supervision from engineers.

Table 4.15: Sizes of lumber used by carpenters

Sizes (mm x mm)	No. of Carpenters working with size	Percentage
50 x 50	45	100
50 x 100	45	100
50 x 150	30	67
50 x 300	32	71
75 x 75	13	29
75 x 100	1	2
75 x 150	0	0
100 x 100	11	24
100 x 150*	1	2
150 x 150*	0	0
150 x 200*	0	0

* Sizes usually used for construction of bridges up to 15m span

All the carpenters (100%) use 50mm x 50mm and 50mm x 100mm sizes of lumber for the their works. Only one carpenter (2%) works with 100mm x 150mm lumber size for his clients and none of the carpenters currently uses 75mm x 150mm, 150mm x 150mm and 150mm x 200mm lumber sizes for their works. The carpenters usually use smaller sizes because of the nature of works requested by clients. Works by the carpenters include furniture, door and window frames, beds, roofing and ceilings, etc. The kind of jobs performed by the carpenters do not require structural size lumber such as 100mm x 150mm, 150mm x 150mm and 150mm x 200mm so they do not work with them.

Table 4.16: Readiness to work with lesser known species in the construction of bridges

Work with secondary species	Frequency	Percent (%)
Yes	41	91
No	4	9
Total	45	100

Ninety-one percent (91%) of the carpenters indicated their readiness to work with other lesser known timber species in the construction of bridges. Only 4 out of the 45 respondents are not ready to work with other secondary species in the construction of bridges because it is not their area of interest.

Table 4.17a: Educational background of Carpenters

Any education	Frequency	Percent (%)
Yes	21	47
No	24	53
Total	45	100

Table 4.17b: Level of education

If Yes	Frequency	Percent (%)
JHS	18	85.7
SHS/STS	2	9.5
NVTI	1	4.8
Tertiary	0	0
Total	21	100

Table 4.17c: Reasons for no education

If No	Frequency	Percent (%)
Funding	19	79.2
Ready apprenticeship	3	12.5
Family trade	2	8.3
Natural skills	0	0
Total	24	100

Fifty-three percent (53%) of the carpenters have had no education of any form while 47% of the carpenters indicated that they have had education of some form (Table 4.17a). Eighteen (85.7%) of the educated carpenters ~~had education~~ up to the JHS level. Two (2) carpenters had education up to SHS level. None of the respondents have had education up to the tertiary level.

Majority of the carpenters (79.2%) who did not have any education attributed it to lack of funding while 12.5% said they had ready apprenticeship from childhood so did not see the need to attend school.

4.5 CONCLUSIONS

The timber bridges in the region of Kumasi had a little or no engineering design in their construction. It was always a group of carpenters in a community who come together to put up a structure which will enable the people to cross the river from one point to another. This was clearly evident in the nature of the bridges. However, the builders demonstrated that they had some ideas in bridge design which was evident in the design of the Anloga-Sobolo Bridge for an example, built as a truss. This means that local builders can build properly engineered bridges with supervision from engineers.

The sawmills in the Kumasi Metropolis have the needed machinery to process lesser known species for bridge construction. The sawmills indicated their readiness to process for the local construction industry if the local customers are ready to pay for the cost of the sawmill lumber. In Table 4.5, 62.5% of the Sawmills said that they produce lumber for the local market only because it is regulation from the TIDD. They do not make profit out of it.

The carpentries do not purchase lumber from the sawmills because it is too expensive for them. The carpenters rely on chain saw lumber which is of inferior quality which in turn affects the quality of work. Quality wood for construction services like the construction of bridges can not be obtained in the local market in the right sizes. Customers of local carpentries do not complain because they are not ready to pay for such quality wood from sawmills. The general public, engineers and builders should be advised to embrace wood sawn from sawmills as the required for usage and be ready to pay for its processing cost. This will make the local markets readily available and deter the sawmills from sending all the best grades of the lumber for the export market.

Ninety-one percent (91%) of the Carpenters indicated their readiness to work with structural sizes of lesser known species in the construction bridges (Table 4.15). They are therefore ready to learn, even for those who do not have any skills in bridge construction. Forty-seven (47%) of the carpenters indicated that they had education up to at least the JHS level (Table 4.16a,b). This means that with little training, some of the carpenters can be equipped with the necessary skills and guide on the use of timber for construction. Practical technical training in construction carpentry and joinery such as those offered by some Technical Institutes in Ghana, who train non-formal students for National Vocational and Technical Institute (NVTI) examinations. Participants who pass such practical examinations are given certificates for practice.

CHAPTER FIVE

DESIGN AND CONSTRUCTION OF PEDESTRIAN BRIDGE

5.1 Introduction

The preceding chapters dealt with relevant literature that helped in formulating a basis for the choice of experimental methods and documentation of work done on other relatively new timber species. Laboratory tests were performed on selected lesser known species specimen to enable their characterization in terms of their mechanical properties. Studies were then undertaken to assess the competency of existing carpentries and the capacity of sawmills in the Ashanti region to construct engineered timber bridges.

In this chapter, the design and construction of a pedestrian bridge is undertaken using the information gathered in chapters 2 - 4. Discussions are on the design criteria, structural form of bridge, load transfer system and design load considerations. The maximum theoretical design forces in the elements of the bridge were calculated for the case of applied design loads. Furthermore, the theoretical failure forces and loads were calculated to give an idea of the inherent structural over-strength possessed by the bridge. Finally the bridge erection procedure is discussed

5.2 Design Criteria

The timber members of the bridge were designed to the Swiss standard SIA 261/265 and the substructure is designed to the British Standard BS 8110 as well as the earth pressure against the retaining wall.

The distance between the river banks at the bridge crossing is 14.0m. The elevations at the tops of the banks at both ends of the stream were almost the same from the surveying conducted on site. The site is liable to flooding during the rainy season and this necessitated the foundations to be built in concrete. The river bed needed de-silting and the heavy presence of weeds impeded the flow of water. This made it necessary to omit middle piers or abutments in the design considerations in order to allow for free flow and future de-silting of the stream. The Bridge was built adjacent to an existing 12.0m concrete bridge currently being used by the large number of students attending lectures from their halls of residence. The bridge span was taken as 14.20m to match up with the existing concrete to promote easy diversion to the new bridge. The width of the bridge is 3.6m with clear walking space of 3.0m as against the 1.2m width of the existing bridge which was seen inadequate. Traffic counts were conducted and students were found sharing the carriageway of the road with vehicles during the peak hours of the day. This necessitated clear span of 3.0m to be designed to allow enough students cross the stream without competing with vehicles on the roadway which constantly posed threat to students' safety on the road.

The loads that were considered in the design were live loads, dead loads of the materials, and wind loads.

5.3 Bridge Layout

Several variants of the design ~~for the site~~ were made. In conjunction with KNUST Architects and Development Officials, the project team in KNUST settled on the 'trough' bridge which is two trusses connected by cross beams with decking on cross beams. The loads from the trusses are transmitted axially through concrete columns joined to the abutments at both ends of the trusses. The truss spans 14.20m and the 180mm x 80mm cross beams are spaced at 1.40m centres. The height of the bridge is 1.8m and is about 0.9m above the highest flood level of the stream .The trusses were fabricated in three units and joined by means of appropriately designed steel plates and nails.

The substructure of the bridge was constructed with concrete and reinforced concrete. The foundations are made of concrete and the abutments on which the timber girders are positioned are made of reinforced concrete walls. The abutments also serve as retaining walls for the backfill of laterite in constructing the approach roads to the bridge.

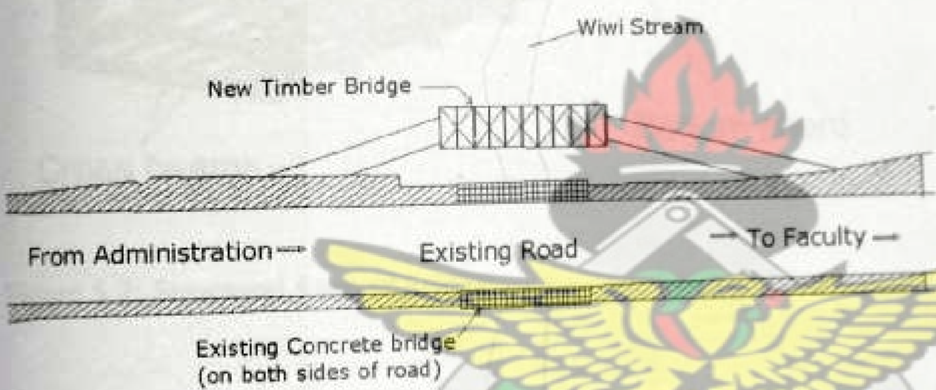


Figure 5.1: Figure showing layout of bridge site

5.4 Structural form of Bridge

The bridge structure is a truss system where two trusses are connected by crossbeams by means of steel plates and nails, and are supported at their ends by concrete columns /abutments. The crossbeams bear the decking which is the walkway of the bridge. The crossbeams receive the loads and transmit them to the trusses. The forces are distributed across the truss members (top and bottom chords, web) and transferred to the supports through the ends. The trusses also serve as guardrails for the bridge and so take up pedestrian and wind loads.

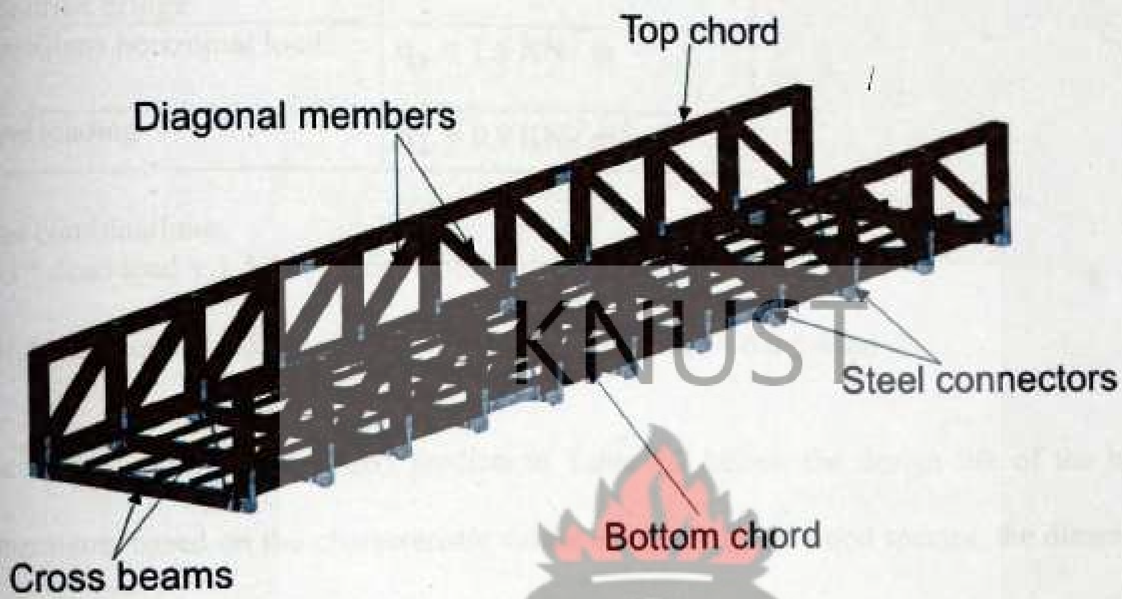


Figure 5.2: Structural form of bridge

5.5 Structural Design and Analysis

In this section, the output of structural design and failure load analysis of the timber bridge is reported.

The unit weights of the various materials used in the design for calculating dead loads are listed in Table 5.1 below.

Table 5.1: Density of materials used in the design

Material	Assumption for analysis (Density)
Wawabima	10.18 KN/m^3
Steel	80 KN/m^3
Akye	9.87 KN/m^3
Rubber	15 KN/m^3

The characteristic live load for pedestrian bridges, wind loading and the horizontal effect of pedestrian movements considered in the design are also listed below.

Characteristic live load for pedestrian bridge	$q_k = 4 \text{ KN} / m^2$
Pedestrians horizontal load	$q_k = 1.6 \text{ KN} / m$
Wind loading	$q_k = 0.9 \text{ KN} / m^2$

Load combinations:
1.35 * dead load + 1.5 * live load / pedestrian horizontal

1.35 * dead load + 1.5 * live load / pedestrian horizontal + 0.6 * wind

The design code, SIA 261/265 predicts in Table 5.2 below, the design life of the bridge components based on the characteristic values of the choice of wood species, the dimensions of members used as a result of the design loads.

Table 5.2: Duration of usage of bridge components

Designation	Material	Proposed duration of usage
Structural system (truss)	Wawabima	50 years
Steel connections	Steel welded	50 years
Cladding	Akye	25 years
Walking surface	Akye	10 years
Guardrail covering	Akye	10 years

The ultimate design load was analysed for the truss shown below:

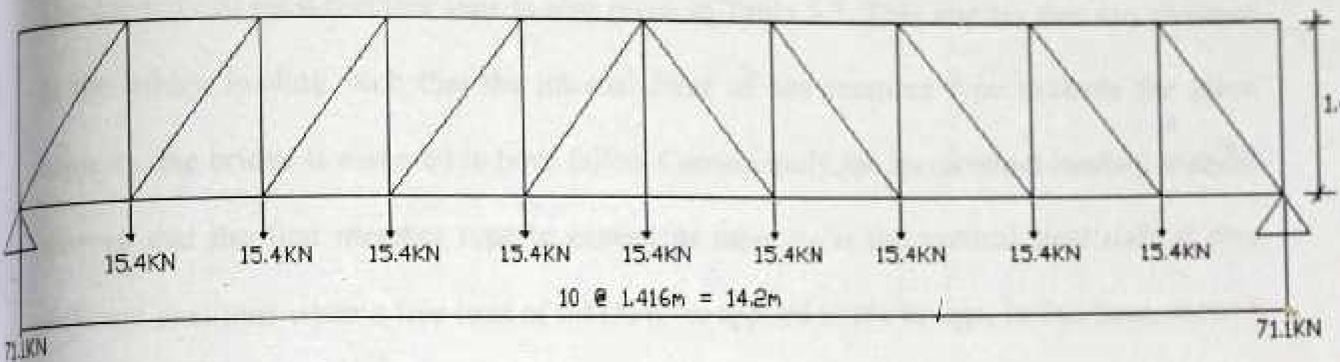


Figure 5.3: Design load analysis of timber truss

The maximum internal force calculated in the elements of the bridge and the selected member sizes based on the above design loads are shown in Table 5.3 below.

