INVESTIGATION OF *LANDOLPHIA BUCHANANII* STEM AS A CONCRETE REINFORCEMENT MATERIAL

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DECLARATION

This thesis is my original work and has not been submitted to any other university for examination.

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DEDICATION

To my parents and my siblings for their continual support.

ABSTRACT

The use of conventional materials in construction of buildings is facing two main challenges of high cost and large scale depletion of the source of the materials thus creating environmental problem. This challenges demand that alternative materials be explored that are not just affordable but are also environmentally friendly. In this regard, *Landolphia buchananii* is proposed as a possible concrete reinforcement material. This research study is on *Landolphia buchananii* as an alternative material for concrete reinforcement.

In this research , stems of *Landolphia buchananii* plant found in Nandi forest in western Kenya was used to reinforced concrete beams which were then subjected to flexural test. Other physical and mechanical tests like moisture content, density, tensile, compression, shear, hardness, bending and pull out test of *Landolphia buchananii* and steel in concrete were also done in order to understand the basic properties of *Landolphia buchananii* as a reinforcement material. Concrete cubes compression test was also conducted in order to ensure that the concrete met the required target strength.

The mean strengths of dry *Landolphia buchananii* was determined as follows; tensile strength as 87.2 N/mm^2, shear strength as 9.8 N/mm^2, compression strength as 23.7 N/mm^2, hardness as 3666.5 N, bending strength as 47.8 N/mm^2 and modulus of elasticity as 3349 N/mm^2. The average compressive strength of the concrete was 20.8 N/mm^2. The average interfacial bond strength of *Landolphia buchananii* in concrete was determined as 1.15 N/mm^2 which was around 13 % that of steel which was 8.56 N/mm^2 in concrete which was at the age of 28 days. The highest bending strength of *Landolphia buchananii* reinforced concrete beams was 2.82 N/mm^2 which was corresponding to the largest area of reinforcement of *Landolphia buchananii* which was 2.14% of the cross section area of the beams casted. The results showed that the bending strength increased with increase in area of reinforcements of *Landolphia buchananii.*

It was concluded that *Landolphia buchananii* can be used as tensile reinforcement of light structures. The strength tests of *Landolphia buchananii* reinforced beams obtained depicted that *Landolphia buchananii* can be used as a substitute for steel reinforcement in beams under low loading regimes such as in low rise lintel beams with load bearing walls.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PLATES

LIST OF TABLES

SYMBOLS AND ABBREVIATIONS

BS-British standards

Mpa-Mega Pascal

N-Newtons

kN-kiloNewtons

Pa- Pascal

mm-millimeters

M-meters

C02- Carbon dioxide

KES-Kenyan Standard

EAS-East African Standards

CHAPTER 1: INTRODUCTION

1.1 Background information

Country's economic development is closely related to housing provision. Developed countries have a well developed housing system. Construction of buildings and other infrastructure are the key inputs for economic activity, aligned to Millennium Development Goals (Ede, 2011) and Kenya's Vision 2030.In building construction, majorly brick, concrete, steel and wood are normally used. Concrete as one of the major construction material achieved its reputation since 1960's (Barr, 1992).

Cement concrete possesses desirable properties hence making it usable in various structures and design. This is because it can be cast to a desired shape, it has high compressive strength and stiffness and low thermal and electrical conductivity. Concrete has negative characteristics as a construction material. It has a low tensile strength, limited ductility and little resistance to cracking. These negative characteristics limit its use to various applications. Internal micro cracks are inherently present in the concrete and its poor tensile strength is due to propagation of such micro cracks which leads to brittle fracture of concrete (Jagadish K.S et al, 2007 and Shetty M.S, 2005).

Difficulties attributed to low efficiency in tensile strength of concrete structural elements; brittle mode of failure, rapid crack propagation and increased overload are common in concrete construction industry. This difficulty has led to development of contemporary concrete technologies; high strength concrete, fiber reinforced concrete, bamboo reinforced concrete, timber reinforced concrete, steel reinforced concrete and many others. (Ghavami, K. 2004 and Daniel and et al, 1998). These show that there is need to explore more new construction materials, which are going on day by day.

Since application of concrete in structural engineering involves reinforcing it with materials. Majorly steel is use to reinforce concrete. This is because it is readily available and has suitable building properties and has high tensile strength. Ordinary steel reinforced concrete construction is becoming expensive in production cost, transportation of precast members, maintenance cost and supply of such amount of steel. This takes huge investment to produce locally especially in developing countries. Thus, new affordable and available constructions materials are needed to substitute steel in developing countries (Rahman M.M, 2011)

Due to low income of most people in developing countries, they cannot afford steel and other construction materials. This has caused rise of slums in urban areas and low quality houses in rural areas. Very little steel is used for reinforcement in construction in most developing countries, this is the reason why there is crumbling of buildings (Mathenge, 2012). Steel reinforcement won't be available forever. Hence there is need for more economical and readily available substitute concrete reinforcement.

Iron and steel manufacturing is the second largest industrial consumer of energy and the largest industrial emitter of carbon dioxide (CO_2) . Use of naturally occurring products as construction materials will decrease the rate of energy consumption and carbon dioxide emission (Harish S. et al. 2012) . Scientist and engineers are looking for new materials for structural system.

Landolphia buchananii can be of multiple purposes assets: it bears edible fruits and can produce rubber. It can solve the problem of hunger, malnutrition, and rural poverty, devastation from unsustainable land practices, added burdens for women, mothers and children and suffering from fearsome diseases. It can be tamed and turned to use in Africa hence contributing to economic progress (NRCNA. 2008).

The use of the *Landolphia buchananii* stem as a possible concrete reinforcement material can solve the problems pertaining to the use of steel as a concrete reinforcement material. It can be used for sustainable environment development without harming our global environment since as a plant it absorbs a lot of nitrogen and carbon dioxide from the atmosphere during its growth.

1.2 Problem statement and Justification

Demand for new houses and the cost of construction materials is on the rise. Steel and concrete are mostly used in building and construction for modern buildings. Steel is not available but costly. Steel being a non-renewable resource will get depleted at some point, hence there is need for a more economical and available substitutes for steel reinforcement for concrete. This research will attempt to address this problem by investigating the properties of *Landolphia buchananii* as concrete reinforcement material.

1.3 Objectives

1.3.1 General

To explore the *Landolphia buchananii* plant stem as an alternative to steel as a reinforcement material in concrete.

1.3.2 Specific

i. To assess the physical and mechanical properties of *Landolphia buchananii* stem.
ii. To assess the bond strength between *Landolphia buchananii* stem and concrete.
iii. To investigate structural performance of *Landolphia buchananii* reinforced concrete beam (flexural, shear and failure pattern).

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

The literature review covers the *Landolphia buchananii* plant descriptions, location and its uses. The review also covers the past research done *Landolphia buchananii* as a construction material. It also covers general structure and properties of wood as a structural material, concrete mix design and structure and behavior of reinforced concrete beam.

2.2 Developments of vegetable fibre reinforced concrete

Many researches have and are being done on many plants to determine if they can be used as a reinforcement of concrete. Among them are bamboo, jute, sisal and coir fibre (Adom and Afrifa, 2011), (Harrish, et al, 2012), (Tara and Jagannatha, 2011), (Ghavami, K., 1995).

2.3 Description of *Landolphia buchananii* plant

Landolphia buchananii is part of *Landolphia* species. There are seventeen (17) Landolphia species (Family *Apocynaceae*). It bears masses of fruits (rubber/gum fruits) which somewhat looks like apricots, with tough skins that are red, yellow or orange in color. They are forest lianas and sprawling shrubs having jasmine-scented flowers and plentiful fruits and latex filled stems. This is the latex that was used to be exported to Europe and other parts of the world (NRCNA. 2008).

Plate 2.1: *L. buchananii* fruits

Plate 2.2: *L.buchananii* stems

Plate 2.3: L. buchananii plant

Landolphia buchananii can grow up to forty meters tall. Its trunk can grow up to twenty three centimeters in diameter. It's found in East Africa, Ethiopia, Sudan, Mozambique, Democratic Republic of Congo, Zambia, Angola, Cameroun and Nigeria. It grows well between altitudes of 0-2500m. It is composed of roots, long stem and leaves/ branches at the bottom, middle and top respectively (Persoon.J.G, et al.1992).

It was used as a rope to tie parts of the traditional houses when constructing them. This was because of its suitable building properties; flexibility and high tensile strength.

2.3.1 Possible utilization of the *Landolphia buchananii* plant

When added as valuable it will raise the economic worth of standing forest thereby dampening the ardor to burn the land or cut tress for lumber. They can also be incorporated into border rows, wind breakers, shelter belts and ex-situ conservation of forest. Some species that cling and climb could be a way to increase the utility of many long term environmental tree-plantings. They can be trained along the fences, up walls or roofs. They can be trained just like grapes (NRCNA. 2008).

2.4 Structure and properties of wood as a structural material

Wood is anisotropic material with high stiffness and strength parallel to the grain than perpendicular to the grain. The grains are made of fibers which are made up of cells which are long and slender and are aligned parallel to the stem. When load is acting along the grain, wood is strong in tension and in compression: it is only strong in compression until it buckles. The wood will crush under compression and tears under tension. (Leornado da Vinci projects, 2008).

(Leornado da Vinci projects, 2008) explains the properties of wood relevant to engineering. The properties are density, strength, shrinkage, colour, fire resistance, electrical resistance and mechanical dumping.

2.4.1 Density

Density is important physical characteristic of timber. This is because mechanical properties of timber depend on density. While on the other hand density depends on cell structure and size and moisture content.

Density= Mass/Volume

The swelling of the wood is caused by water which has penetrated into the cell wall layer. When the cell wall is fully saturated, the moisture content at this point is termed as fiber saturation point. No swelling will occur when fiber saturation point has been exceeded.

2.4.2 Wood and moisture

Moisture content is defined as the ratio of mass of removable water to dry mass of the wood. Dry mass is obtained by oven drying at 103+-2c. It can be expressed as a fraction or percentage. Moisture content of the wood normally ranges from 6 to 28%.

Methods of measuring moisture content.

i. Electric moisture meters. Its advantage is that it is simple while its disadvantage is that it is less exact method of determining moisture content.
ii. Oven dried weight in %. In this method a representative sample is weighed and then dried in an oven at 103+2c until no further loss of weight takes place. The specimen is reweighed and the difference between final weight and original weight by final oven dry weight.

    When wood specimen is dried, the free water is lost first from the cell lumens. This water in not part of the molecular level of the wood. Fiber saturation point will cause changes in mechanical and physical properties of the wood. For this reason most properties of wood are determined slightly above the fiber saturation point. Care should be taken not to allow moisture content to exceed the fiber saturation point since it increase susceptibility to decay.

2.4.3 Shrinkage and swelling

Moisture can force its way in to the cell wall. Moisture pushes micro-fibril apart causing swelling of the cells. The swelling is equivalent to the volume of the absorbed water.