Table 5.3: Internal forces in members and loads expected to cause failure in members

Member type	Size (mm)	Grade stresses (N/mm^2)	Internal force at $4kN/m^2$ design live load (kN)	Capacity of member (kN)
Bottom chord	140 x 200	15.9	192 (Tension)	445.2
Top chord	140 x 200	18	184.8 (Compression)	259.6
Diagonal	80 x 200	18	98 (Compression)	148.3
Vertical	19mm steel rods	250	53.9 (Tension)	70.9

Since the dead load (self-weight) component of the design load is constant for the bridge, the critical load required to cause the failure of the bridge will be dependent on how much live load is available on the bridge. A characteristic live load of $4kN/m^2$ used in calculating the design loads shown in Figure 5.3 pre-supposes that the bridge can accommodate 6 people weighing 70kg each occupying a square metre of the bridge deck.

The capacity of each member type is also given in Table 5.3. This implies that any increase in the bridge loading such that the internal force of any member type exceeds the given capacity, the bridge is assumed to have failed. Consequently, an incremental loading analysis showed that the first member type to exceed its capacity is the vertical steel rods at four different positions when a live load of 5.5kN/m^2 is applied to the bridge. In this case, about 8 people of mass 70kg each are deemed to have occupied a square metre of the bridge deck for collapse to occur. The internal forces generated by such a load for the various member types are compared with their capacities in Table 5.4 below.

Table 5.4: Internal forces due to 5.5kN/m^2 live load and capacities of members

Member type	Size (mm)	Internal force at 5.5kN/m^2 live load (kN)	Capacity (kN)
Bottom chord	140 x 200	253	445.2
Top chord	140 x 200	243.5	259.6
Diagonal	80 x 200	129.2	148.3
Vertical	19mm steel rods	70.9	70.95

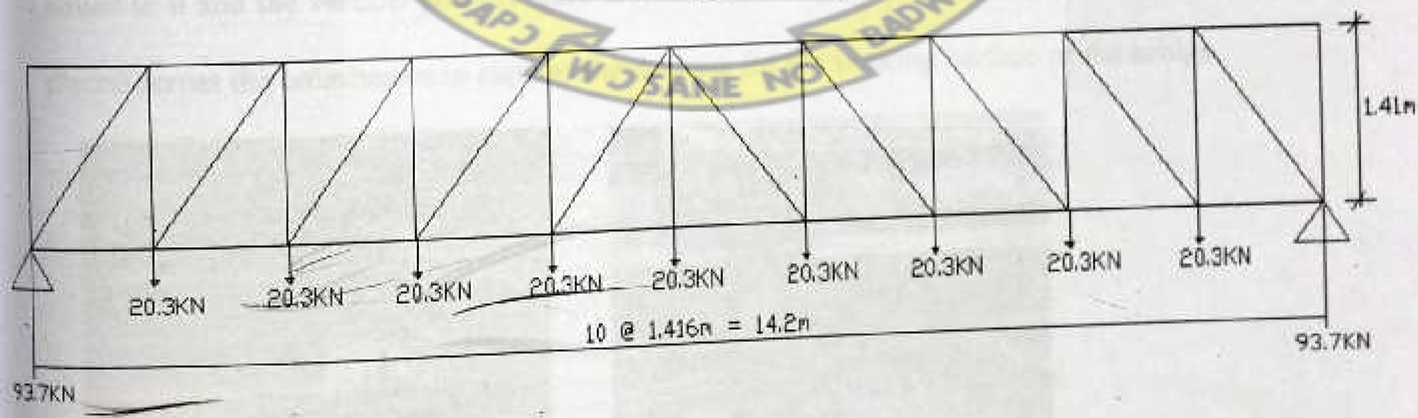


Figure 5.4: Failure load analysis of truss

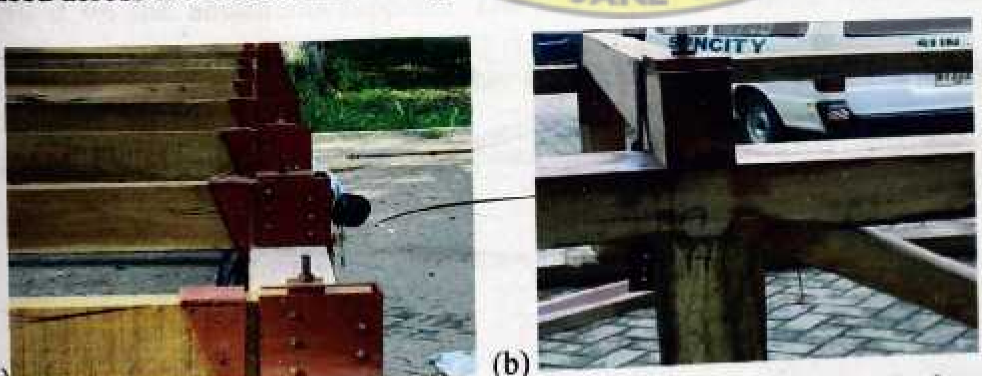
5.6 Description of structural elements

5.6.1 Trusses

A howe type of truss with a flat top chord was designed. The web systems were selected for convenience of connection and economy. Three beams, 4.74m in length with the dimensions of 200mm x 140mm were connected by steel plates to obtain each of the 14.20m top and bottom chords of the truss. The vertical web members were connected with holes drilled in them to hold steel rods from the top chord through to the bottom chord reaching down to hold the crossbeams. The vertical members again connect to the cross beam by steel plates (gussets). The diagonal members are locked in place by the geometry of cut at the joints of the truss when the steel rods through the vertical posts are tightened at the bottom of the crossbeams.

5.6.2 Crossbeams

Crossbeams of dimensions 180mm x 80mm were designed and placed at 1.4m centres along the bridge span. They were placed below the bottom chords connecting the two trusses. They are held in place by the steel rods from the top of the top chord and also by the steel plates nailed to it and the vertical posts (Figure 5.5). Members of 80mm x 40mm dimensions were placed across the crossbeams to support the decking for the walking surface of the bridge.



(a) (b)

Figure 5.5: Picture showing crossbeam connected by steel plates to the bottom chord (frame turned upside down)

5.6.3 Decking

Akye Members of dimensions 100mm x 30mm were connected to form panels by means of screwing onto connector members. These panels were then designed to be screwed to the members on the crossbeams. The panels were fabricated in the workshop.

5.7 Bridge Erection

5.7.1 Pre-Assembly of the Bridge

The 14.20m span trusses had to be connected before erection since there were no interior piers or walls to support the assembling and connections on site. The truss members were first connected with the steel plates and gussets. The trusses were turned upside down and the crossbeams were fixed. The steel rods through the vertical members of the truss were tightened with nuts after the crossbeams. This formed the main structural framework of the bridge. The assembling was done at a car park which is about 200 metres from the site. The assembling took 4 days to complete.

5.7.2 Structural Frame Erection

At the site, two 10 ton cranes had to lift the framework at both ends of the span in order to install it on the abutments/supports. The installation was done within one day. After the installation, all connections were further tightened before the decking began. The decking was made up of panels which were screwed onto planks across the crossbeams.

5.7.3 Cladding

The cladding members were also formed into panels. 70mm x 25mm planks were screwed together on to connectors to form 1.44m x 1.55m panels. The panels were then screwed on the trusses. This ensured easy construction of the cladding members. A scaffold was made in the river to provide platform for the fixing of the outer cladding. The inner cladding panels were easier to fix because the decking had been made which was enough platform for the carpenters.

Figure 5.6: The almost completed bridge

5.8 Conclusion

The bridge was constructed as part of the research objectives. The aim of the project is to develop and promote light bridges built with locally available lesser known species. This has been necessitated by the high cost of bridge construction due to the overdependence on foreign technology and materials for construction; and the overexploitation of few noble timber species in the forests.

The pedestrian Bridge in KNUST campus will ease the movement of students from their halls of residence to the faculty for lectures. It will also reduce the congestion on the roadway during peak hours. It will ensure safety of the students when crossing the stream at that section of the road. Many of such bridges can be put up with these lesser-utilized timber species to curb the above mentioned problems by addressing the local transportation problems where many streams have become traffic barriers; reducing the heavy reliance on a few widely known and used noble timber species, thus contributing to the national and international efforts to conserve the tropical mixed forest; and conserving scarce foreign reserves by using local timber instead of imported cement and steel.

The bridge is designed with a characteristic live load of 4 kN/m^2. This means that the bridge is designed to accommodate 255 people each weighing about 70 kg at a time. However, the bridge can accommodate 340 people each weighing 70 kg at a time (when loaded with a characteristic live load of 5.5 kN/m^2) beyond which the bridge will fail. The 340 people weighing about 70 kg is therefore the loading limit which can cause the bridge to fail.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

Ten lesser known timber species were investigated in terms of mechanical properties to determine their suitability for the construction of light bridges. The species are *Albizia ferruginea* (Awiemfosamina), *Sterculia rhinopetala* (Wawabima), *Blighia sapida* (Akye), *Canarium schweinfurthii* (Bediwonua), *Petersianthus macrocarpus* (Esia), *Sterculia oblonga* (Ohaa), *Cola gigantea* (Watapuo), *Celtis zenkeri* (Esa), *Antiaris toxicaria* (Kyenkyen) and *Amphimas pterocarpoides* (Lati). Four out of the ten species were selected for further tests after the first series of tests. The selection of the four species was based on the performance of the species in the first series of tests and the availability of the species in the forest. The species with the best four results were *Canarium schweinfurthii* (Bediwonua), *Sterculia rhinopetala* (Wawabima), *Albizia ferruginea* (Awiemfosamina) and *Petersianthus macrocarpus* (Esia). At the end of the second series of testing, *Sterculia rhinopetala* (Wawabima) was found to be the species with the overall best material properties suitable for consideration in the design of structural members. Blighia sapida (Akye) was also found to be the 2nd best in terms of material properties. It can also be used as a structural member for construction. *Albizia ferruginea* (Awiemfosamina) produced some good results such as very good compressive stress. It is also very high in durability. However it cannot be recommended for construction as a structural member because it had a low bending strength from the further test results.

Sterculia rhinopetala (Wawabima) was therefore used in the design of a prototype timber bridge as the elements of the main structural frame. Blighia sapida (Akye) was used in the design of the bridge as cladding elements due to its good durability properties aside its structural suitability.

The local wood technology was assessed. The Sawmills in Kumasi have the capacity to saw the species. All the Sawmills have at least one band mill which is able to saw all the tropical hardwoods (redwood). Species with similar sawing properties and conditions as Wawabima and Akye are being sawn in the Sawmills. Three (3) out of seven (7) Sawmills visited have Kiln Dryers for seasoning wood. Modern Wood Technology, located at Kaasi, for example, has 10 chamber kiln drying system fueled by sawdust generated from their operations. This system saves energy and eliminates the environmental hazard of burning sawdust. Most of the carpentries visited use simple hand tools. The carpenters were conversant with working with hardwoods (redwood). The Anloga wood working village has all the basic machines for cutting, sawing and planning wood members for all sorts of construction and moulding. Carpentry Workshops visited possessed the basic hand tools to work with the species.

A prototype pedestrian timber bridge was designed with the species and constructed on KNUST campus to demonstrate the use of the species in construction. The bridge demonstrates the suitability of Wawabima and Akye in use as a construction material. The bridge is designed with a characteristic live load of 4 kN/m^2. This means that the bridge is designed to accommodate 255 people each weighing about 70 kg at a time. However, the bridge can accommodate 340 people each weighing 70 kg at a time (when loaded with a characteristic live load of 5.5 kN/m^2) beyond which the bridge will fail. The 340 people weighing about 70 kg is therefore the loading limit which can cause the bridge to fail.

6.2 RECOMMENDATIONS

1. From Table 3.3a it is seen that *Albizia ferruginea* (Awiemfosamina) has a very high average bending stress but low 5th percentile bending stress value. This is because the numbers of test beams were few. From the 1st series of testing, *Albizia ferruginea* (Awiemfosamina) was found to have high bending strength. It is therefore recommended that further bending test be conducted to ascertain the irregularities in their bending results.

2. Assumption that joints fail after members must be checked by use of experimental connection test. A prototype connection must be studied in a laboratory.

3. The bridge must be monitored for deflections and durability whilst in use.

REFERENCES

Abbiw, D. (1990). Useful Plants of Ghana. Intermediate Technology Publications Ltd. 103/105 Southampton Row, London WCIB 4HH, UK and Royal botanic Gardens, Kew, Richmaond, Surrey TW9 SAB, UK. pp 337.

Addae-Mensah, A. (1998). Industrial Utilisation and Improved marketing of some Ghanaian lesser-used timber species. In: International Conference on value-added Processing and Utilisation of lesser-used timber species, 17-19th February, 1998. Forestry Research Institute of Ghana, Kumasi, Ghana.

Alik, D. and Badorul, H. B. (2006). Strength performance of full-size structural timber of Dryobalanops species of Sarawak, Malaysia. 7th World Conference on Timber Engineering, WCTE, Germany, June 2006.

Alik, D. and Nakai, T. (1997a). Preliminary study for structural grading based on full size bending test of *Resak durian* and *Keruing utap* of Sarawak. Proceedings of the International Tropical Wood Conference, Kuala Lumpur. June 17-20. pp.252-260.

Alik, D. and Nakai, T. (1997b). Effect of Moisture Content on Bending Properties of Wood. Proceedings of the TRTTC/JICA Research Seminar 97. Kuching. pp.138-144.

Allotey, I. (1992). "Design and Construction of a new timber bridge in Ghana: Problems and Solutions". Presented at the 1992 International Winter Meeting. The American Society of Agricultural Engineers, Nashville.

Allotey, I. A. and Dolan, J.D. (1991). "A built-up timber girder for bridge construction in developing countries". Building and Road Research Institute (BRRI), Kumasi.

American Institute of Timber Construction (AITC). 1994. Timber Construction Manual. Fourth edition. John Wiley & Sons Inc. New York. ISBN 0-471-30970-2.

Anon (1996). Ghana Forest Development Master Plan 1996 – 2020. Ministry of Lands and Forestry, Accra, Ghana.

Arya, C. (2003). Design of Structural Elements. Second Edition. Spon Press – Taylor & Francis Group. London, UK. pp 271.

Ashby, M. F. and Jones, D. R. H. (1980). Engineering Materials 1: An Introduction to their Properties and Applications. Pergamon Press, ISBN 0-08-026138-8

Ashiabor, W. K. (1967). The strength properties of Ghanaian timbers. Tech. Newsletter for.Prod. Res. Inst. Ghana. 1: 14-23.

ASTM D 143-52. Standard Test Methods for Small Clear Specimens of Timber. ASTM International. West Conshohocken, PA.

ASTM D 4761. (1999). Standard Methods for Determining Mechanical properties of lumber and wood-base structural materials. American Society for Testing Materials International. West Conshohocken, PA

Ayensu, E. S. and A. L. Bentum, (1974). Commercial timbers of West Africa. Smithsonian Contributions to Botany, No. 14. 69pp.

Baiden, B.K., Badu, E. and Menz, F.S. (2005). Exploring the barriers to the use and potential of timber for housing construction in Ghana. Journal of Construction and Building Materials. June 2005. Gale Group, farrington Hills, Michigan

Bedford A. And Liechti K.M. (2004). Mechanics of Materials. Prentice Hall, U.S.A. ISBN 7302102279. pp518-534

Bentum , A. L. K. (1969). Properties of Ghana timbers: Kaku (*Lophira alata var. Procera*) Tech. Newsletter For. Prod. Res. Inst. Ghana. 1:27-29.

Bentum, A. L. K. (1970). Properties of Ghana timbers: Dahoma (*Piptadeniastrum africanum* Hook. F.). Tech. Newsletter, For. Prod. Res. Inst. Ghana. 4:19-21.

Bentum, A. L. K. and Addo-Ashong, F. W. (1977). Weathering performance of some Ghanaian timbers. Tech. Bull. For. Prod. Res. Inst. Ghana 26:13pp.

Bolza, E. and Keating, W. G. (1972). African timbers: the properties, uses and Characteristics of 700 species. CSIR, Div. of Build. Res. Australia. pp 699

Bostrom, L. And Holmquist, C. (1999). Determination of the modulus of elasticity in bending of structural timber – Comparison of two methods. SP Report 1999: 17.

Brazier, J.B. (1978). Harvesting mixed tropical timber for paper. Unasylva, Food and Agricultural Organisation (FAO). Volume 30. No. 122, p14.

British Building Research Establishment (BBRE). 1984. Wood Handbook: Digest 417. National Building Specification, Watford, UK.

BS 5268. (2002). Code of Practice for Structural Use of Timber. British Standard Institution.

BS 5268. (1985). Code of Practice for Structural Use of Timber. British Standard Institution.

BS 373. (1957). Methods of Testing Small Clear Specimens of Timber. British Standard Institution.

BS 5820. (1979). Methods of Test for Determination of Certain Physical and Mechanical Properties of Timber in structural sizes. British Standard Institution.

Calil, C.J. (2006). Emerging timber bridge program to Sao Paulo State: A four-year report. 7th World Conference on Timber Engineering, WCTE, Germany, June 2006.

Chen Y., Lam F. and Barrett J.D. (2006). Bending strength and modulus of elasticity of British Columbia coastal timbers. 7th World Conference on Timber Engineering, WCTE, Germany, June 2006.

Chudnoff, M. and Youngs, B.L. (1980). Evaluating concepts for improved utilization of tropical timber resources. In summary I of Studies presented to the fifth Session of the FAO Committee on Forest Development in the tropics. Rome, Italy. Unasylva 32 (127).

Dagadu, P. (2008). State of the Adomi Bridge. An interview granted to journalists of Graphic Communications for Graphic front page report. Graphic front page report "Adomi Bridge Re-opens August 2". July 27, 2008. Graphic Communications, Accra.

Dodge, Y. (2003). The Oxford Dictionary of Statistical Terms. OUP, ISBN 0-19-920613-9

Eddowes P.J. (1980). Technical aspects of marketing unfamiliar species. Paper presented at the 11th Commonwealth Forestry Conference. Trinidad and Tobago.

prEN 14544. (2005). Timber stuctures – Structural timber with round cross-section - requirements. CEN/TC 124, European Committee for Standardisation, Brussels, Belgium.

EN 384. (2004). Structural Timber – Determination of characteristic values of mechanical properties and density. European Committee for Standardisation, Brussels, Belgium.

EN 408. (2003). Timber Structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties. European Committee for Standardisation, Brussels, Belgium.

EN 338. (2003). Structural Timber – Strength Classes. European Committee for Standardisation, Brussels, Belgium.

EN 14251. (2003). Structural round timber – Test methods. European Committee for Standardisation, Brussels, Belgium.

Food and Agriculture Organisation. (1983). Food and Agriculture Organisation Year Book of Forest Products. Forestry Series No. 16, Rome, Italy. FAO Statistics Series 44.

Food and Agriculture Organisation. (1988). Food and Agriculture Organisation Year Book of Forest Products. Forestry Series No. 23, Rome, Italy. pp50-58.

Forest Products Research Laboratory. (1998). Hand book of Hardwoods. Forest Products Research Laboratory, Princes Risborough, U.K.

Forest Products Laboratory. (1999). Wood handbook – Wood as an engineering material. General Technical Report FPL-GTR-113. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison. WI, USA. pp463.

Freezaillah B.C.Y. (1990). Lesser-known tropical wood species – How bright is their future? Malayan Nature Journal. 42.

Freas, A. D., M. et al, (1973). Factors influencing the utilization of tropical wood species. Agency for International Dev. Report, Washington, D.C. No. TA/OST 73-22, pp 31.

Gere J. M. and Timoshenko S.P. (1998). Mechanics of Materials. Third SI Edition. Stanley Thornes (Publishers) Ltd, Cheltenham, U.K. ISBN 0-7487-4084-8

Ghaboussi, J. (1986). Bending of nail-laminated beams of I-section. Engineering Institute of Canada, EIC-70. BR – STR 12, Vol. 13, No. A-8.

Grant, R. G. (2006). The evolution of timber truss road bridges in New South Wales. Paper presented at the 7th World Conference on Timber Engineering (WCTE), Germany, June 2006.

Hall, J. B. and Swane, M. D., (1981). Distribution and ecology of vascular plants in a Tropical rain forest: forest vegetation in Ghana. Geobotany (W.Junk ed.) The Hague. 383pp.

Halperin D.A. and Bible G.T. (1994). Principles of Timber Design for Architects & Builders. John Wiley & Sons Inc. New York. ISBN 0-471-55768-4.

Hansom, O. P. (1983). Promotion of commercially less accepted species. Paper presented at 1st UNIDO/FAO Consultation on the Wood and Wood Products Industry, Helsinki, 19 – 23 September, 1983.

Hashemi, J. and Smith, W.F. (2006). Foundations of Materials Science and Engineering. 4th Edition. McGraw-Hill, 2006. ISBN 007-125690-3.

Hawthorne, W. (1990). Field guide to the forest trees of Ghana. Ghana Forestry Series 1. Overseas Dev. Agency. London, U. K. 276pp

Huang, Z. (2004). Decay of some less-utilised Ghanaian tropical hardwoods using decay By white rot fungi and brown rot fungi. A paper published on the internet. (http://www.irg-wp.com/documents/2004/IRG35Documents.pdf)

Irvine, F. R. (1961). Woody plants of Ghana with special references to their uses. Oxford Univ. Press Ltd. London. U. K. pp 868.

Jayanetti, D. L. 1998. Lesser-Used Timber Species in Construction. Technical Session V. Timber Engineering and Re-constituted Wood. International Conference on Value-Added Processing and Utilisation of Lesser-Used Timber Species. Forestry Research Institute of Ghana (FORIG), International Timber Trade Organisation (ITTO) and Timber Export Development Bureau (TEDB) Proceedings. Pg. 145-156.

Koch, P. (1991). Wood as non-wood materials in US residential construction: Some energy related international implications. Working Paper 36. Food and Agricultural Organisation (FAO), Seattle, WA,USA. Centre for International Trade in Forest Products, University of Washington.

Kollman, F. F. P. and Cote, W. A., (1984). Principles of Wood Science and Technology. Volume 1: Solid Wood. Springer-Verlag, Berlin, Heidelberg, New York, Tokyo, 592pp.

Kretschmann, D.E. and Green D. W. (1999). Wood handbook- Wood as an engineering material. Forest Products Laboratory. General Technical Report FPL-GTR-113. Madison, WI. 463p

Kumi-woode, B. G. (1996). Natural decay resistance of some Ghanian Timbers and wood decay hazard potential for Ghana. Msc. Thesis. Faculty of Foestry. Lakehead University Thunder bay, Ontario. Canada. 98pp.

Lecomte de Mello, R. (2006). Eucalyptus structural poles for medium cost houses in central Brazil. World Conference on Timber Engineering, WCTE, Germany, June 2006.

Lincoln, M. E. (1986). Think and Explain with Statistics. Addison-Wesley Inc. pp.1-3

Madsen, B. (1992). Structural Behaviour of Timber – Size effects. Timber Engineering Ltd. North Vancouver, B.C. Canada. ISBN 0-9696162-0-1

Moor, H. W. (1940). Gold Coast Timber Industry. Nigeria Forestry Research Institute. Journal 1:3-8.

Nyarko G. (1999). Treatability and Durability of Three Lesser Utilised Ghanaian Hardwood Timber Species. A BSc. thesis presented to the Department of Wood Science and Technology, Kwame Nkrumah University of Science & Technology, Kumasi. 57pp.

Ocloo, J. H. and Laing, E. (1991). Anatomical properties of the wood of some Celtis Sp. Indigenous to Ghana. Disc. And Innov. 3:89-98.

Oduro, J. and Gotah, E. (2008). Wood Basics. Hand out for an advanced training in roof construction. Wood Industries Training Centre (WITC), Kumasi.

Ofori, J. (1985). A comparative study in minimising the splitting of plantation grown Emire, *Terminalia ivorensis* A. Chev. Fencing posts during drying. Tech. Bull. For. Prod. Res. Inst., Ghana. 5:1-8.

Ofori, J. (1987). Corrosion of metal fasteners in contact with Kusia *(Nauclea diderrichi)* Stakes. Tech. Bull. For. Res. Inst., Ghana. 3:46-52.

Okoh, I. K. A. (1977b). Moisture dependence of DC electrical reistivity of some Ghanaian Woods. Tech. Bull. For. Prod. Res. Inst., Ghana. 1:33-41.

Oteng-Amoako, A. A., Ebanyenle, E. and Awuku, F. A. (1998). The identification of 14 Under-exploited promotable lesser-used timber species of Ghana. International Conference on Value-Added processing and Utilisation of Lesser-used Timber species. Forest Research Institute of Ghana (FORIG), International Timber Trade Organisation (ITTO) and Timber Export Development Bureau (TEDB) Proceedings. Pg. 77-85.

Oteng-Amoako, O.A. (2006). 100 Tropical African Timber Trees from Ghana – Tree description and wood identification with notes on Distribution, Ecology, Silviculture, Ethnobotany and Wood uses. Graphic Packaging, Accra, Ghana. ISBN 9988-7943-4-7

Panshin, A. J. and de Zeeuw, C. (1980). Textbook of Wood Technology. Vol. 1 4th Edition. New York. McGraw-Hill Books company. Inc. New York. Pg. 185-200.

Quartey, G., Frimpong-Mensah, K. and Zuercher, E. (2008). "On natural durability of lesser known West African Species: Results from a Soil-block test and some relations with sorption properties, density, mechanical properties and anatomical properties", International Conference *IAWA*, Mexico, July 2007.

Shames, I.H. and Cozzarelli F.A. (1991). Elastic and inelastic stress analysis. Prentice Hall. ISBN 1-56032-686-7

Solli, K.H. (1999). Modulus of elasticity – local or global values. CIB W18 paper 33-10-3.

Solli, K.H. (1996). Determination of modulus of elasticity in bending according to EN 408. CIB W18 Meeting 29 paper 29-10-3.

Stalnaker, J. J. and Harris, E. C. (1989). Structural design in wood. Structural Engineering Series. Van Nostrand Reinhold, NY, USA. pp20-31.

Timoshenko S. P. (1976). Strength of Materials. 3rd Edition. Krieger Publishing Company, ISBN 0-88275-420-3

TIDD. (1998). Norms and regulations for operations of timber processing firms. Timber Industry Development Division, Forestry Commission, Ghana.

TRADA. 1985. Timbers of the World. Timber Research and Development Association (TRADA), High Wycombe, U.K.

Usher, M. B. and Ocloo, J. K. (1979). The natural resistance of Eighty-five West African hardwood timbers to attack by termites and microorganisms. The Tropical Pest Bulletin No.5, Natural Resources Institute. ISBN 10:0851351034. December 1979.