Shrinkage occurs when the moisture is removed from the cell wall. Shrinkage and swelling on a timber structure cause movements. Linear changes in dimension will occur within the range of 5-20% of moisture content. This can be calculated from:

$$h_2 = h_1\left[1 + \frac{B}{100}(w_2 - w_1)\right] \quad \ldots\ldots 2.1$$

h_1 = thickness at moisture content w1, h_2=thickness at moisture content w2

B=coefficient of swelling- +ve when swelling, -ve when shrinking, units is in %

In conclusion, as long as the moisture content is above the fiber saturation point, moisture content has no effect on the volume or strength of wood. When below the fiber saturation point, the cell wall looses moisture shrinkage begins and strength increases. Shrinkage is more tangentially, less radially and very little longitudinally.

2.4.4 Moisture content and mechanical properties.

The more the moisture content the lower the strength and elasticity value. This is because swelling results in less available material per unit area cross-sectionally. Water also weakens the hydrogen bonds which hold together the cell wall. Above the fiber saturation point, any increase in moisture content has no effect on mechanical properties of wood

Wood fails in compression under load parallel to the grain. This is due to fiber buckling being accelerated by moisture influencing hydrogen bond. under tension, the failure by rupture of covalent bonds due to tearing apart of the cell wall.

When comparing mechanical properties, it is advisable to do it at a constant temperature and humidity, preferably room temperature and humidity. Otherwise if it is to be done under different conditions, the specifications or standards are to be used to adjust. For instance, EN384-Structural timber determination of characteristics values of mechanical properties and density.

2.4.5 Duration of load

Wood looses strength over period of time. Changes in moisture content increases creep of timber. All these shorten the time of failure of timber. Wood is always surface treated to control moisture content variation. This lengthens wood life (Leornado da Vinci projects, 2008).

2.4.6 Mechanical properties.

(America wood council, 1986) gives the following as the mechanical properties of wood.

i. Wood as a structural material

Wood is not isotropic, strength differs along different axes. Wood is very strong when loaded to induce stress parallel to grain both in tension and compression

ii. Tension parallel to the grain

It creates tendency to elongate the wood fibers and cause them to slip by each other. Resistance to tension applied forces strictly parallel to grain is the highest strength property wood. This resistance is reduce when the force is applied at an angle to the grain or when the cross section of a piece is reduced by knots or holes

iii. Compression parallel to the grain

It creates a tendency to compress the wood fibers in lengthwise position. Its compressive strength is affected by angle of load to grain and by presence of knots or holes.

iv. Fiber stress in bending

When force is applied perpendicular to beam, it creates compression in the extreme fibers of the upper part and tension in the lower side. Stress is distributed from extreme fibers towards the centre of neutral axis of the piece it reduces in intensity.

Deviation of grains, presence of knots or holes in the outside phases reduces the resistance in extreme fibers and the bending strength of the beam.

2.4.7 Deterioration and preservation of wood

Wood can deteriorate and loose its structural properties due to decay or attacked by worms and insects. Decay in timber is caused by fungi. Fungi grow well at moisture content above 20%. Condition essential for fungal growth are presence of food supplies, adequate moisture content, suitable temperature and oxygen. Worms and insects activity results in tunneling of wood which reduces the strength of timber (Descih, H.E. and Dinwoodie J.M., 1981).

According to (Descih, H.E. and Dinwoodie J.M., 1981), principal causes of deterioration of timber are fungal infection, termite and other insects, mechanical failure and fire. Resistance of wood to agent of destruction is increased by application of preservatives. In order to improve resistance to fungi and insects, wood preservatives are used. There various types of preservatives used include tar and oils and water-borne preservatives like preservative salts and organic solvent preservatives. Preservatives are applied through different methods. It can be by brush coating, spraying and dipping, open tank heating, double vacuum treatment, pressure process, sap displacement or diffusion process methods. In order to improve resistance to fire, application of flame-retardant chemicals is done. The chemicals include mono and di-ammonium phosphates, ammonium sulphates, boric acid and borex. These chemicals are applied through impregnation.

2.5 Concrete mix design

According to (Neville A. M. 1999**),** Concrete mix design normally describes in terms of proportions by weight of materials which they contain or in terms of the strength required of the concrete at a particular age. Mixed design is the choosing of the ingredients to provide economical concrete with desired properties. It is the deliberate proportioning of the cement, fine and coarse aggregate and water, taking into account not only the specified concrete particles but also the characteristic of the materials used.

There are factors and sequence involved in the process of designing a concrete mix. Sequence of operation can be clarified to provide ease of reference, by dividing flow process into five stages. Particular aspects of the design are dealt at each stage which ends with an important parameter or final unit proportion. The stages of mix design process are as shown below:

Stage 1 : strength is dealt with to get the free water cement ratio

Stage 2 : workability is dealt with to get the free water content

Stage 3 : stage 1 and 2 results are combined to get the cement content

Stage 4 : total aggregate content is determined

Stage 5 : fine and course aggregate content is selected

The main objective of designing a concrete mix consist of selecting the appropriate proportions of cement, fine and course aggregate and water, to produce concrete having the specified and desired properties. The properties usually specified are:

i) The workability of the fresh concrete

ii) The compressive strength of concrete

iii) The durability of concrete

2.5.1 Workability of Concrete

Workability of concrete implies the ease with which a concrete mix can be handled from the mixer to its finally compacted shape. The three main characteristics of workability are consistency, mobility and compatibility. The required workability should give maximum density, minimum void and no segregation.

Factors Effecting Workability

There are various factors that affect workability of concrete. These factors include water content, time and temperature, type of cement, aggregates and others

i) Water Content

Water content of the mix is the main factor, it is expressed in kilogram (or liters) of water per cubic meter of concrete. It is assumed that, for a given type of grading and workability of concrete, the water content is independent of the aggregate /cement ratio or of the cement content of the mix. When the water content and the other mix proportions are fixed, workability is governed by maximum size of aggregate, its grading, shape and texture. Aggregate particles of sharp edges and a rough surface, such as crushed stone, need more water than of smooth and rounded particles to produce concrete of the same workability.

In general, a crushed aggregate concrete may have a higher strength than a smooth or rounded aggregate concrete at the same water/cement ratio. Therefore the fine and course aggregates should be proportioned to obtain the required degree of workability with minimum amount of water.

ii) Cement type

Different types of cement require different amount of water to produce pastes of standard consistence. Different types of cement also produce concrete that have a different rates of strength development. Therefore the choice of type of cement is the most important to produce desired of concrete.

iii) Aggregates

Aggregates have an important influence on its properties; this is because it occupies about 70 to 80% of the volume of concrete. Aggregate should contain no constituent that might adversely affect the hardening of the cement or the durability of the hardened mass aggregate. Shape and texture affect the workability of fresh concrete. Sufficient paste is required to coat the aggregates and to provide lubricating to decrease interactions between aggregate particles during mixing. Aggregate will require different amount cement paste due to shape and texture. For desired proportion to be determined for suitable concrete mixes, certain properties must be known such as shape and texture, size gradation, moisture content, specific gravity and bulk unit weight. These properties affect the paste requirements for workable fresh concrete (Neville A. M. 1999).

2.5.2 The Compressive Strength of Concrete

Compressive strength of concrete depends on age, cement content and the water /cement ratio. An increase in any of these factors will result to an increase in strength.

Mixing of Fresh Concrete

Mixing is done to coat the surface of all aggregate particles with cement paste and blend the ingredients into a uniform mass. The mixing can either in rotation or stirring operation method. The rotation operation can be done using drum mixer. While the stirring operation is done using pan type mixer.

Age at Test and Curing Conditions

The strength of concrete increases for many months under favorable conditions, but in the majority of specifications the strength is specified at an age of 28 days. The strength development of concrete made with Portland cement depends on the temperature and humidity conditions during curing. Higher temperatures increase the speed of the chemical reaction and thus the rate of strength development. Constant presence of water order to achieve higher strengths at later ages loss of water from the concrete must be prevented. For test purposes the concrete test specimens is stored in water at a constant temperature of around 20^oC as specified in BS 1881: Part 3.

2.5.3 Durability of Concrete

Durability of concrete is the ability of concrete to withstand the damaging effects of the environment and of its services conditions without deterioration for a long period of time.

The concrete should be designed in such a way that it can be used without it deteriorating for a long period of time. Durability is affected by external environment surrounding the concrete or internal causes within the concrete, internal environment. The external environment causes can be physical, chemical or mechanical. This may be due to weathering, abrasion, effects of extreme temperature, action of electrolyte, and attack by industrial effluent. The degree of damage by these causes mainly depends on the concrete quality.

The internal environment causes include the alkali aggregate reaction, volume change due to moisture content or temperature difference. The durability is important because it ensures that the structure withstand the condition exposed to for the entire life of the structure. In summary the external causes include the effects of environment and service conditions to which concrete is subjected such as weathering, chemical and others. The internal environment causes are effects of salt, for instant chlorides and sulfates (Neville A.M 1999).

Specifications, for example British Standard, contain clauses which provide durability requirements for concrete subjected to different condition of exposure and they provide the constraints on the mix design.

2.6 Design consideration for reinforcement concrete beam.

According to (Mosley W.H and Bungey J.H., 1990), reinforced concrete beam design aims at producing a property which will resist the ultimate bending moment, shear forces and torsional moments. Serviceability requirements must also be considered to ensure that the member behave satisfactorily under serviceability loads. The design procedure of beam consists of series of steps and checks to take care of serviceability limit state and ultimate limit state. The procedure can be split into three stages. These stages are always followed to get accurate results. These stages are:

• Preliminary analysis and member sizing

• Detailed analysis and design of reinforcement at ultimate limit state

• Serviceability conditions checks and design

2.6.1 Reinforcement requirement in beam.

Minimum area of reinforcement

This will prevent thermal and shrinkage cracking. The acceptable limits are provided by standards/specification, for instance BS8110.

For tension reinforcement of rectangular section is given by;

$$\frac{100A_s}{A_c} = 0.13\% \qquad \ldots 2.2$$

Where A_s= Area of tensile reinforcement steel, A_c= Area of concrete.

For compression reinforcement of rectangular beam is given by;

$$\frac{100A_{sc}}{A_c} = 0.2\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..2.3$$

Where A_{sc} = Area of compression reinforcement of steel, A_c = Area of concrete.

Maximum area of reinforcement

This will enable the achievement of adequate compaction of concrete around the reinforcement. This is also specified in the in BS8110 and is given by:

$$\frac{100A_s}{A_c} or \frac{100A_{sc}}{A_c} \leq 4\% \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .\ldots 2.4$$

Where A_{sc} = Area of compression reinforcement of steel, A_c = Area of concrete, A_s= Area of tensile reinforcement steel, A_c= Area of concrete.

2.6.2 Structural behavior of reinforced concrete beam

Deflection

There are relation between applied load, stress and deformation/deflection that occur in a beam. There is always a limit of deflection when designing a beam. This is always given in specifications (BS 8110). This is because excessive deflection will cause cracks on plaster walls and ceilings. It will also interfere with serviceability of a structure.