Vries de P., Gard W.F, Schuch R. (2006). Development of a strength grading system of small diameter round wood. World Conference on Timber Engineering, WCTE, Germany, June 2006.

Vries de P. and Gard, W.F. (1998). Strength Grading for small diameter roundwood. Heron, Volume 43. No.4, pp. 183-197.

Wuethrich, K. (2008). Local Wood Processing Technology in Kumasi, Ghana. Report submitted for the Timber Bridges Conference organised by Kwame Nkrumah University of Science & Technology(KNUST) and Bern University of Applied Sciences (BFH) at the Engineers Centre, Accra. September, 2008.

Youngs, R.L. (1977). Research in tropical wood utilization. Unasylva 29 (117), pp9-11.

APPENDIX I

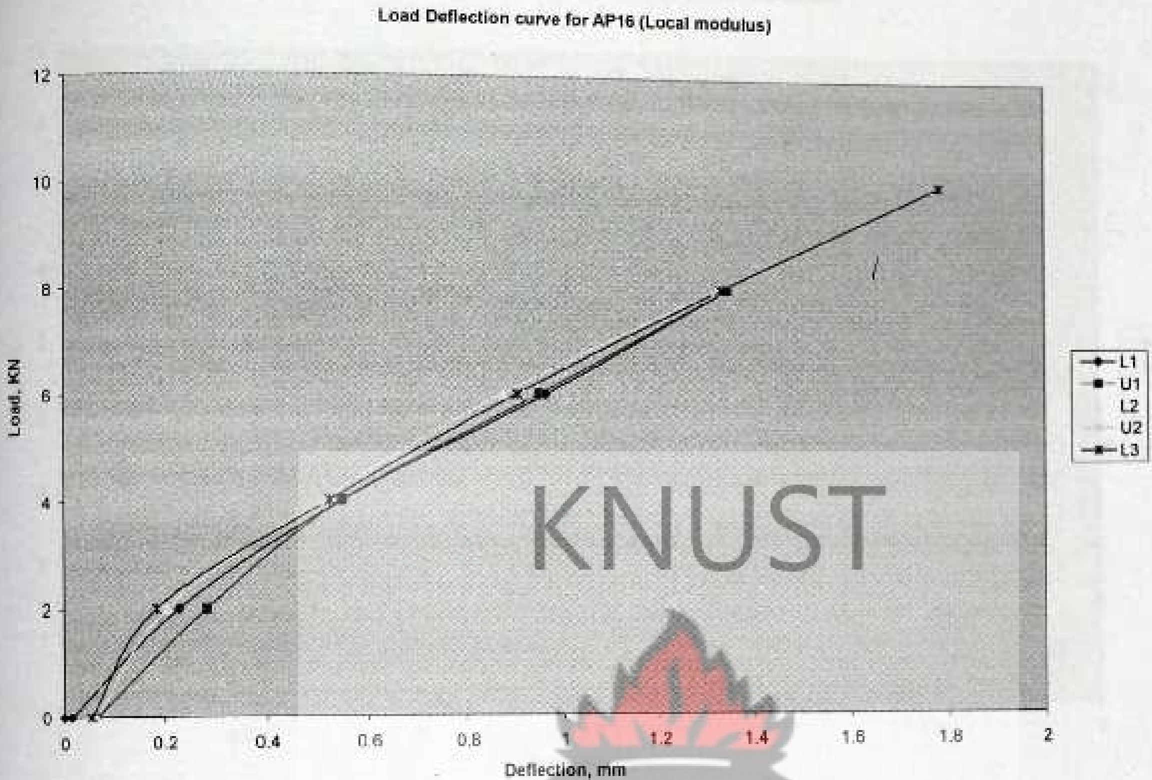


Fig. I-1a: Load Deflection curve for AP16 (Local modulus)

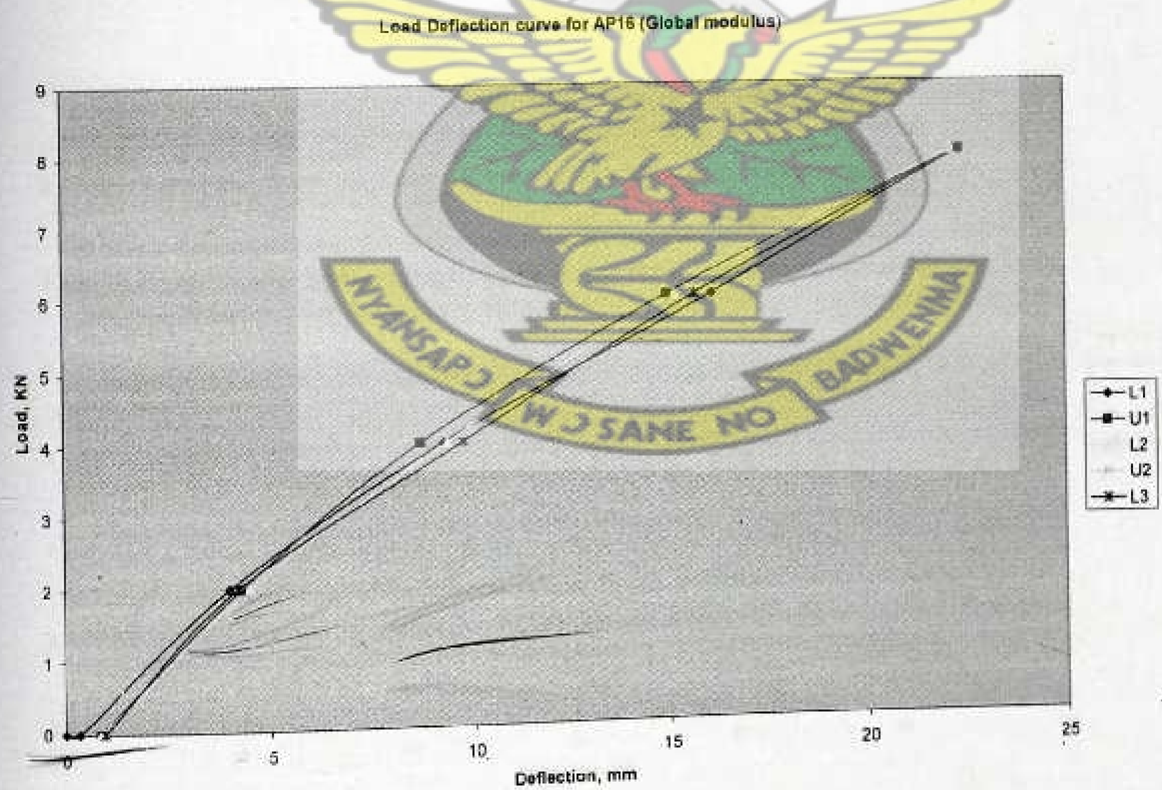


Fig. I-1b: Load Deflection curve for AP16 (Global modulus)

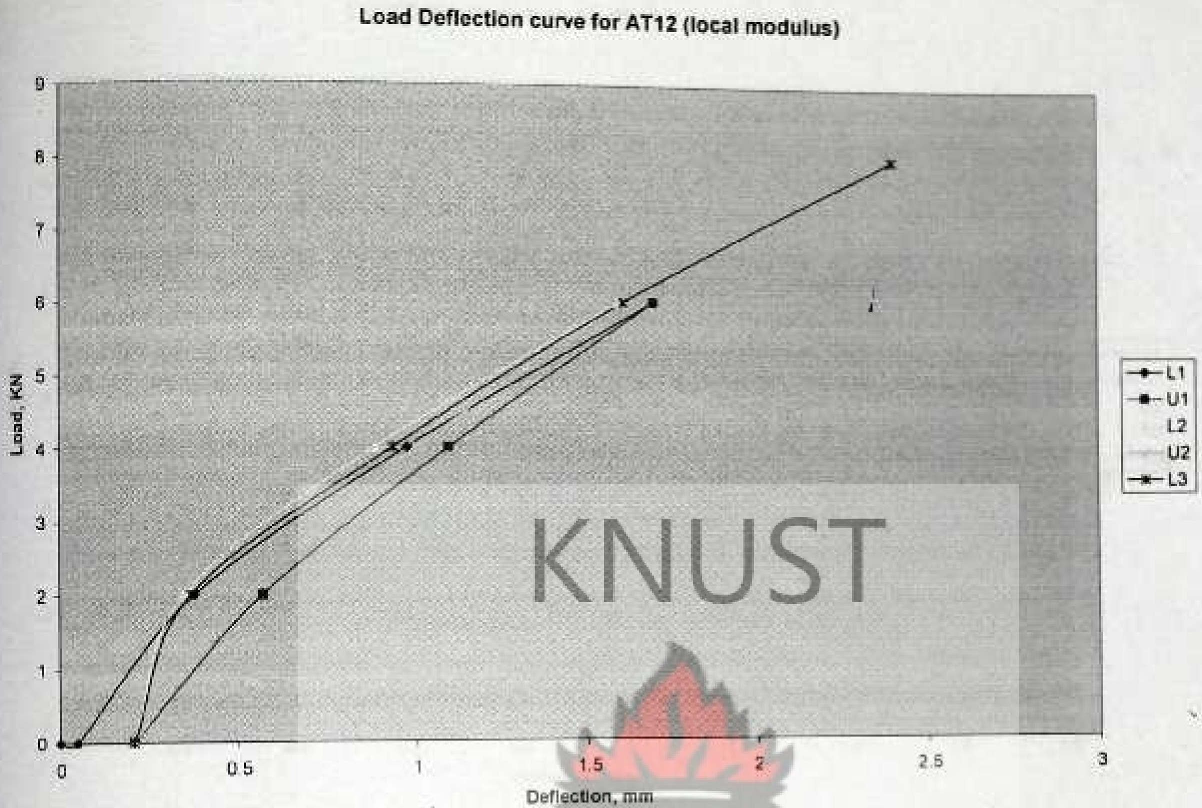


Fig. I-2a: Load Deflection curve for AT12 (Local modulus)

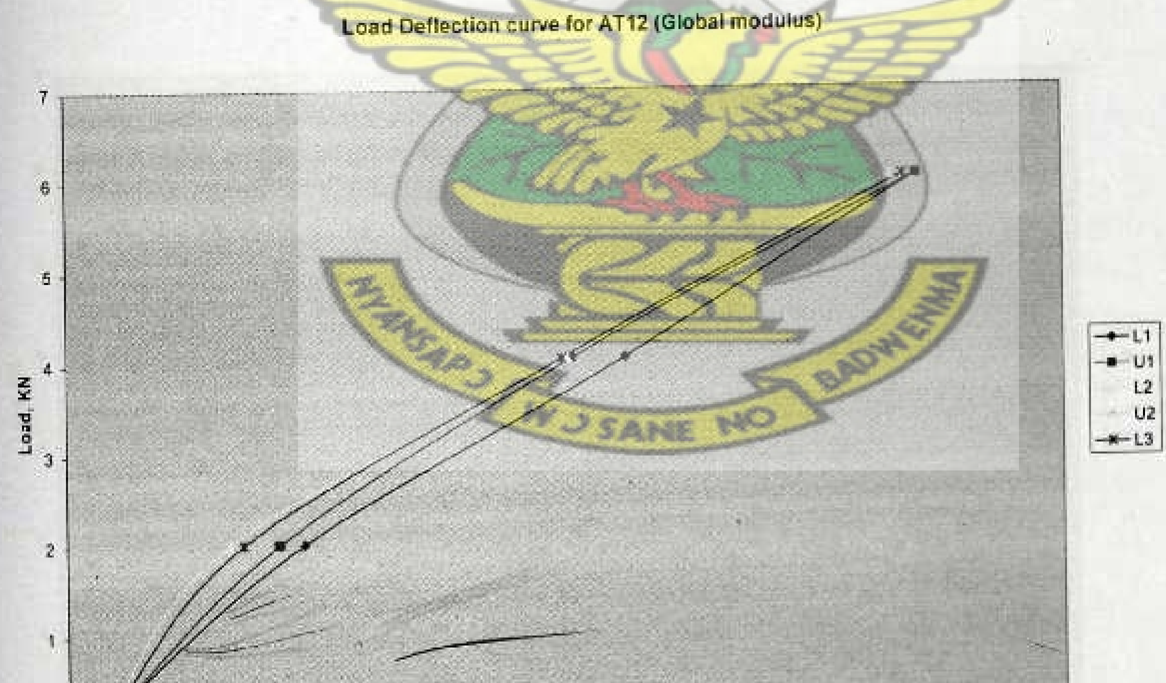


Fig. I-2b: Load Deflection curve for AT12 (Global modulus)

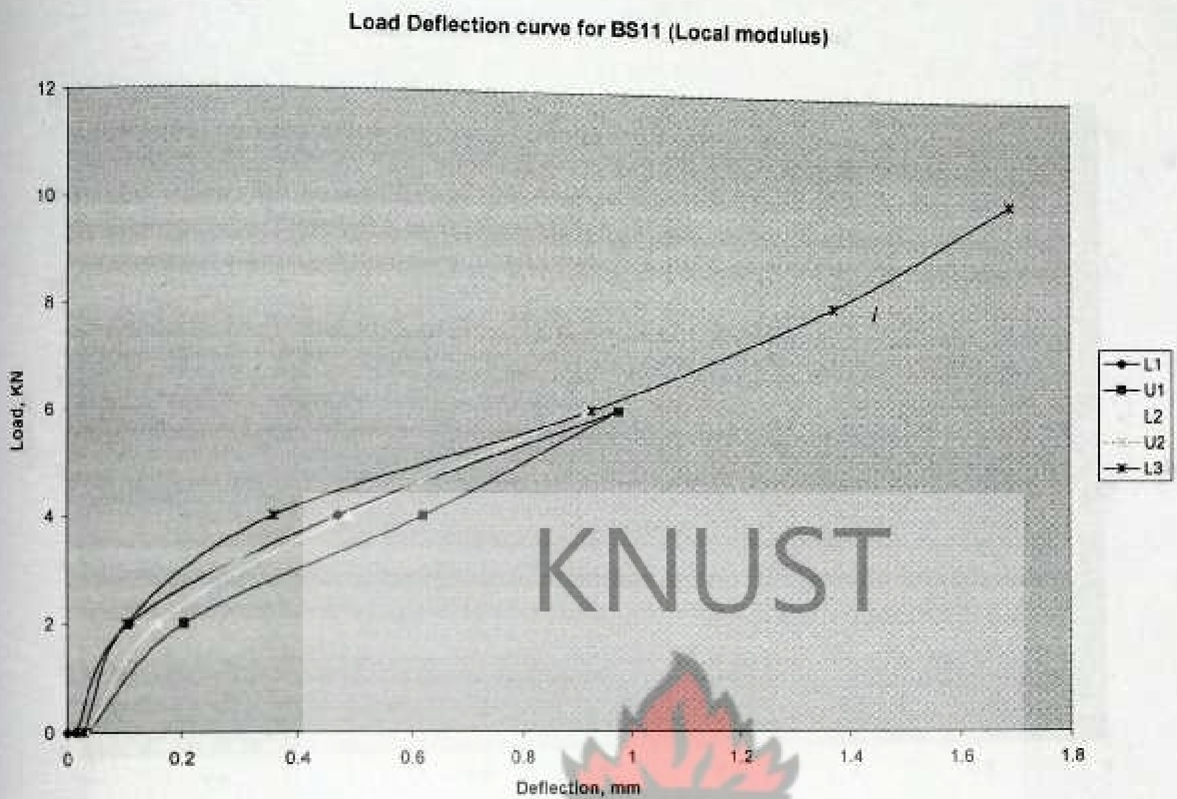


Fig. I-3a: Load Deflection curve for BS11 (Local modulus)

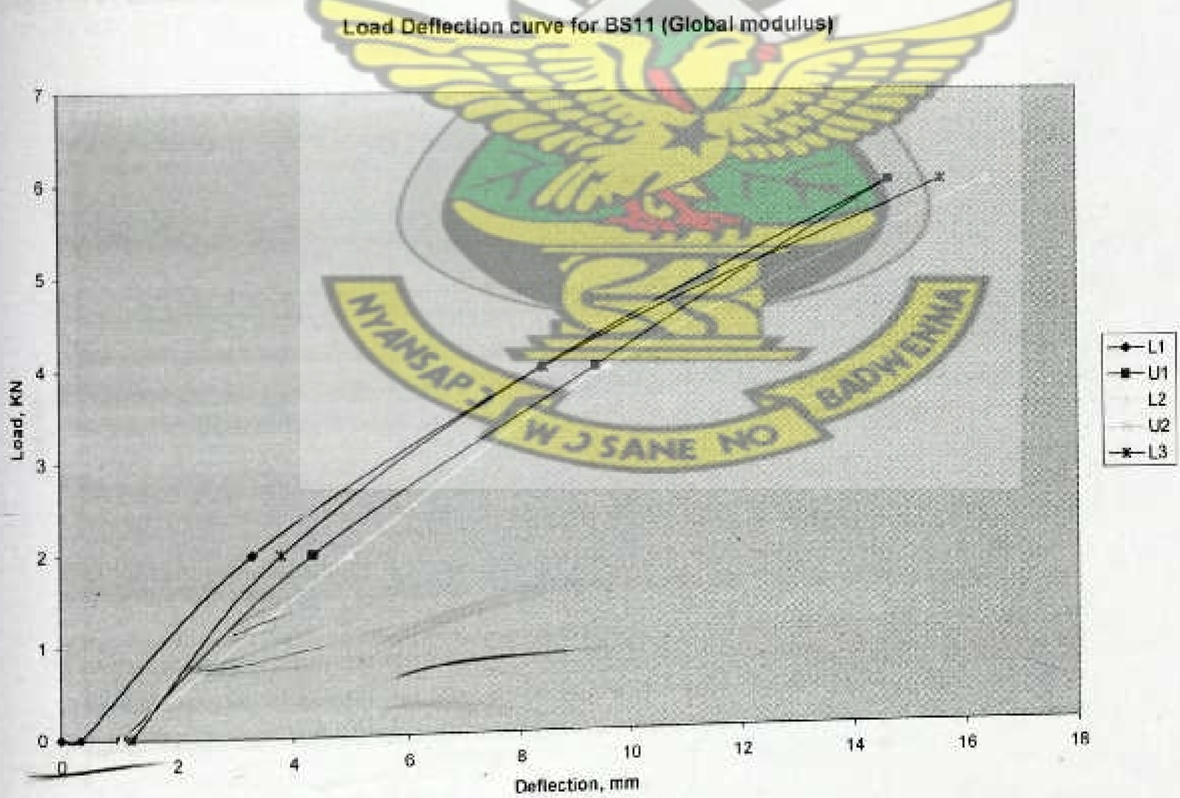


Fig. I-3b: Load Deflection curve for BS11 (Global modulus)

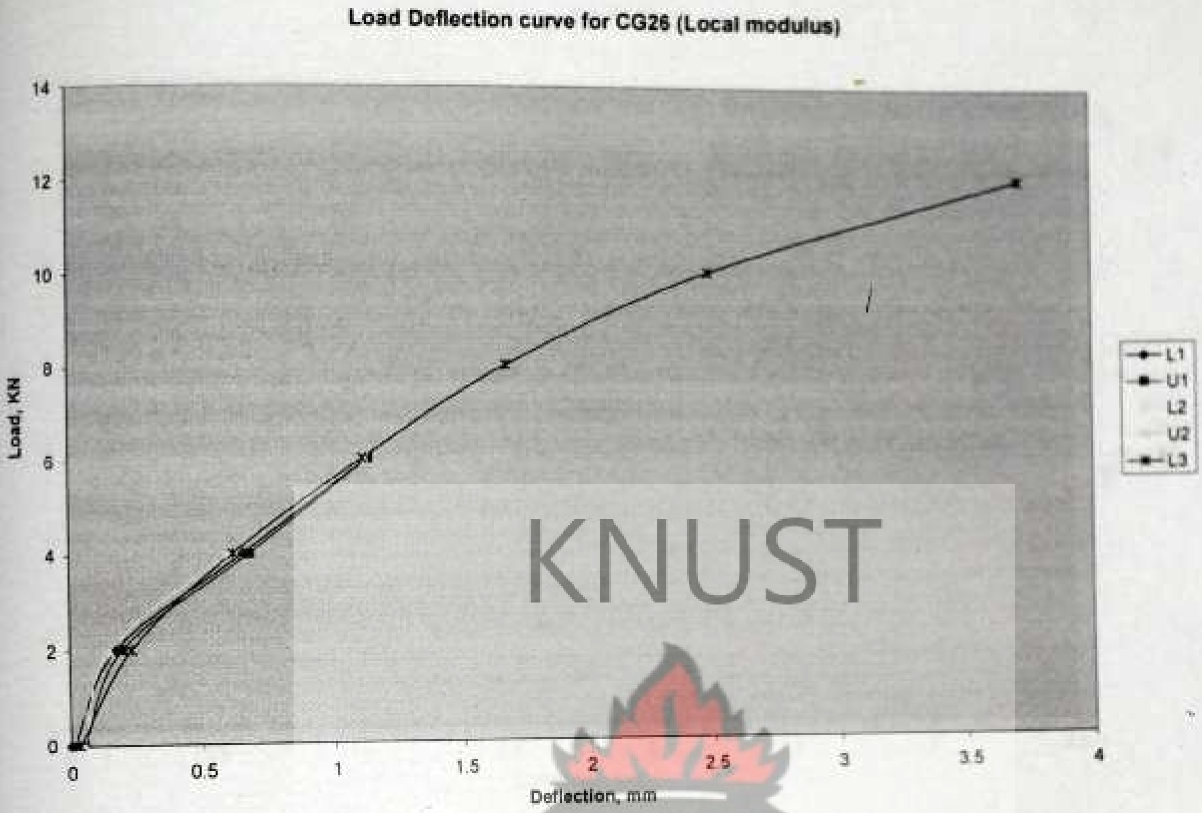


Fig. I-4a: Load Deflection curve for CG26 (Local modulus)

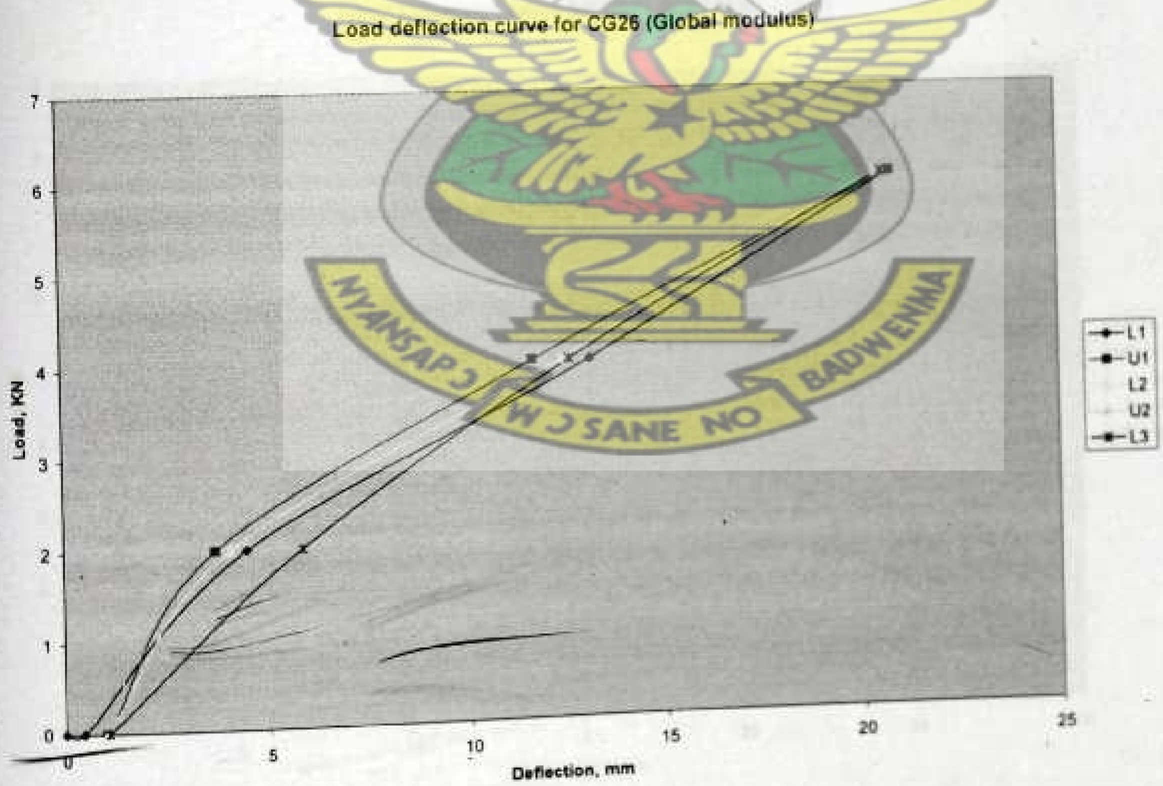


Fig. I-4b: Load Deflection curve for CG26 (Global modulus)

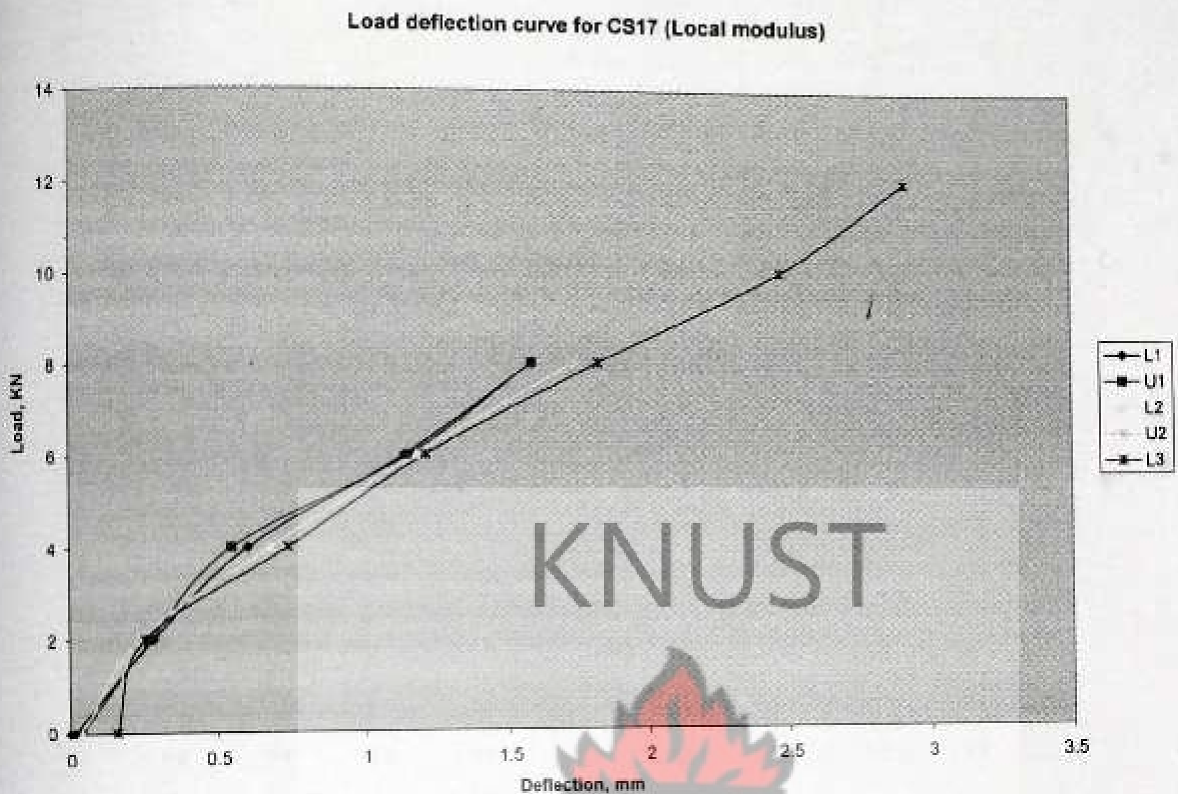


Fig. I-5a: Load Deflection curve for CS17 (Local modulus)