Macaulay's method is normally used to determine deflection. This is a double integration method. This method is uses established deferential equation that governs beam deflection. The equation is based on the assumption that the plane section within the beam remains plane before and after loading and deformation of the fiber is proportional to the distance from neutral axis (Mosley W.H and Bungey J.H., 1990).

The following are the assumptions made on elastic curve equation:

i) The beam deflection due to shearing stress is negligible

ii) The value of elastic modulus, E and second moment of inertia, I remain constant along the beam.

Macaulay's Method

According to (Benham, P.P, et al), Macaulay's method is based on double integrated method. It consists of derivation of elastic curve of beam.

$$EI\left(\frac{d^2y}{dx^2}\right) = M(x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.2.5$$

Where;

E = Modulus of elasticity for the material;

I = Moment of inertia about neutral axis;

M (x) = Bending Moment along the beam as function x.

The deflection of a beam depends on four general factors;

i) Stiffness of the material that the beam is made of;

ii) Length of beam;

iii) Applied loads; and

iv) Types of beam supports.

Behavior of flexural Member under Deflection

According to (Mosley W.H and Bungey J.H., 1990), when a flexural member is subjected to a bending moment, different stress configuration along the member at various cross sections will occur. Tension and compression zone divided by neutral axis will occur. Concrete flexural member fails under maximum moment at outer fibers of tension when minute cracks are formed. The cracking extends closer to the neutral axis as cracks widen. Concrete between cracks will still carry some tension forces hence tensile stress in steel. Tensile stress in steel between the cracks is more than at the crack.

There are two phases in behavior of typical beam when load is applied. First phase is when the section has not cracked. Here the un-cracked section behavior predominates. The second phase is when the section has cracked where the cracked section behavior dominates

2.6.3 Cracking

Tension cracking will normally occur under even under a service load on reinforced concrete member. This cracking develops when concrete with a limited capacity for elongation tends to deform with the tensile reinforcement.

Cracking mechanism is based on redistribution of concrete stress at crack formation that corresponds to observe internal and surface cracking. This happens through bonding action.

It’s among the requirement for serviceability limit state in reinforced concrete structure that is cracking of concrete does not affect durability of structures.

Cracking can be controlled by ensuring that the spacing of reinforcements does not exceed certain limits. This is achieved by using crack width equations (Mosley W.H and Bungey J.H., 1990).

Subcritical crack growth also exists in structures. It is observed under slow crack growth in material, especially metal under cyclic load. This cracking is associated with neither ductile fracture nor plastic deformation. Crack growth characteristics determined by combination of factors. These factors are loading, geometry and size of specimen, material and environment (Sih G. C., 1983).

CHAPTER 3: METHODOLOGY

3.1 Introduction

This chapter presents the methodology undertaken to achieve the specific objectives of this research. The methodology consists of tests for determining physical and mechanical properties of *Landolphia buchananii*, interfacial bond strength of reinforcement in concrete and flexural properties of reinforced concrete beams.

3.2 Materials required

In order to perform the relevant tests for determining the properties required to achieve the objectives of this research, the following materials were required:

i. **Cement:** Local Portland Pozzolona cement to be used. It is manufactured to Standard Specification **KS EAS 18-1: 2001** and is classified as **CEM IV/B-P 32.5N Portland Pozzolana Cement**. This cement, produced by Bamburi Cement Limited has a wide range of applications from domestic concrete to large civil engineering projects.
ii. **Course aggregate:** Local coarse aggregate to be used of maximum size of 20mm
iii. **Water:** Local water supplied through taps in the laboratories
iv. **Fine aggregate/Natural sand:** normal river sand to be used
v. ***Landolphia buchananii* stems**: available in Nandi forest in western Kenya. The diameters of the stems varied from 6 mm to 40 mm
vi. **High Yield twisted and Mild Steel Bars:** Y12, Y8, R8 and R6 and binding wires
vii. **Plywood:** 1 inch thickness for formwork.

3.3 Physical and Mechanical properties of *Landolphia buchananii* stem

The physical and mechanical properties of the stems were determined for both green and dry *Landolphia buchananii* stems. The physical properties determined were moisture content, density and dry density. The mechanical properties determined were tensile strength, compressive strength, shear strength, hardness, modulus of elasticity and modulus of rupture.

3.3.1 Moisture content

This was determined using BS 373-1957. The test piece for compression test of 2cm by 2cm x 6cm was used. The mass of the specimen before and after oven drying was recorded.

The moisture content of each test piece was calculated as the loss in mass expressed as a percentage of the oven dry mass.

$$Mc(\%) = \frac{M - M_o}{M_0} \times 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.1$$

Where: Mc=moisture content, M= the mass of the test specimen before drying, M_o= the over dry mass of the test specimen.

The characteristic moisture content was calculated. This was the arithmetic mean associated with standard deviation of the specimen's moisture content results obtained.

3.3.2 Density and Dry density

Density at a natural moisture content of the test specimen was determined in accordance to BS 373-1957. The same test piece that was used to determine the moisture content was subsequently used to determine the dry density. All test pieces were weighed and their dimensions determined before test. The densities before oven drying and after oven-drying were determined for each test piece.

Density before oven drying

$$D = \frac{w_1}{V_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.2$$

Dry density after oven drying

$$D_D = \frac{w_0}{V_1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.3$$

Where D= density in g/cm^3, D_D =Dry density, W_0=oven dried mass in gm of test specimen, W_1= Weight of sample at test in grammes, V_1= the oven-dry volume of the test specimens in mm^3

The characteristic density and dry density was then calculated as the arithmetic mean associated with standard deviation of the specimen's densities results obtained.

3.3.3 Determination of tensile strength

Tensile strength was determined through tensile test method. In this case, the tensile test method of tension parallel to the fibers was adopted. The test was done in accordance to BS 373-1957 to determine tensile strength. The load was applied to the test piece at a constant head speed of 1.27mm/min. The specimen was shaped to allow failure at gauge section.

The ultimate tensile strength was determined by using:

$$\sigma_T = \frac{P_{uh}}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.4$$

Where σ_T= ultimate tensile strength in Mpa. A=mean cross-sectional area of gauged portion in mm^2. P_{uh}= the max load at which the test piece fail in Newtons.

The characteristic tensile strength was then calculated. This characteristic tensile strength calculated was taken as the arithmetic mean associated with standard deviation of the specimen's tensile results obtained.

3.3.4 Determination of compressive strength

The test was done in accordance to BS 373-1957. The compression test by method of compression parallel to the fibers on the specimen was conducted. The test pieces were of 60mm length 20mm breadth and 20mm depth. The experimental setup is as shown in plate 3.2. The load was applied to test piece in a rate of 0.635 mm/min on parallel to the grain. The ultimate compressive stress was determined by:

$$\sigma_C = \frac{P_{uh}}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.5$$

Where σ_C = ultimate compressive strength in Mpa. A=mean cross-sectional area of gauged portion in mm^2. P_{uh}= the max load at which the test piece fails in N.

The characteristic compressive strength taken as the arithmetic mean associated with standard deviation of the specimen's compressive strength of results obtained was then calculated.

3.3.5 Static Bending test with centre point loading

In the study, bending test was done to obtain the modulus of elasticity and modulus of rupture. This test was carried out as per BS 373-1957 test specifications. The test was based on three point loading method of the test piece beams. The dimensions of the central loading test piece were 2 cm by 2 cm by 30 cm. The loading rate was at a constant speed 6.6 mm/min. The supports were unrestrained. The experimental set is as shown in plate 3.3. The deflection of the beam at mid length was measured with reference to the outer points of loading.

The bending strength/ modulus of rupture was determined by:

For square/ rectangular specimen

$$\sigma_B = \frac{My}{I} = \frac{3PL}{2bh^2} \quad \text{...3.6}$$

The modulus of elasticity was determined by:

$$E = \frac{PL^3}{4\Delta' bh^3} \quad \text{...3.9}$$

Where, σ_B = bending strength. P= the applied maximum load in N. L= the length of the free span in mm (clear span). Δ'=deflection, b= breath, h=height, y=distance from neutral axis, E= modulus of elasticity.

The characteristic modulus of rupture and elasticity was calculated. This is regarded as the arithmetic mean associated with standard deviation of the specimen's modulus of elasticity and modulus of rupture results obtained.

3.3.6 Shear test

Shear test was done through BS 373-1957 to determine shear strength. The test piece was a cube of 2 cm sides. The load was applied at a constant rate of crosshead movement of 0.625 mm/min. The direction of shearing was parallel to the grain. The experimental set up was as shown in plate 3.4. Shear strength was determined by using the formula given below from which the ultimate load is P and cross-sectional area is A

$$\sigma_S = \frac{\text{load}}{\text{area}} = \frac{P}{A} \quad \text{...3.10}$$

The characteristic shear strength value was then computed as the arithmetic mean associated with standard deviation of the specimen's shear strength of results attained.

3.3.7 Janka hardness test

This was done to determine the load necessary to force into the test piece, to a depth of 0.222 in., the hemispherical end of a steel bar, or a steel ball, 0.444 ± 0.002 in. in diameter. This test is in accordance with BS 373-1957. The test piece was of size 100mm long with a square section 20 mm by 20 mm. The rate of penetration of the hardness tool was 6.35 mm /min. The test set up was as shown in plate 3.4. The hardness was determined in tangential surfaces.

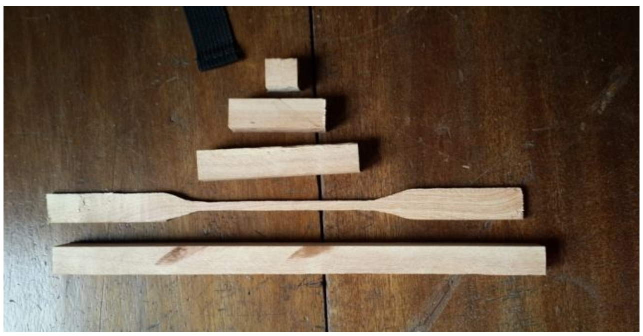

Plate 3.1: The samples to be tested

Plate 3.2: Compression test

Plate 3.3: Bending test

Plate 3.4: Shear test

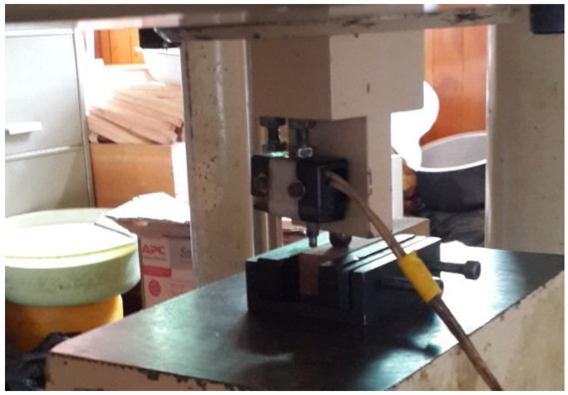

Plate 3.5: Janka hardness test

3.4 Pullout test

3.4.1 Introduction

Pull out test is used to determine the interfacial bond strength of reinforcement in concrete. In this test the reinforcement is embedded in concrete whose crushing strength is predetermined. The test is conducted by applying load to pull the reinforcement out of the concrete. It is from this load that the interfacial bonding strength is derived.