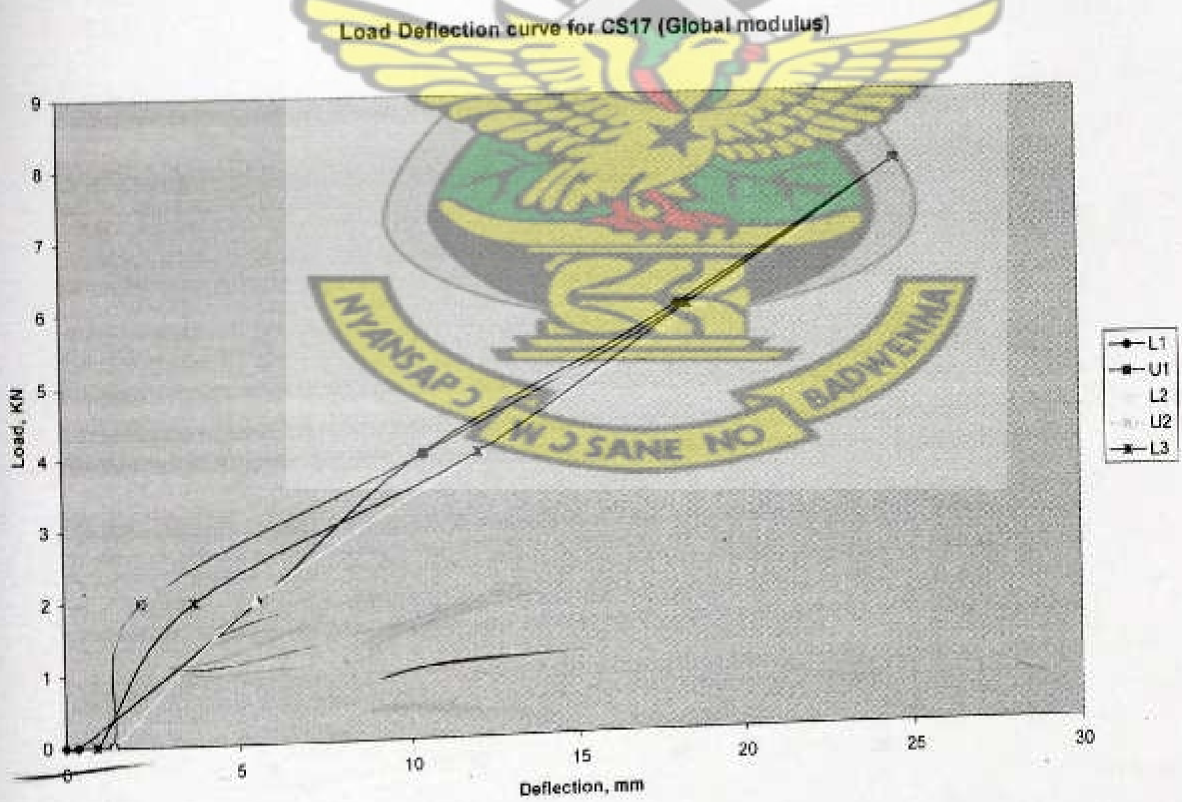


Fig. I-5b: Load Deflection curve for CS17 (Global modulus)

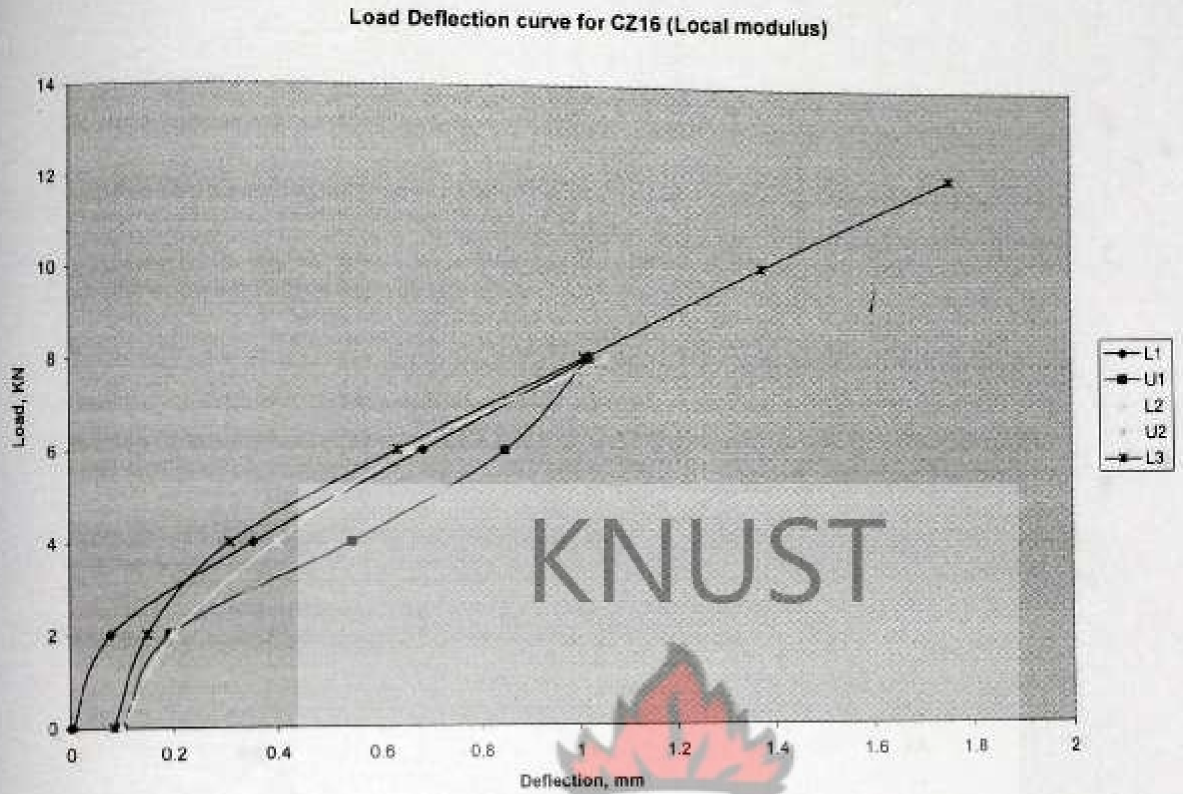


Fig. I-6a: Load Deflection curve for CZ16 (Local modulus)

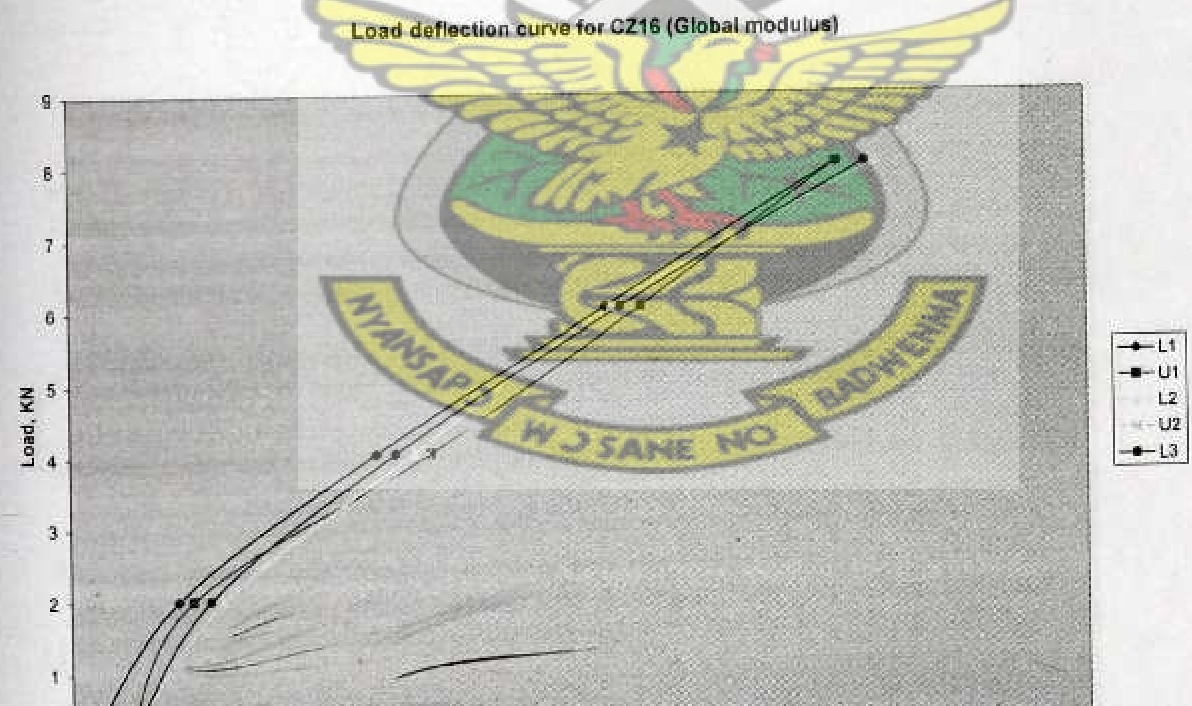


Fig. I-6b: Load Deflection curve for CZ16 (Global modulus)

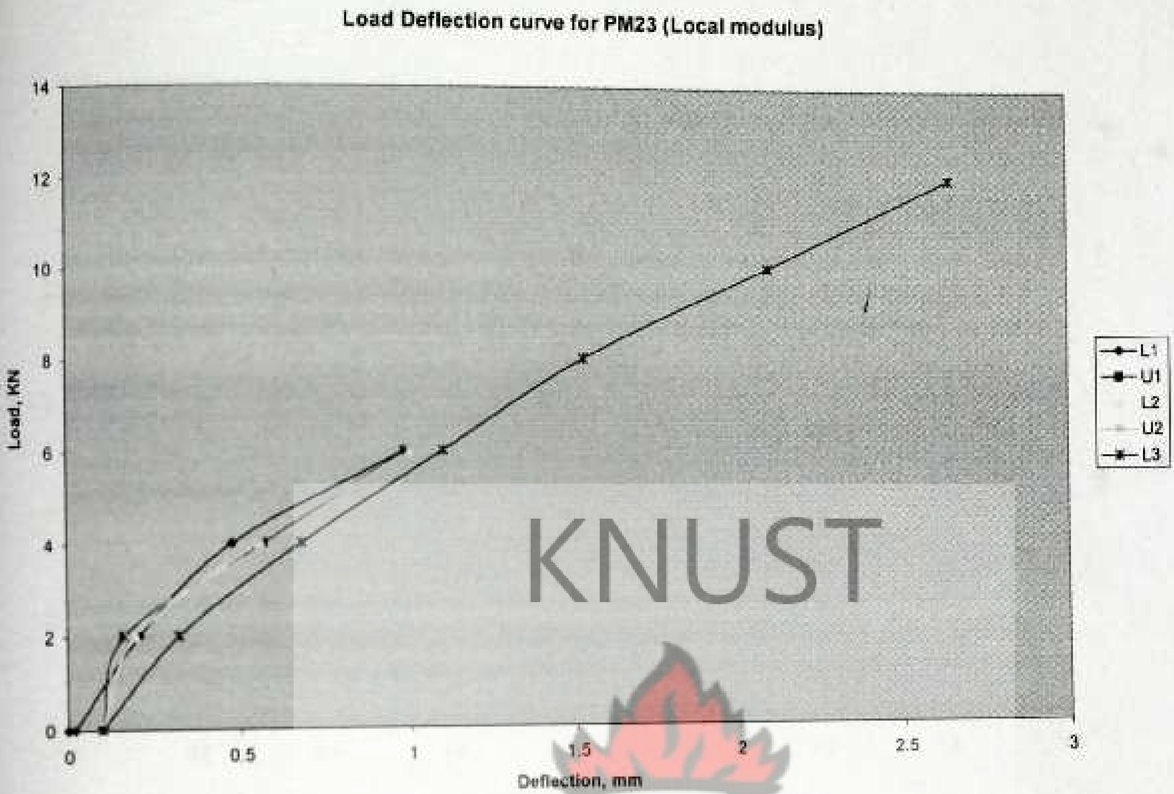


Fig. I-7a: Load Deflection curve for PM23 (Local modulus)