3.4.2 Preparation

Concrete standard nominal mix ratio of 1:2:4 by volume was adopted from British standards BS 8500 which is expected to normally yield target strength of 20 N/mm^2 at the concrete age of 28 days. Slump test value was maintained at 30-60mm. This was to avoid excess water which can cause the swelling of the stem. The local Portland Pozzolona cement manufactured to Kenyan Standard Specification **KS EAS 18-1: 2001** which is classified as **CEM IV/B-P 32.5N Portland Pozzolana Cement** was used. The maximum size of aggregate used was 20mm. The reinforcements used were *Landolphia buchananii* stems of diameter 9-17 mm and high strength twisted steel bars of Y8. A total of 11 specimens were prepared: 3 specimens of steel and 8 specimens of *Landolphia buchananii*. The reinforcements were embedded into a concrete in a cube of 150x150x150mm and then cured for 28 days when the pull-out test was performed as shown in plate 3.6 and 3.7. Load was applied to pull reinforcement out of the concrete. The bond strength was determined using the formula given below.

$$f_s = \frac{P}{\pi DH} \quad \text{3.11}$$

P=Maximum load at failure.

D= Diameter of the bar embedded in concrete in mm.

H= depth of the bar embedded in the concrete.

f_s= shear strength value between the two surfaces (bar and concrete) in N/mm^2

The average bond strength was calculated for steel and plant stem in concrete from the bond strength results obtained.

Plate 3.6: Pull out test samples

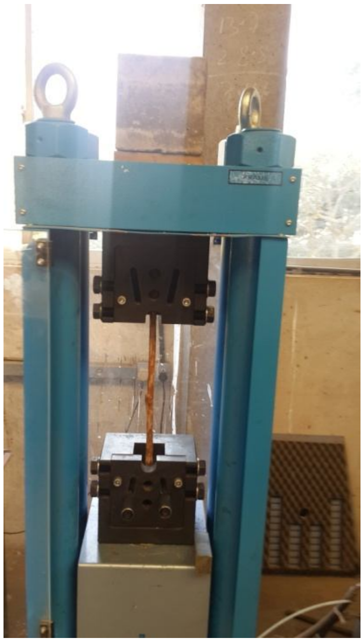

Plate 3.7: Pull out test

3.5 *Landolphia buchananii* reinforced concrete beam test

3.5.1 Introduction

This test was selected to study behavior of the reinforced beam subjected to force acting transversely to their longitudinal axis so as to investigate the cracking load, ultimate load, failure pattern, deflection and flexural strength of *Landolphia buchananii* and steel reinforced concrete beam. Concrete compressive test was performed to determine the properties of concrete for the purpose of ensuring that the concrete met the required target strength in order to ensure that the results for *Landolphia buchananii* and steel reinforced concrete beams were reliable.

3.5.1 Compressive tests cubes

Three cubes of 150 mm by 150mm by 150 mm were casted for every batch. There were three batches in total. The concrete mix of the concrete used in casting the compressive concrete cubes was drawn from the concrete used in making the beams**.**

3.5.2 Set and sizes of beams casted.

In this case, three types of beams were utilized that is, steel reinforced concrete beams (control), tensile *Landolphia buchananii* reinforced concrete beams and shear *Landolphia buchananii* reinforced concrete beams. These beams were of 1100 mm length having 150mm width and 250 mm depth were casted. Mid-span loading beam test method was adopted for the beam tests to be performed (plate 3.15 and figure 3.1). The summary of the details of the beams casted were as shown in the table 3.1.

Table 3.1: Summary details of beams casted

Type of Beam	S/no	Reinforcement	Size of reinforcement,		Percentage of reinforcement, %
			Tensile, mm2	**Shear, dia. mm**	**Tensile**
Control	Beam 01	2R8 top, 2Y8 bottom, R8 links spacing 200mm	101	R8	0.26
	Beam 02	2Y12 top, 2Y12 bottom, R6 links spacing 200mm	226	R6	0.6
Landolphia buchananii as a tensile reinforcement	Beam 01	2 *L. buchananii*. top dia. 10mm, 3L.buchananii bottom, dia.21mm, 11mm, 21mm, R8 links at 200mm spacing	787	R8	2.09
	Beam 02	2 *L. buchananii* top dia. 10mm, 3L.buchananii bottom, dia.21mm, 12mm, 21mm, R8 links at 200mm spacing	805	R8	2.15
	Beam 03	2 *L. buchananii*. top dia. 10mm, 3 *L. buchananii* bottom, dia.20mm, 10mm, 20mm, R8 links at 200mm spacing	741	R8	2
	Beam 04	2 *L. buchananii*. top dia. 10mm, 3L.buchananii bottom, dia.13mm, 14mm, 14mm, R8 links at 200mm spacing	419	R8	1.12
	Beam 05	2 *L. buchananii*. top dia. 10mm, 3LB bottom, dia.18mm, 16mm, 18mm, R8	710	R8	2

		links at 200mm spacing			
	Beam 06	2 *L. buchananii*. top dia. 10mm, 3L.buchananii bottom, dia.14mm, 12mm, 13 mm, R8 links at 200mm spacing	399	R8	1.06
Landolphia buchananii as a shear reinforcement	Beam 01	2Y12 top and bottom, *L. buchananii* links dia. 6mm spacing at 200mm	226	6	
	Beam 02	2Y12 top and bottom, *L. buchananii* links dia. 8mm spacing at 200mm	226	8	
	Beam 03	2Y12 top and bottom, *L. buchananii* double links dia. 10mm spacing at 200mm	226	2x10	
	Beam 04	2Y12 top and bottom, *L. buchananii* double links dia. 13mm spacing at 200mm	226	2x13	

3.5.3 Preparation of the beams

The following activities were carried out in the preparation of *Landolphia buchananii* and steel reinforced concrete beams:

a) Selection of reinforcement bars

The plant stems with no visible defects and little and minimum variation in diameter along the length were selected and the bark was removed.

b) Preparation

i. Reinforcement Sizing and fixing.

The reinforcement was cut into specific sizes. *Landolphia buchananii* stems for tensile reinforcements were cut to 1.05m length pieces while steel was cut to 1.25m length pieces. Conversely, for shear reinforcement, *Landolphia buchananii* stem were cut to 0.65m length pieces whereas that of steel were cut to 0.85m length pieces. The tensile reinforcements were then fixed with shear links spaced at 200 mm from centre to centre. The fixed reinforcements were as shown in plate 3.8 and 3.9.

ii. Formwork preparation.

Formwork was prepared using plywood of 1 inch thickness. The plywood was used to prepare a formwork of internal dimension of 250x150x1100mm. The formwork was as shown in plate 3.10.

iii. Reinforced beam concrete test specimen preparation

The conventional techniques of preparing a concrete beam were used in all the beams. The concrete cover of 20mm to the links was maintained.

iv. Concrete mix preparation

Standard nominal mix ratio of 1:2:4 by volume was adopted which was expected to yield target strength of 20 N/mm^2 at the age of 28 days. Slump test value was maintained at 31-37 mm. This was to avoid excess water which can cause the swelling of the *Landolphia buchananii* stem. The local Portland Pozzolana cement was used. Maximum size of aggregate used was 20mm. Standard nominal mix ratio was adopted from British Standard BS 8500.

v. Strain gauges fixing

The surface for sticking the strain gauge was cleaned using sand paper and ethanol before sticking with super glue to the surface. It was then wrapped with waxed bandage cloth to protect it. The strain gauges used were electrical strain gauges.

For tensile reinforced concrete beams the strained gauge was placed at the mid-span of the bottom reinforcement while for the shear reinforced beams the strain gauge was place at mid-height of the shear reinforcement next to the support. In both tensile and shear reinforced beams the second strain gauge was placed at concrete at the mid-span bottom of the beam. The strain gauges were protected from damage during casting by covering it with waxed bandage. The strain gauge placement was as shown in plate 3.11and 3.12.

vi. Casting

The beams were casted and compacted using poker vibrator with concrete of slump value between 31 mm and 37 mm then cured for 28 days.

vii. Laboratory testing

The testing equipment consisted of;

i. Hydraulic jack for loading
ii. Loading frame
iii. Strain gauges
iv. Transducer
v. Load cell.
vi. Data logger where load cell, strain gauges and transducer are connected to it.

The beams surfaces were painted with white emulsion paint (plate 3.13) before testing so as to make the crack visible during testing. The beam specimens were subjected to static loading until failure and carefully observed it so as to determine the mode of failure. The deflection was monitored by placing the transducer at the mid span of the beam (plate 3.15). The occurrence of crack was checked by naked eye. All reading of strains from the gauges, load cell and mid span deflections is recorded by data logger (plate 3.16).

Plate 3.8: Fixed tensile reinforcements

Plate 3.9: Fixed shear reinforcements

Plate 3.10: Casting of beams

Plate 3.11: Fixed strain gauge

Plate 3.12: Covered/bandaged strain gauge

Plate 3.13: Painted beam

Plate 3.14: Hydraulic jack and load cell

Plate 3.15: Transducer and the strain gauges

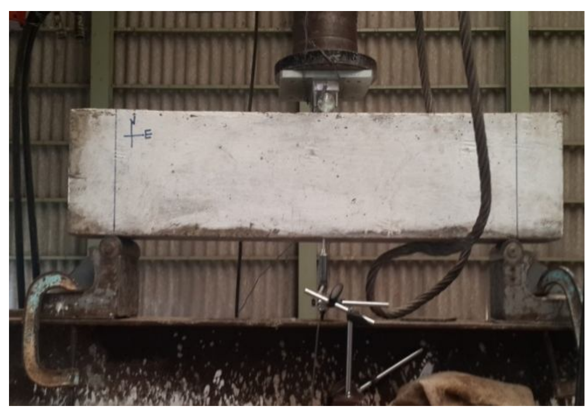

Plate 3.16: Data logger

Plate 3.17: Beam set up for testing

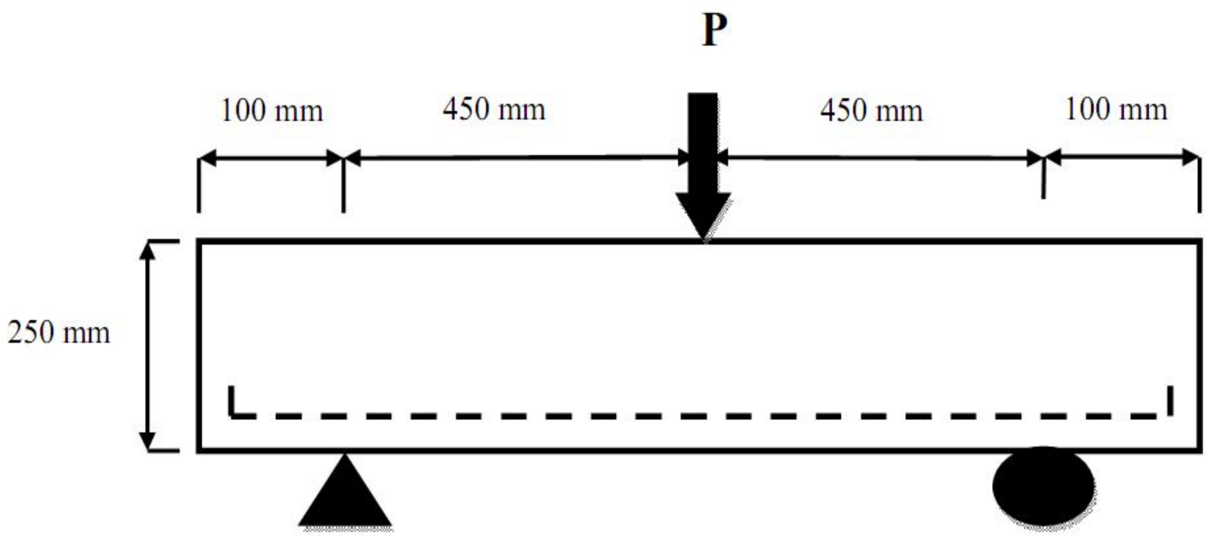

Figure 3.1: Three Point Bending Test Set Up

CHAPTER 4: RESULTS, ANALYSIS AND DISCUSSION

4.1 Introduction

In this chapter, results of several material properties of *Landolphia buchananii,* concrete, and flexural behavior of *Landolphia buchananii* and steel reinforced concrete beams have been presented and discussed. All the results obtained in this chapter were obtained from the test conducted as shown in chapter 3.

4.2 Physical and mechanical properties of *Landolphia buchananii* stems

This section is presented with an aim of showing the physical and mechanical properties of *Landolphia buchananii* whose understanding and values are necessary in determining the viability of *Landolphia buchananii* as an alternative non-conventional material for reinforcing concrete. These physical and mechanical properties determined were moisture content, density, dry density, tensile strength, compressive strength, shear strength, bending strength, modulus of elasticity and hardness. These properties were determined for both dry and green *Landolphia buchananii* stems.

4.2.1 Determination of moisture content, density and dry density

The moisture content, density and dry density of twenty five specimens of *Landolphia buchananii* stems were determined and the results were as shown in table 4.1. Strength properties of wood are influenced by its moisture content, density and dry density. Moisture content of approximately 12% is required to ensure accurate results for clear specimen tests. Specimens with moisture content below fiber saturation point gives more reliable results than those with moisture content above the fiber saturation point.

The average moisture content in dry *Landolphia buchananii* was found to be 13 % while for green *Landolphia buchananii* was 56% (table 4.1). Moisture content in green *Landolphia buchananii* was around 4.5 times that of dry *Landolphia buchananii* as shown in figure 4.1.

From table 4.1 the average density for green *Landolphia buchananii* stem was 1.27 g/cm^3 while that of dry *Landolphia buchananii* stem was 0.53 g/cm^3. Density of green *Landolphia buchananii* stems is thus about 2.5 times that of dry *Landolphia buchananii* stems as shown in figure 4.2.

As observed from the results in table 4.1, the average dry density for green *Landolphia buchananii* stems was found to be 0.46 g/cm^3 while that of dry specimen was 0.55 g/cm^3. The dry density of green *Landolphia buchananii* stem is around 1.2 times that of dry *Landolphia buchananii* as shown in figure 4.3.

4.2.2 Determination of tensile strength

As with determination of moisture content, density and dry density, the same number of specimens of green and dry *Landolphia buchananii* stems was also used in determination of tensile strength. The results obtained were as shown in table 4.2. From the results, it was observed that the average tensile strength for green *Landolphia buchananii* stems was 25.8 N/mm^2 whereas that of dry *Landolphia buchananii* stem was 87.2 N/mm^2. This therefore shows that the tensile strength of dry *Landolphia buchananii* stem is about 3.5 times that of green *Landolphia buchananii* stems as shown if figure 4.4. Plate 4.1 shows the mode of failure of the tensile tests specimen.

4.2.3 Determination of compressive strength

In determining the compressive strength, also twenty five specimens of green and dry *Landolphia buchananii* stems was taken. The results were obtained and presented as shown in table 4.3. It was found out from the results in table 4.3 that the average compressive strength for green *Landolphia buchananii* stems was at 16.8 N/mm^2 whilst that of dry *Landolphia buchananii* stem was 23.7 N/mm^2. Based on these results, it is estimated that the compressive strength of dry *Landolphia buchananii* stem is 1.5 times that of green *Landolphia buchananii* stems (figure 4.5)

4.2.4 Static Bending test with centre point loading

The short-term bending test results of both green and dry *Landolphia buchananii* stems with average moisture content of 56% and 13 % respectively were as shown in table 4.4. Figure 4.11 shows a typical load deflection curve from static bending test conducted on dry and green

Landolphia buchananii stem. From figure 4.11, it can be seen that curve have two sections which are linear, between deflection 0-2.5mm and between 2.5-14mm. Figure 4.11 shows that the curve for the green *Landolphia buchananii* stem was more or less similar to that of dry *Landolphia buchananii*. The mean bending stress for green and dry *Landolphia buchananii* was 22.5 N/mm^2 and 47.8 N/mm^2 respectively. The mean bending strength for dry *Landolphia buchananii* stem was higher than that for green *Landolphia buchananii* stem by around 112 % as shown (figure 4.8). The average modulus of elasticity of green *Landolphia buchananii* from these two linear sections was 3693 N/mm^2 and 2244 N/mm^2 while for dry *Landolphia buchananii* was 3349 N/mm^2 and 2414 N/mm^2 which was around 0.9 and1.1 times that of green *Landolphia buchananii* respectively (figure 4.6 and 4.7) . The bending test specimen did not break or crack when bend, the specimen underwent bending as load increased as shown plate 4.2 and plate 4.3.

4.2.5 Shear test

The shear strength was also determined for twenty five specimens of green and dry *Landolphia buchananii* stems and the results were as shown in table 4.5. From the results, the average shear strength for green *Landolphia buchananii* stems was 5.5 N/mm^2 and that of dry *Landolphia buchananii* stem was 9.9 N/mm^2. In comparison, the results indicated that shear strength of dry *Landolphia buchananii* stem was around1.9 times that of green *Landolphia buchananii* stems (figure 4.9).

4.2.6 Janka hardness test

The hardness test was also carried out in the study. This test was similarly conducted as with the above tests in which twenty five specimens of green and dry *Landolphia buchananii* stems were used and the results were as presented in table 4.6. The study results indicated that the average hardness for green *Landolphia buchananii* stems was 2483 N while the one for dry *Landolphia buchananii* stem was 3666 N. Upon calculation, it was found out based on the results that the hardness of dry *Landolphia buchananii* stem was 1.5 times that of green *Landolphia buchananii* stems as shown in figure 4.8.

4.2.7 Summary of the results

Table 4.7 present the summary of the average values of results for both dry and green *Landolphia buchananii*. It can be seen that the values of the properties of dry *Landolphia buchananii* are higher than those of green *Landolphia buchananii*. The values are higher by 350% for moisture content, 150% for density, 250% for tensile strength, 50% for compression strength, 112% for bending strength, 90% for shear strength and 50% for hardness. Table 4.8 present the summary of the maximum and minimum property values of the results for both dry and green *Landolphia buchananii*.

Table 4.1: Density, Dry density and Moisture Content test results

	Density (g/cm^3)		Dry Density(g/cm^3)		Moisture Content, %	
Sample no.	**Green**	**Dry**	**Green**	**Dry**	**Green**	**Dry**
1	1.207	0.441	0.513	0.391	57.5	12.8
2	1.205	0.469	0.563	0.419	53.3	12.1
3	1.245	0.474	0.53	0.411	57.4	15.3
4	1.202	0.48	0.479	0.424	60.1	13.2
5	1.396	0.486	0.663	0.429	52.5	13.3
6	1.318	0.49	0.57	0.431	56.7	13.7
7	1.261	0.491	0.519	0.434	58.9	13.1
8	1.22	0.494	0.53	0.436	56.6	13.3
9	1.229	0.503	0.575	0.444	53.2	13.3
10	1.198	0.505	0.525	0.442	56.2	14.2
11	1.171	0.508	0.509	0.45	56.5	13.0
12	1.417	0.515	0.693	0.456	51.1	12.8
13	1.336	0.519	0.548	0.46	59.0	12.9
14	1.284	0.52	0.552	0.463	57.0	12.3
15	1.188	0.523	0.482	0.465	59.4	12.5
16	1.156	0.533	0.489	0.47	57.7	13.5
17	1.277	0.539	0.561	0.468	56.1	15.1
18	1.223	0.544	0.507	0.485	58.5	12.2
19	1.354	0.552	0.631	0.484	53.4	14.1
20	1.22	0.568	0.525	0.499	57.0	13.8
21	1.348	0.572	0.577	0.507	57.2	13.0
22	1.149	0.581	0.469	0.512	59.2	13.3
23	1.252	0.581	0.585	0.511	53.3	13.6
24	1.192	0.586	0.561	0.503	53.0	16.4
25	1.258	0.61	0.532	0.538	57.7	13.4
26	1.342	0.615	0.617	0.542	54.0	13.4
MEANS	**1.26±0.07**	**0.53±0.05**	**0.55±0.06**	**0.46±0.04**	**56±3**	**13±1**

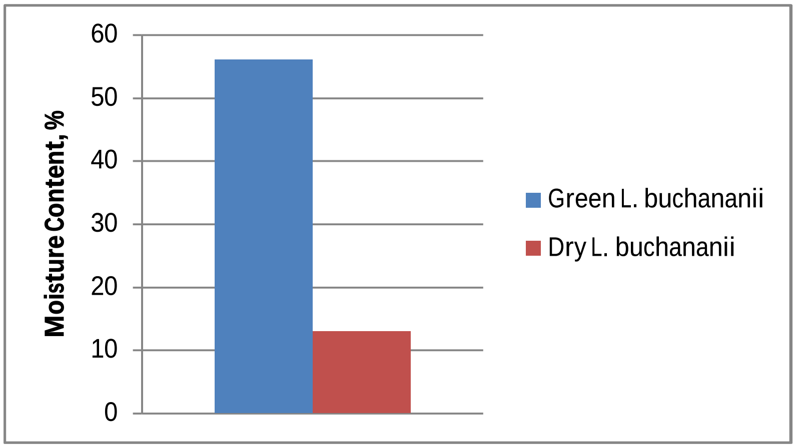

Figure 4.1: Comparison of moisture content

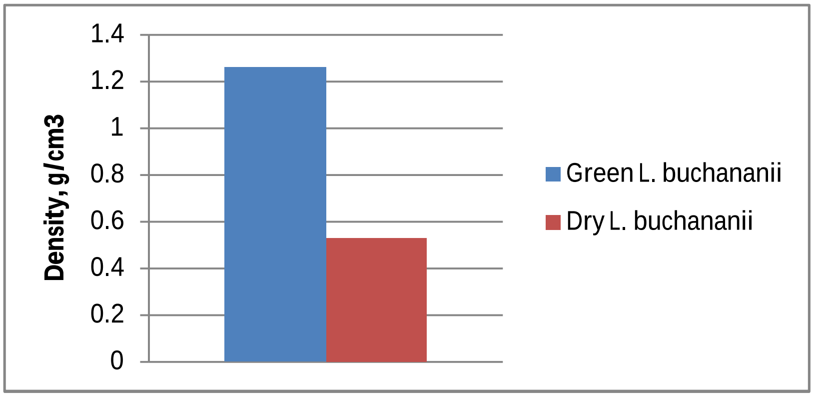

Figure 4.2: Comparison of density

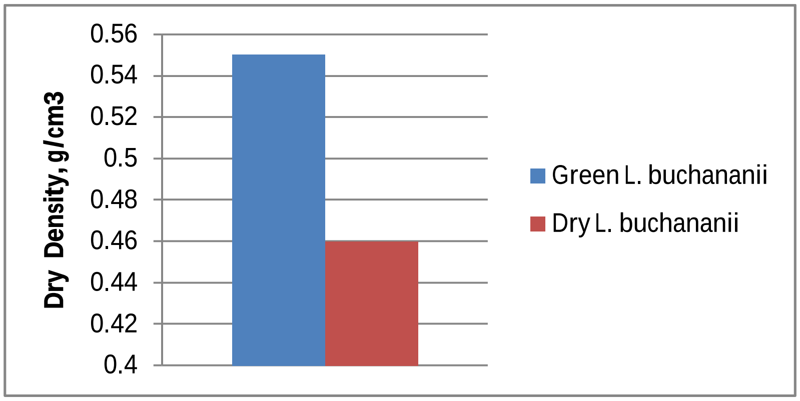

Figure 4.3: Comparison of dry density

Table 4.2: Tensile test results

	GREEN SAMPLES	DRY SAMPLES
Sample no	**Tensile Strength, N/mm^2**	**Tensile Strength, N/mm^2**
1	23.8	101.6
2	38.6	81.6
3	26.3	87.6
4	16.3	76.33
5	25.1	112.6
6	17.7	99.3
7	4.1	80.0
8	30.4	87.6
9	26.3	89
10	44.6	68.3
11	29.4	85.0
12	36.5	65.0
13	26.8	70.0
14	21.3	88.3
15	17.3	79.3
16	36.0	83.3
17	26.9	73.3
18	23.1	97.3
19	25.8	92.0
20	27.3	95.0
21	26.0	96.0
22	24.7	86.6
23	27.3	91.3
24	17.7	105.6
25	24.2	81.6
26	26.3	93.6
MEAN	**25.8±7.9**	**87.2±11.5**

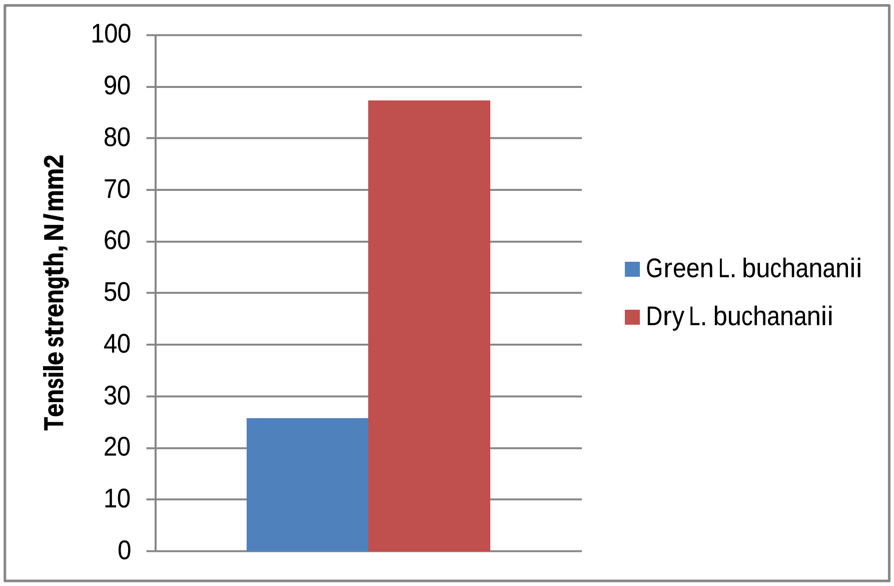

Figure 4.4: Comparison of tensile strength

Table 4.3: Compression test results

	GREEN SAMPLE	DRY SAMPLES
Sample no	**Compressive strength, N/mm^2**	**Compressive strength, N/mm^2**
1	12.164	22.425
2	21.917	23.875
3	17.4	20.6
4	16.14	22.8
5	12.943	20.225
6	15.333	25.725
7	12.857	24.925
8	17.451	20.95
9	17.7	27.025
10	17.689	20.675
11	15	25.45
12	19.083	28.95
13	17.686	23.425
14	18.672	22.875
15	15.229	21.6
16	22.316	21.8
17	13.434	21.275
18	16.975	26.6
19	19.35	25.675
20	13.341	20.95
21	15.673	24.55
22	21.361	22.15
23	17.075	26.575
24	21.404	27.95
25	13.11	22.8
MEAN	**16.8±2.9**	**23.7±2.5**

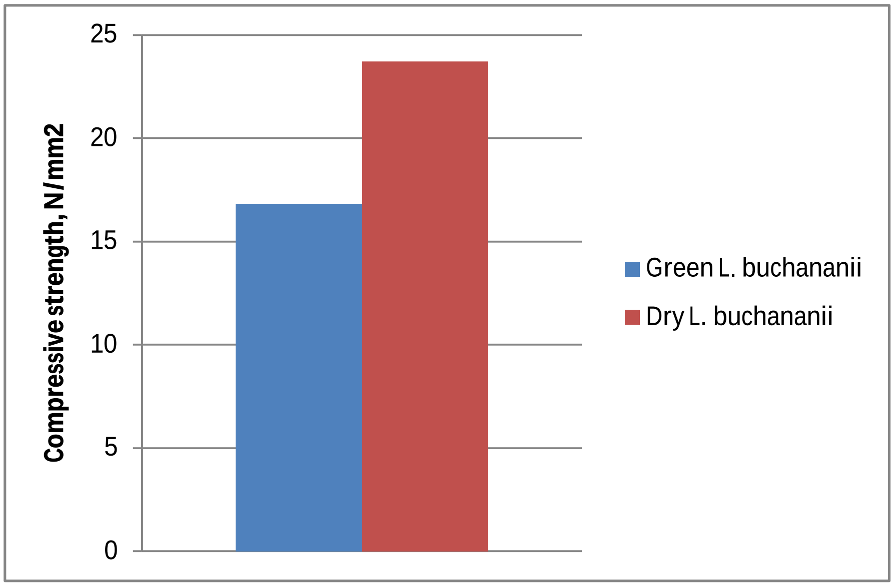

Figure 4.5: Comparison of compressive strength

Table 4.4: Bending test results

	GREEN SAMPLE			DRY SAMPLE		
Sample no	**Elastic modulus (at deflection 0-2.5mm) (N/mm^2)**	**Elastic modulus (at deflection 2.5-14mm) (N/mm^2)**	**Bending strength (N/mm^2)**	**Elastic modulus (at deflection 0-2.5mm) (N/mm^2)**	**Elastic modulus (at deflection 2.5-14mm) (N/mm^2)**	**Bending strength (N/mm^2)**
1	3620.9	2234.3	33.079	2506.71	2481.65	47.766
2	2549.5	1480.7	24.681	3801.12	2368.45	45.923
3	1990.2	1673.8	25.726	2721.52	2002.9	40.413
4	3287.3	1655.3	29.393	2857.64	2324.78	47.766
5	3875.5	2456.7	16.796	3450.19	2395.22	47.766
6	6176.8	3509	15.751	4029.98	2647.82	52.668
7	3836.1	2533.1	19.418	3000.36	2114.24	47.766
8	4435.8	2198.2	12.597	3801.12	2236.5	42.237
9	3676.7	2362.5	34.124	4432.94	3213.36	60.002
10	2539.5	1509.4	25.707	2793.3	1977.01	29.374
11	2479	1718	26.239	3143.09	2610.75	56.335
12	3308.5	1705.5	30.457	3933.21	2474.31	53.884
13	3875.5	2574.5	17.841	2399.61	2460.13	48.374
14	6431.2	3786.3	17.309	3523.99	2330.25	45.296
15	3890.4	2668.4	20.463	2838.49	1984.58	41.648
16	4435.8	2246.5	13.129	2857.64	2382.06	48.374
17	3608.8	2190.5	32.015	3450.19	2377.13	47.766
18	2590.9	1462.7	25.194	4029.98	2713.6	52.668
19	2133.1	1715.5	24.681	3000.36	2259.02	47.766
20	3287.3	1645.1	28.88	3801.12	2203.87	41.021
21	3875.5	2635.3	18.373	4432.94	3213.36	60.002
22	6431.2	4248	17.309	2793.3	2011.46	31.844
23	3984.2	2690.8	20.482	3143.09	2661.26	55.1
24	4435.8	2329.7	14.174	3933.21	2524.87	53.884
25	1587.2	879.52	18.373	3067.36	2404.98	50.198
MEAN	**3693.71±1269.1**	**2244.37±771.0**	**22.5±6.5**	**3349.7±591.5**	**2414.9±320.2**	**47.8±7.4**

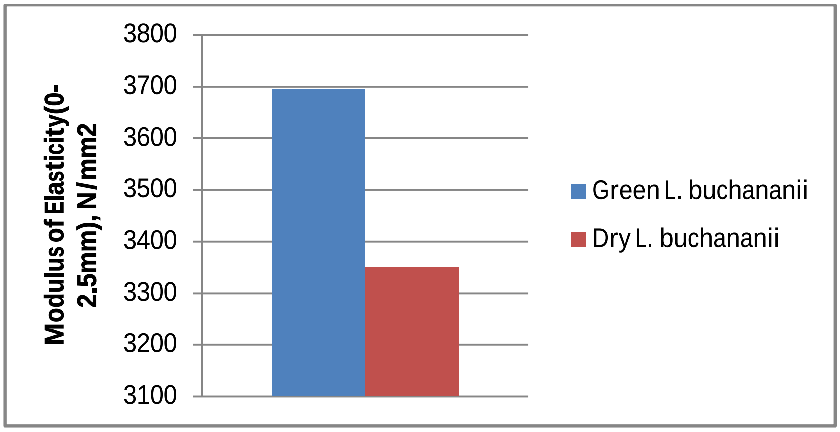

Figure 4.6: Comparison of Modulus of elasticity (0-2.5 mm deflection)