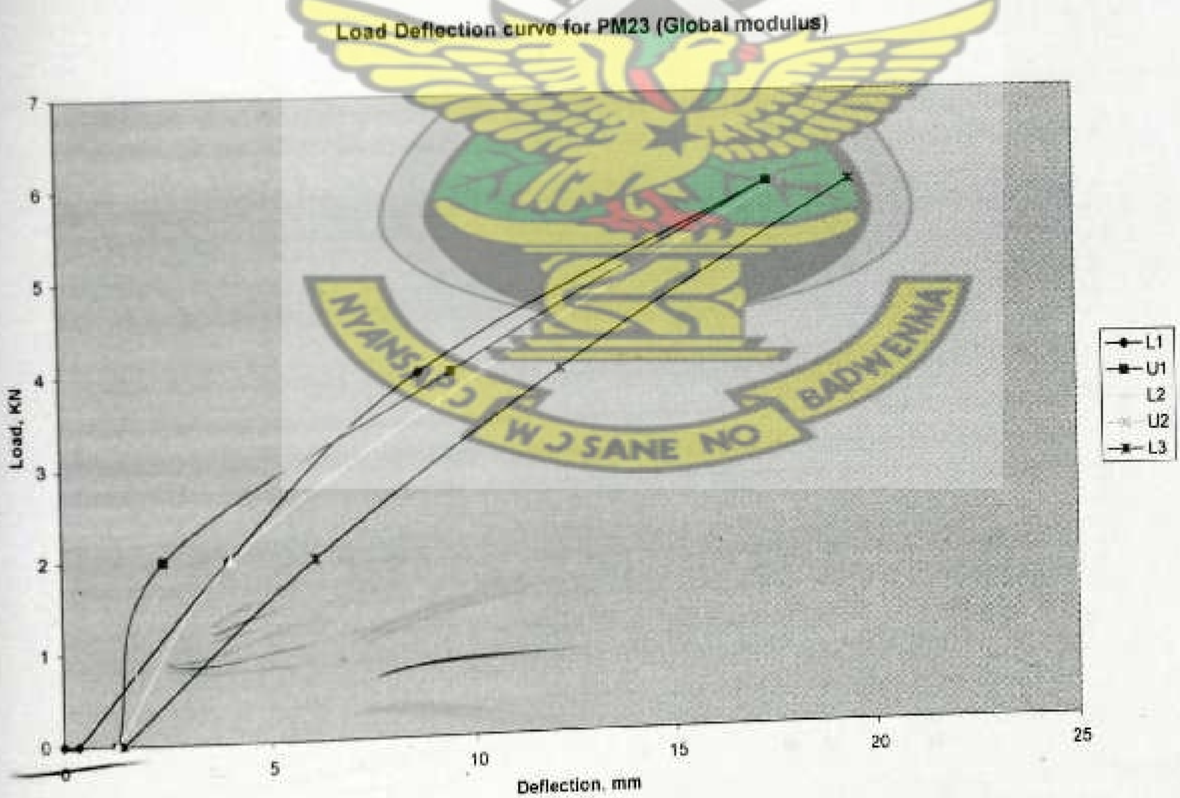


Fig. I-7b: Load Deflection curve for PM23 (Global modulus)

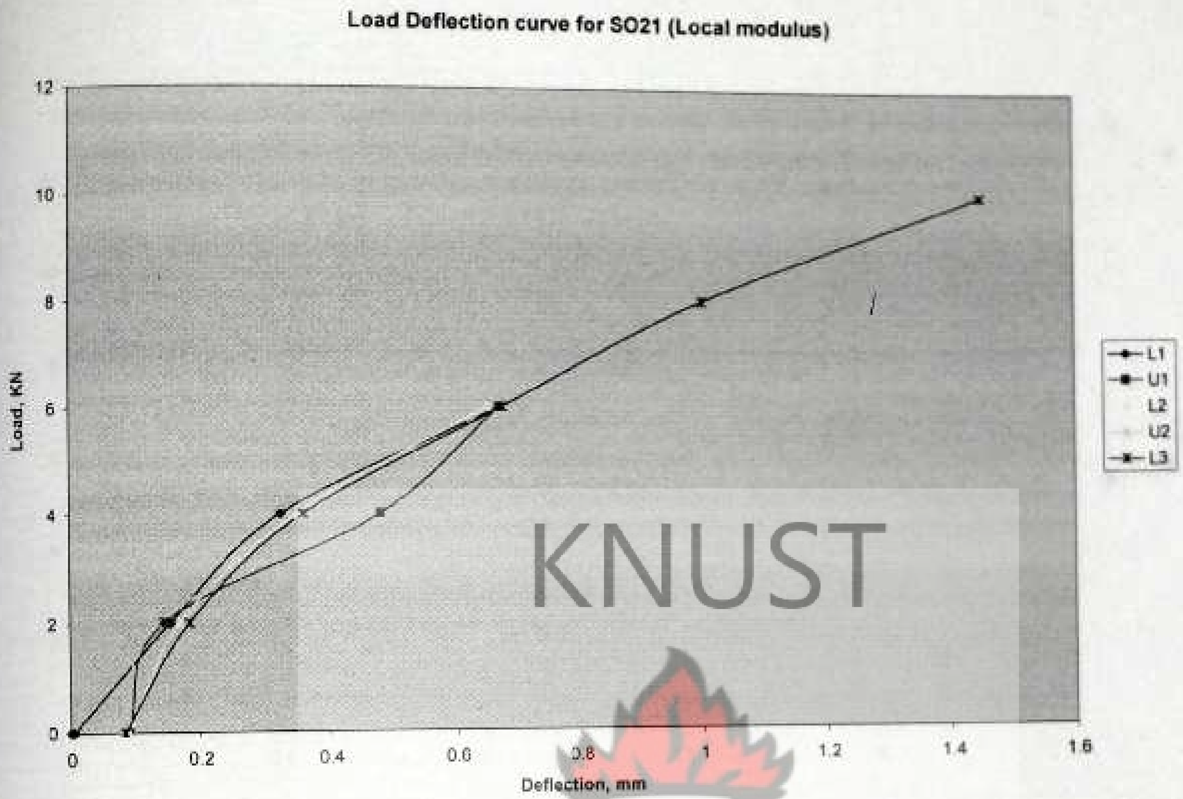


Fig. 1-8a: Load Deflection curve for SO21 (Local modulus)

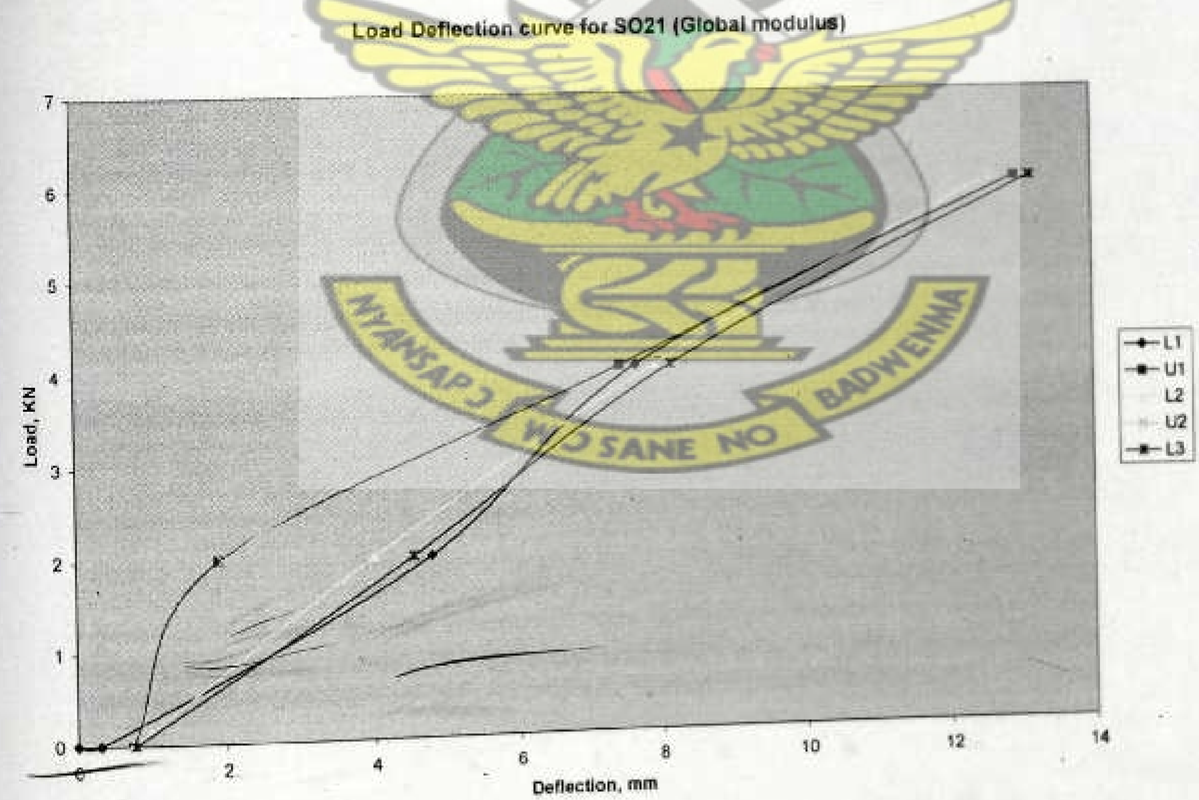


Fig. 1-8b: Load Deflection curve for SO21 (Global modulus)

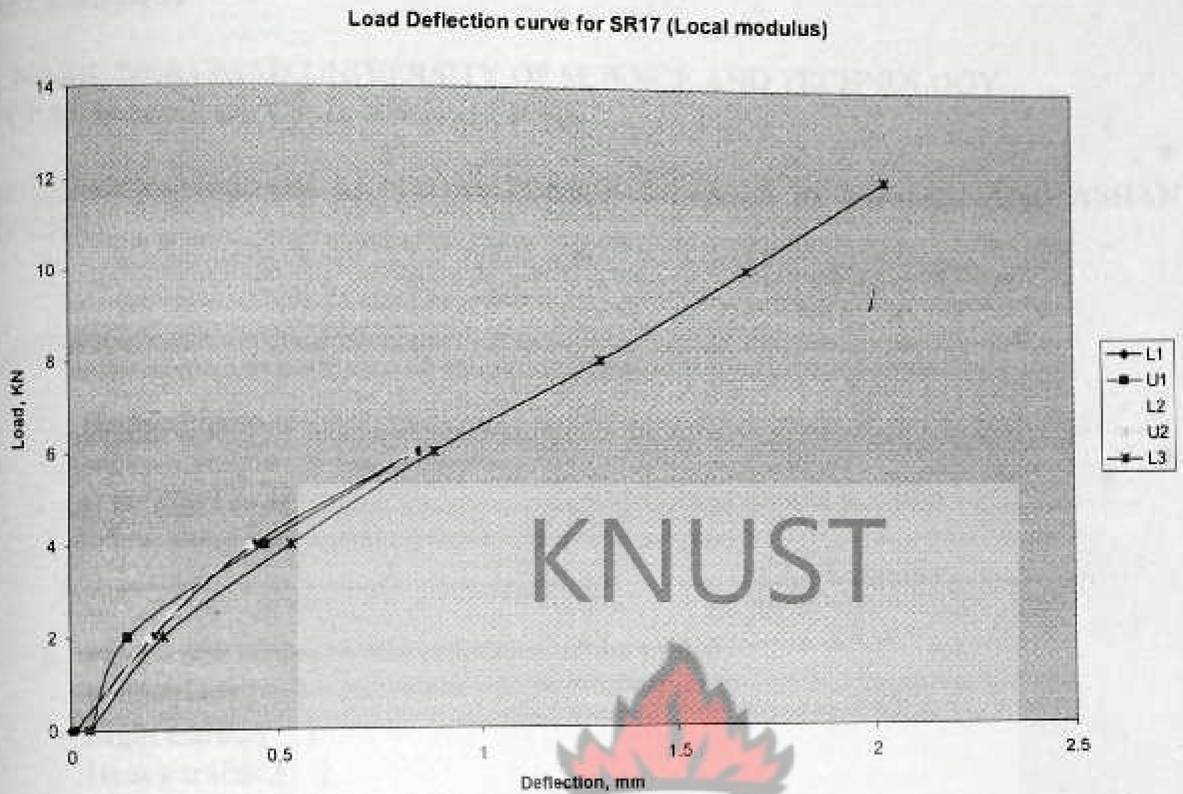


Fig. I-9a: Load Deflection curve for SR17 (Local modulus)

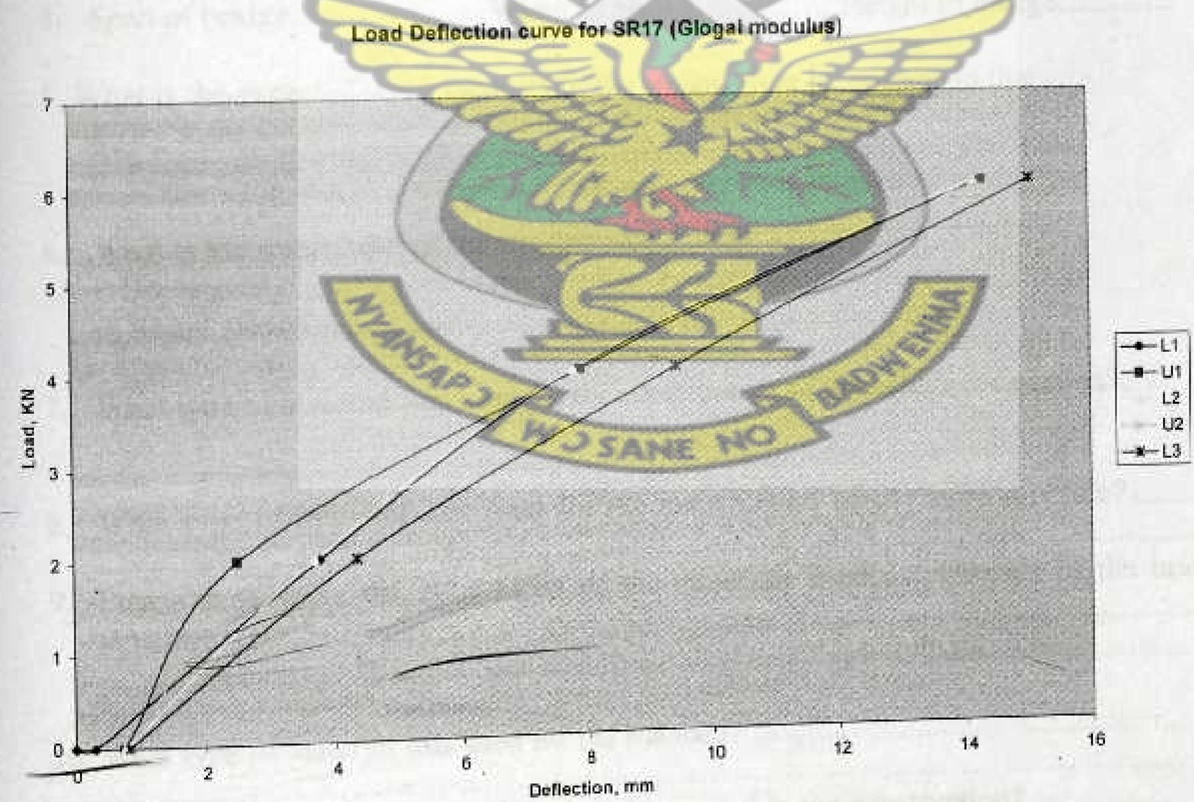


Fig. I-9b: Load Deflection curve for SR17 (Global modulus)