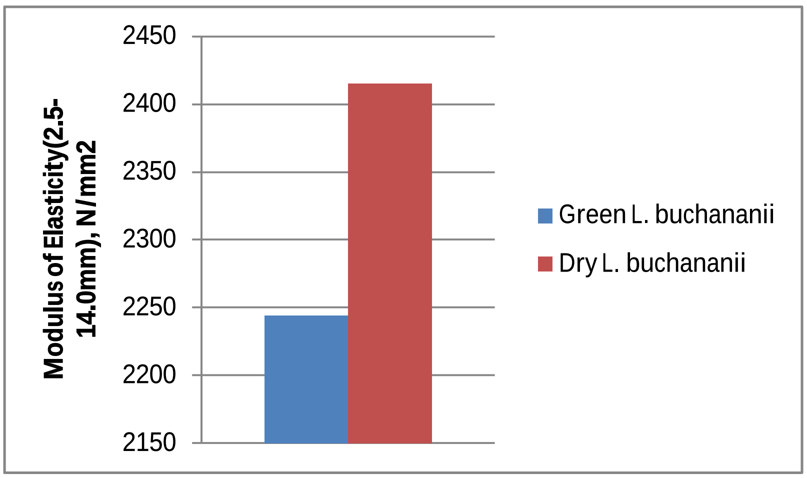

Figure 4.7: Comparison of Modulus of elasticity (2.5-14 mm deflection)

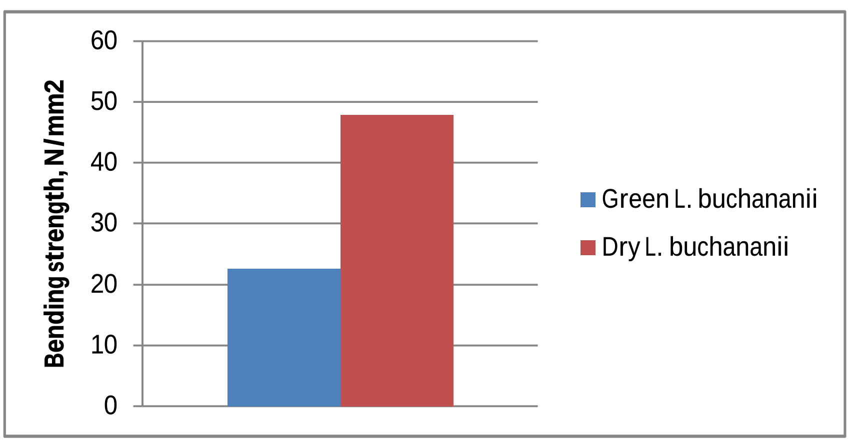

Figure 4.8: Comparison of bending strength

Table 4.5: Shear test results

	GREEN SAMPLES	DRY SAMPLES
Sample no	**Shear strength (N/mm^2)**	**Shear strength (N/mm^2)**
1	4.575	8.325
2	5.05	7.075
3	5.45	9.5
4	4.8	9.625
5	7.775	9.675
6	5.69	9.55
7	4.5	8.35
8	6.375	9.575
9	5.225	11.675
10	4.375	10.45
11	4.075	10.55
12	5.2	9.625
13	7.1	9.475
14	4.875	9.825
15	4.825	11.425
16	5.725	8.275
17	4.525	10.725
18	8.1	9.975
19	5.875	11.475
20	4.875	9.775
21	5.9	8.825
22	6.675	9.7
23	5.125	11.225
24	4.925	10.6
25	4.9	11.425
26	6.625	10.15
MEAN	**5.5±1.0**	**9.8±1.1**

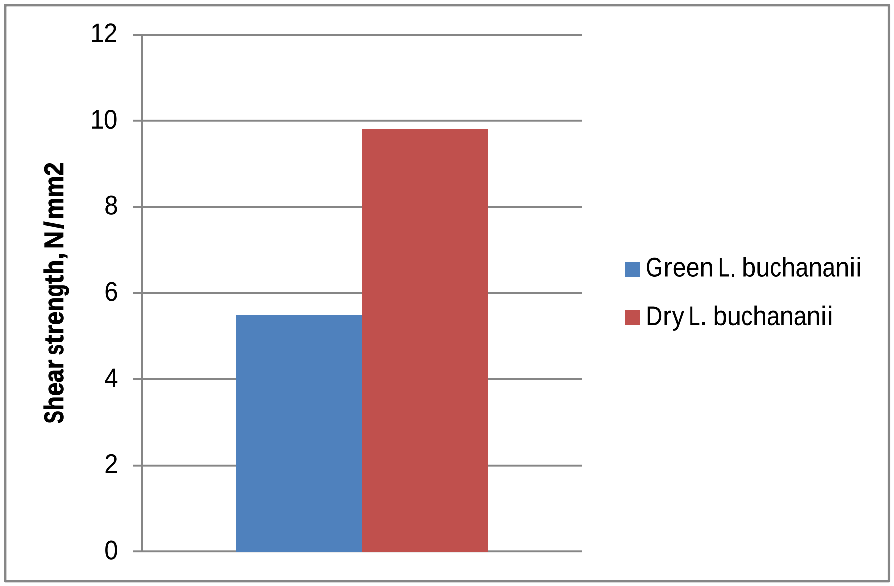

Figure 4.9: Comparison of shear strength

Table 4.6: Janka hardness test results

	GREEN SAMPLES	**DRY SAMPLES**
Sample no	**Force, N**	**Force, N**
1	2790	3520
2	3200	3630
3	2230	3330
4	2130	3610
5	2520	3720
6	2610	3740
7	2160	3580
8	3020	3380
9	2970	3570
10	2130	3840
11	2290	4490
12	2670	4100
13	2750	3590
14	1820	3340
15	3020	3980
16	2970	3180
17	2130	3570
18	2290	4320
19	2670	3300
20	2750	3340
21	1820	2990
22	2380	3530
23	2590	3480
24	2860	4470
25	1920	3690
26	1870	4040
MEAN	**2483.1±412.0**	**3666.5±377.6**

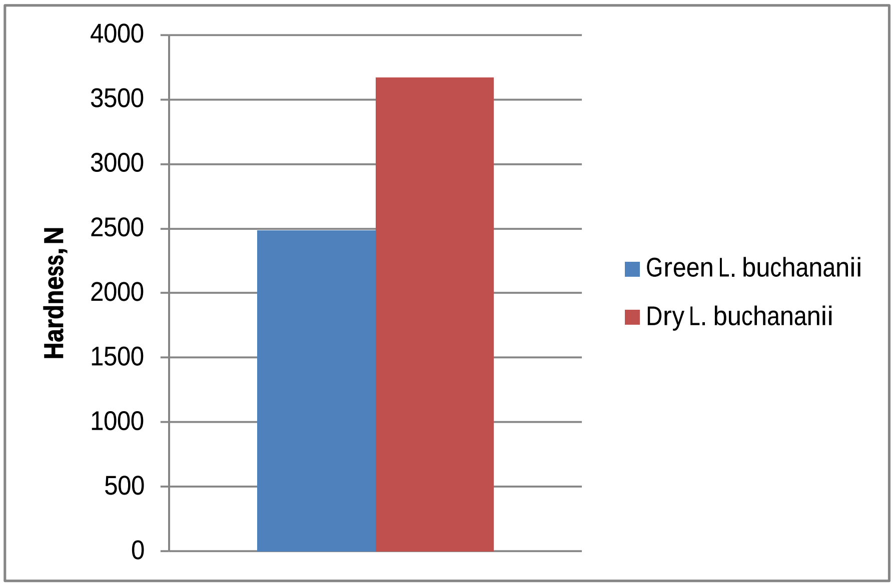

Figure 4.10: Comparison of hardness

Plate 4.1: Tensile test samples mode of failure

Plate 4.2: Beam test specimen under loading

Plate 4.3: Beam test sample after loading

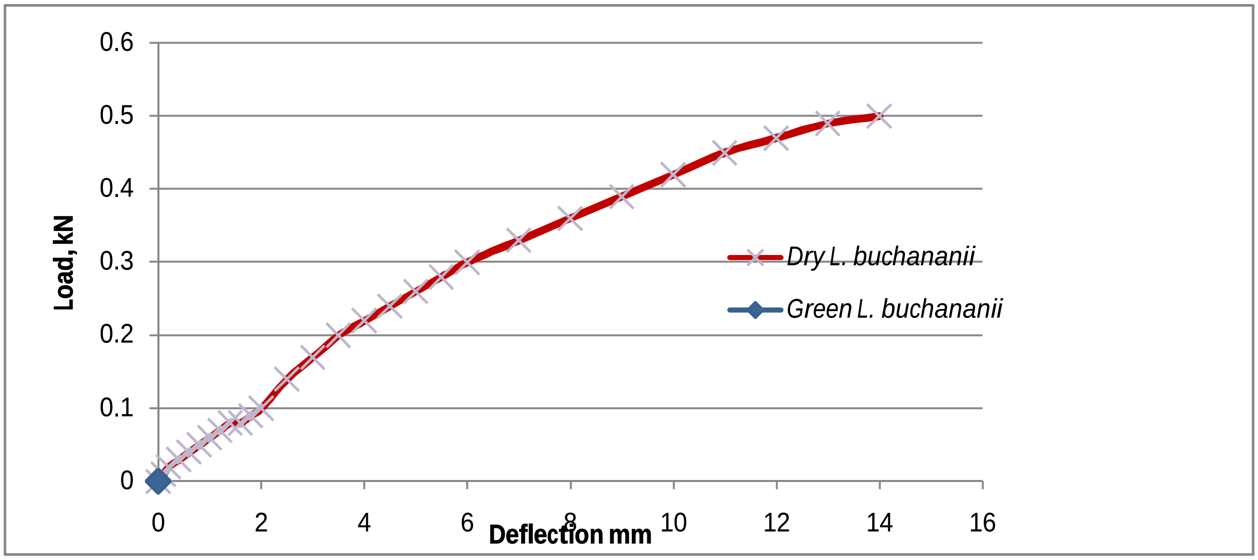

Figure 4.11: Load-deflection curve of typical samples of *L. buchananii* from bending test

Table 4.7: Summary of the physical and mechanical test results

Property	Test	Samples no.	Property determined	Average Green	Average Dry
Physical property	Moisture content	25	Moisture content, %	56±3	13±1
	Dry density	25	Dry density, g/cm^3	0.55±0.06	0.46±0.04
	Density	25	Density, g/cm^3	1.26±0.07	0.53±0.05
Mechanical properties	Tensile test	25	Tensile strength, N/mm^2	25.8±7.9	87.2±11.5
	Compression test	25	Compression strength, N/mm^2	16.8±2.9	23.7±2.5
	Shear test	25	Shear strength, N/mm^2	5.5±1.0	9.8±1.1
	Hardness test	25	Hardness, N	2843±412	3666±378
	Bending test	25	Modulus of elasticity, N/mm^2	3693.71±1269.1	3349.7±591.5
		25	Modulus of rupture, N/mm^2	22.5±6.5	47.8±7.4

Table 4.8: Maximum and minimum values of the physical and mechanical properties

Property	Test	Property determined	Dry		Green	
			Max.	**Min.**	**Max.**	**Min.**
	Moisture content	Moisture content, %	16.4	12.1	60.1	51.1
	Dry density	Dry density, g/cm^3	0.54	0.39	0.69	0.47
	Density	Density, g/cm^3	0.62	0.44	1.42	1.15
Mechanical properties	Tensile test	Tensile strength, N/mm^2	112.6	65	44.7	4.1
	Compression test	Compression strength, N/mm^2	30.0	20.2	22.3	12.2
	Shear test	Shear strength, N/mm^2	11.7	7.1	8.1	4.1
	Hardness test	Hardness, N	4490	2990	3200	1820
	Bending test	Modulus of elasticity, N/mm^2	60.0	29.3	34.1	13.1
		Modulus of rupture, N/mm^2	4432.9	2399.6	6431.2	1587.2

4.3 Pull out test

4.3.1 Introduction

Pull out test was conducted to determine the interfacial bond strength of reinforcement in concrete. This section presents the results obtained from *Landolphia buchananii* stems with varying diameters and embedded depth which were tested at the age of 28 days.

4.3.2 Results, Analysis and Discussion

Interfacial bond strength of dry *Landolphia buchananii* and steel in concrete was tested at age of 28 days and the results were as shown in table 4.9. From the results, the average interfacial bond strength for *Landolphia buchananii* was found to be 1.15 N/mm^2 with that of steel at 8.56 N/mm^2. The results shows that bond strength of *Landolphia buchananii* was around 13% that of steel (figure 4.12).

It was observed that the interfacial bond strength of steel was higher than that of *Landolphia buchananii* in concrete. This can be attributed to the fact that steel does not absorb water hence not swelling in wet concrete and again steel being twisted improves bonding in concrete. It can be seen however, in plate 4.5 that the interfacial bond strength of steel in concrete is high enough to pull out concrete hence creating a larger hole due to the steel reinforcement. On the other hand, *Landolphia buchananii* is round and untwisted and can absorb water and swell causing low bonding strength. It can be seen from plate 4.4 that the *Landolophia buchananii* did not pull out any concrete, the size of the hole fits the diameter of the reinforcement meaning that the low bonding strength can be attributed to swelling and the untwisted nature of the *Landolphia buchananii* stem.

Plate 4.4: Mode of interfacial bond failure of *L. buchananii* in concrete

Plate 4.5: Mode of interfacial bond failure of steel in concrete

Table 4.9: Pull-out test results

	Steel		
S/no	**Force, N**	**Surface Area embedded, mm^2**	**Bond Strength, N/mm^2**
1	19166	2262.7	8.47
2	17331	1697	9.34
3	11611	1538.6	7.89
	Average		**8.56**
	Landolphia buchananii		
	Force, N	**Surface Area embedded, mm^2**	**Bond Strength, N/mm^2**
1	2746	2828.5	0.97
2	2951	2728	1.08
3	2746	2262.8	1.21
4	2860	2121.4	1.35
5	2758	2941.7	0.94
6	3156	2294.2	1.38
7	2814	2640	1.07
8	4774	3740	1.28
	Average		**1.15**

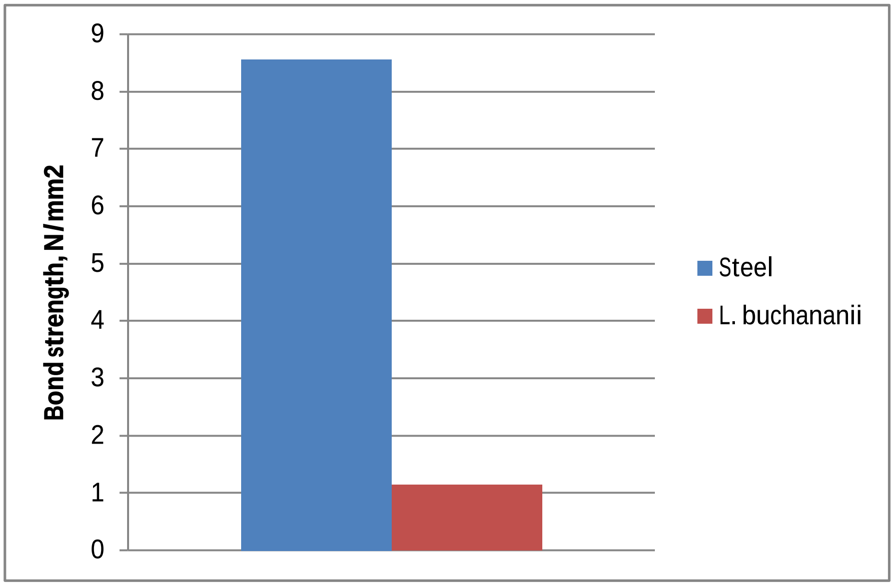

Figure 4.12: Comparison of bond strength

4.4 *Landolphia buchananii* reinforced concrete beam test

4.4.1 Introduction

The results, analysis and discussion of *Landolphia buchananii* and steel reinforced concrete beams are divided in two groups in this section. First group consist of beams reinforced with *Landolphia buchananii* as a tensile reinforcement while the second group consisted the beams reinforced with *Landolphia buchananii* as shear reinforcement. Each of the two groups of reinforced concrete beams had a steel reinforced beam as a control. Compressive tests results for the concrete cubes are also presented, analyzed and discussed in this section.

4.4.2 Compressive tests results

The compressive strength and slump values were as shown in table 4.10. The mean compressive strength of concrete cubes at the age of 28 days was 20.8 N/mm^2 which was higher than the target strength of 20N/mm^2. The mean slump value was 34mm which was between 31-37mm.

Table 4.10: Concrete cubes compressive test results

	Compressive strength, N/mm^2		
S/no	**Batch 1**	**Batch 2**	**Batch 3**
1	21.47	18.37	21.62
2	22.79	19.33	21.02
3	21.7	20.31	21.2
Slump, mm	31	37	34
Average compressive strength per batch N/mm^2	**21.98667**	**19.33667**	**21.28**
Average compressive strength for all batches, N/mm^2	**20.8**		

4.4.3 *Landolphia buchananii* as a tensile reinforcement beams

There were six beams, each with shear reinforcement of 8 mm diameter mild steel provided at a spacing of 200 mm center to centre throughout the length of the beam. The size of the beams was

1100 mm length with 150 mm width and 250 mm depth. The areas of tensile reinforcements were 805(2.14%), 787(2.1%), 741(2%), 710(1.9%), 419(1.11%), 399(1.1%) mm^2. For the top reinforcements, two *L. buchananii* stems of diameter 10mm were used for all specimens. The following notations have been applied for ease of identification in this section:

L.B.805: Beam with 805 mm^2 area of reinforcement of *L.buchananii*.

L.B.787: Beam with 787 mm^2 area of reinforcement of *L.buchananii*.

L.B.741: Beam with 741 mm^2 area of reinforcement of *L.buchananii*.

L.B.710: Beam with 710 mm^2 area of reinforcement of *L.buchananii*.

L.B.419: Beam with 419 mm^2 area of reinforcement of *L.buchananii*.

L.B.399: Beam with 399 mm^2 area of reinforcement of *L.buchananii*.

Steel 101: Beam with 101 mm^2 area of reinforcement of steel.

4.4.3.1 L.B. 805 beam

The results from the test were as presented in table 4.11. The first crack on concrete occurred at a load of 7.5 kN at deflection of 0.19 mm, the second crack on concrete occurred at a load of 16.4 kN at a deflection of 5.71 mm. The ultimate load was 19.6 kN at a deflection of 9.38 mm. The maximum bending stress achieved was 2.83 $N/mm^{2.}$

From figure 4.13, it can be seen that there was minimal strain on both concrete and reinforcement until when the first crack occurred at 7.5kN, the reinforcement strain increased even with no increased in load before it started increasing with load increase with some relaxation. This increase in strain in the reinforcement can be attributed to cracking. The strain gauge on concrete stopped recording when the first crack occurred because the crack damaged the strain gauge. From figure 4.14, it can be observed that at there was minimal deflection at initial stages till when the first crack occurred at 7.5kN when it increased rapidly even when the load decreased. Thereafter deflection increased with load increased until at second crack load was reached at 16.4 kN where again the deflection was observed to increase even with decrease in load. The deflection increased until the ultimate load at 19.6 kN was reached. The first crack occurred at the bottom mid-span of the beam due to bending widening progressively as the loading was increased. The second crack occurred just before the failure at around 200mm from the mid-span as shown in plate 4.6.

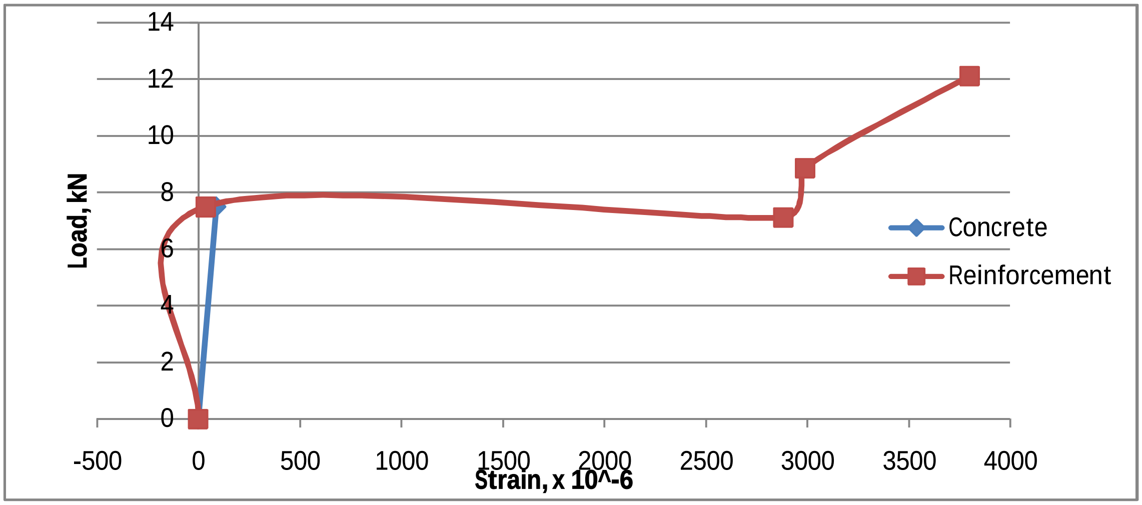

Figure 4.13: Load-strain curves for L.B.805 beam

Table 4.11: Results from experimental loading for L.B.805 beam

L.B.805 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain , x 10^{-6}**	**Concrete strain, x10^{-6}**	**Bending stress, N/mm^2**
0	0	0	0	0
7.5	0.19	36.7	87.3	1.08
7.1	1.16	2882.5		1.03
8.9	1.74	2988.7		1.28
12.1	3.52	3798.6		1.75
14.8	4.56			2.12
16.4	5.71			2.36
16.3	6.52			2.34
18.3	7.94			2.63
19.6	9.38			2.83
	CAUTION: Concrete strain appeared to be defective or damaged on this results			