APPENDIX II

KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

QUESTIONNAIRE ON EXISTING TIMBER BRIDGES IN KUMASI AND ASHANTI REGION

PART I

1. Bridge Name……………………………………………………………………………

2. (a) Bridge Location…………………… (b) Geographical position………………

 (c) Terrain…………………………………………… (d) Air humidity………………

3. Please tick as apply, the utilization of the bridge.
 Pedestrians [ ]
 Light traffic [ ]
 Heavy traffic [ ]
 Other [ ] Please specify……………………………………

4. Span of bridge……………… Width of bridge………… Height of bridge……….

5. What is the expected water quantity of the stream/river by volume in the;
 i. Dry season………..
 ii. Rainy season…………

6. What is the water level of the stream/river in the;
 i. Dry season…………
 ii. Rainy season…………

7. What type of material was used for the construction of the bridge foundation?.......
…………………………………………………………………………………………

8. What type of material was used for the construction of the superstructure?............
…………………………………………………………………………………………

9. Please state some the dimensions of the materials used as elements of the bridge structure?……………………………………………………………………………
…………………………………………………………………………………………

10. What type of material was used for the roadbed?...
…………………………………………………………………………………………

11. What type of bearings and connections were used in the construction? ....
…………………………………………………………………………………………
…………………………………………………………………………………………

12. (a) Was there any preservation of the materials used in the construction?
Yes [ ] No [ ]

(b) If yes, state the type of material or chemical used in the preservation............
....
....

13. Has the bridge any security features like guardrails, sidewalks etc?
Please specify..
....
....

14. What type of equipments was used for the assembling of the bridge?...............
....
....

PART II

15. In which year was the bridge constructed?..

16. Which institution is responsible for the repairs and maintenance of the Bridge?
....
....

17. Please tick as appropriate the general assessment of the bridge:
Good [ ] Poor [ ] Dangerous [ ] Safe [ ]

18. What type of repair measures are routinely undertaken and at what intervals?......
....
....
....
....
....

19. Are there any weak spots? Specify if any..
....
....
....

20. What type of measures would you recommend to improve durability and reduce maintenance?..
....
....
....

APPENDIX III

KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

QUESTIONNAIRE FOR WORK ON EXISTING SAWMILLS IN KUMASI AND ASHANTI REGION

2. Sawmill Name/Identification……………………………………………………………

3. Sawmill Location……………………………………………………………………

4. a. List of machinery used

	Response	
Machine	New	Used
Band Mill		
Wood Mizer		
Cross cut saw		
Planner		
Kiln Dryer		
Edger		
Ripper		

b. How long have you been using them? ……………………………………
……………………………………………………………………
c. How often do they break down? ……………………………………………
……………………………………………………………………
d. How available are the spare parts for repairs? ……………………………
……………………………………………………………………
……………………………………………………………………
e. Do you have maintenance department? ……………………………………

4a. Do you process lumber for the local market? YES [ ] NO [ ]

4b. If you answered YES to question 4a, please give reason?

Reason	Response
Easy to transport to local market	
Not expensive in handling	
It is a regulation by TIDD	
It is a ready market	

4c. If you answered NO to question 4a, please give reason?

Reason	Response
Low patronage from local users	
Production cost is expensive for local market	
High demand from exports	
Difficulty in transport from sawmill to markets	
Local market is not readily available	

5a. Do you process lumber for export? YES [ ] NO [ ]

5b. If you answered YES to question 5a, please give reason

Reason	Response
Foreign exchange	
A lot of profit	
Production cost is expensive for local market	
Local market is not readily available	

5c. If you answered NO to question 5a, please give reason

Reason	Response
Difficulty in obtaining grades required	
Difficulty in obtaining logs from forest	
Low processing capacity to meet deadlines	

6. What volume (%) do you produce for the local market?..

7. What volume (%) do you produce for export?..
8. Do you treat (preserve) the wood produced for the local market?..............................
....

9. Product quality

   a. Do you know about the grading (grades) of lumber?

   b. ~~If Yes~~, what type ~~of grades~~ do you make?

Grade	Response
FAS	
No.1 C&S	
No.2 C&S	

c. How do you measure those grades? ……………………………………………
…………………………………………………………………………………………

d. Where and how do you store your finished products? ……………………
…………………………………………………………………………………………
…………………………………………………………………………………………
…………………………………………………………………………

e. At what moisture content do you saw and store your products respectively?……………………………………………………………………
…………………………………………………………………

10. What is the maximum size of round wood for cross cut (length and diameter)?
…………………………………………………………………………………

11a. What are your standard products? ……………………………………………
…………………………………………………………………………………………
…………………………………………………………………………………………

11b. What lumber sizes do you normally produce or process

Sizes	Response
2" x 2"	
2" x 4"	
2" x 6"	
3" x 3"	
3" x 4"	
4" x 4"	
4" x 6"	
6" x 6"	
6" x 8"	

12. What is the cutting cost per m^3? ……………………………………………
…………………………………………………………………………………………

13. How much round wood do you have in stock? ……………………………………

14. What species are they?……………………………………………………………
…………………………………………………………………………………………

15. Are you ready to process secondary species for the construction of bridges in the local industry? ……………………………………………………………………

16. Any comment on the use of timber for bridge construction?……………………
…………………………………………………………………………………………
…………………………………………………………………………………………

APPENDIX IV

KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

QUESTIONNAIRE ON EXISTING CARPENTRIES IN KUMASI AND ASHANTI REGION

General Information

1. Carpentry Name/Identification...

2. Carpentry Location...

3. a. List of machinery/tools used

	Response	
Machine	New	Used
Spirit levels		
Portable Drills		
Cross cut saw		
Planner		
Chisels		
Hand saws		
Builder's square		

b. How long have you been using them?...
....
c. How often do you replace or repair them? ....
d. How available are the spare parts for repairs? ....
....
e. Do you have maintenance section? ....

4. Product quality
   a. Do you know about the grading (grades) of lumber?

   b. If Yes, what type of grades do you work with?

Grade	Response
FAS	
No.1 C&S	
No.2 C&S	

   c. How do you determine those grades? ....

d. Where and how do you store your finished products? ....

e. At what moisture content do you saw and store your products respectively?..

8. What sizes of lumber do you normally work with?

Sizes	Response
2" x 2"	
2" x 4"	
2" x 6"	
3" x 3"	
3" x 4"	
4" x 4"	
4" x 6"	
6" x 6"	
6" x 8"	

9a. Do you obtain your lumber from Sawmill? YES [ ] NO [ ]

9b. If you answered YES to question 9a, please give reason?

Reason	Response
Wood is of good quality	
Guarantee of the wood grade	
Wood from sawmill is easy to work with	
Clients prefer sawmill wood	
It is illegal to obtain from other sources	
Sawmill is close to workshop	

9c. If you answered NO to question 9a, please give reason?

Reason	Response
Too expensive	
Sawmills are too far from workshop	
Bureaucracy and security checks at sawmill	
Difficulty in transport from sawmill to markets/workshop	

10a. Have you ever worked on the construction of a timber bridge?

YES [ ] NO [ ]

10b. If you answered NO to question 10a, please give reason?

Reason	Response
Don't have the skills for bridge	

construction	
Never got the opportunity	
Don't have the required tools for bridge construction	

10c. If you answered YES to question 10a, please indicate the type (span and width)

Span	Response	Width	Response
< 5m		< 1m	
About 5m		1m – 1.5m	
5m – 10m		1.5 – 2m	
10m – 15m		2m – 2.5m	
15m – 20m		2.5m – 3m	
> 20m		> 3m	

11. What species do you work with usually?..
....

12. Are you ready to work with other secondary species in the construction of timber bridges? YES [ ] NO [ ]

13. Do you have the skills to construct timber bridges? YES [ ] NO [ ]

14a. Have you had any form of education or technical training? YES [ ] NO [ ]

14b. If you answered YES to question 14a, please indicate your level of education

Educational level	Response
J.H.S	
S.H.S/S.T.S	
NVTI	
Tertiary	

14c. If you answered NO to question 14a, please give reason

Reason	Response
Lack of funding	
Ready apprenticeship	
Family trade	
Have natural skill in carpentry	

15. Any comment on the construction of timber bridges?..
....
....

APPENDIX V

TENSILE STRENGTH TEST RESULTS SHEETS

Zwick / Roell Standardprotokoll

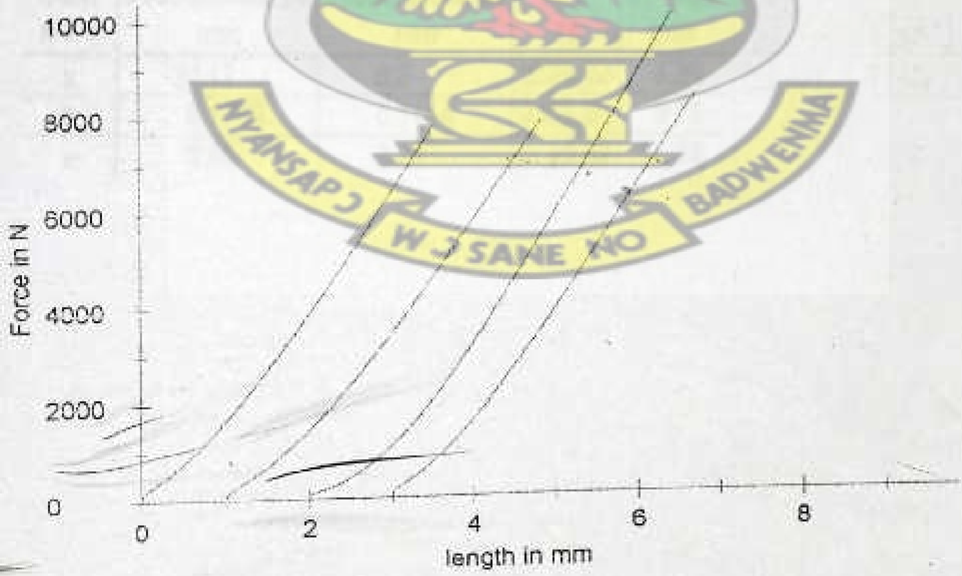
Parametertabelle:

Kunde	: Ghana-Projekt
Prüfer	: Emmanuel Appiah-Kubi rrd1
Prüfnorm	: DIN52188
Material	: AF1_1-3
Kraftaufnehmer	:
Wegaufnehmer	:
Probenhalter	:
Maschinendaten	: 1454MO WN:073436
	Traversenwegaufnehmer WN:073436
	Kraftsensor ID:0 WN:082109 20 kN

Ergebnisse:

Nr	Probenbraite b0 mm	Probendicke a0 mm	Rm N	ε-F max mm	(mm²) A = a·b	Tensile stress (N/mm²)
1	20.2	6.1	7795.31	3.54	123.22	[illegible]
2	20.2	6.1	7859.68	3.85	123.22	[illegible]
3	20.1	6.1	10114.65	4.46	122.61	[illegible]
4	20.3	6	8370.25	3.73	121.8	[illegible]

Seriengrafik:



Zwick / Roell Standardprotokoll 22.01.2007

Statistik:

Serie n = 4	Probenbreite b0 mm	Probendicke a0 mm	Rm N	ε-F max mm	A = a·b (mm²)	Tensile Stress N/mm²
x	20.2	6.075	8534.97	3.89	122.72	69.5
s	0.08165	0.05	1084.07	0.40		
v	0.40	0.82	12.70	10.23		

Zwick / Roell Standardprotokoll

Statistik:

Serie n = 4	Probenbreite b0 mm	Probendicke a0 mm	Rm N	ε-F max mm	A = a·b (mm²)	Tensile stress (N/mm²)
x	20.17	6.2	9334.24	4.36	125.05	74.6
s	0.1708	0.1414	2335.02	0.75		
v	0.85	2.28	25.02	17.28		

Zwick / Roell **Standardprotokoll** **22.01.2007**

Parametertabelle:

Kunde : Ghana-Projekt
Prüfer : Emmanuel Appiah-Kubi rrd1
Prüfnorm : DIN52188
Material : BS1_1-4
Kraftaufnehmer :
Wegaufnehmer :
Probenhalter :
Maschinendaten : 1454MO WN:073436
Traversenwegaufnehmer WN:073436
Kraftsensor ID:0 WN:082109 20 kN

Ergebnisse:

Nr	Probenbreite b0 mm	Probendicke a0 mm	Rm N	ε-F max mm	Area (mm²)	Tensile Stress N/mm²
1	20.4	6.4	7164.23	4.06	130.56	54.9
2	20.2	6.1	11430.31	5.14	123.22	92.8
3	20.1	6.2	7467.04	3.49	124.62	59.9
4	20	6.1	11275.38	4.83	122	92.4

Seriengrafik: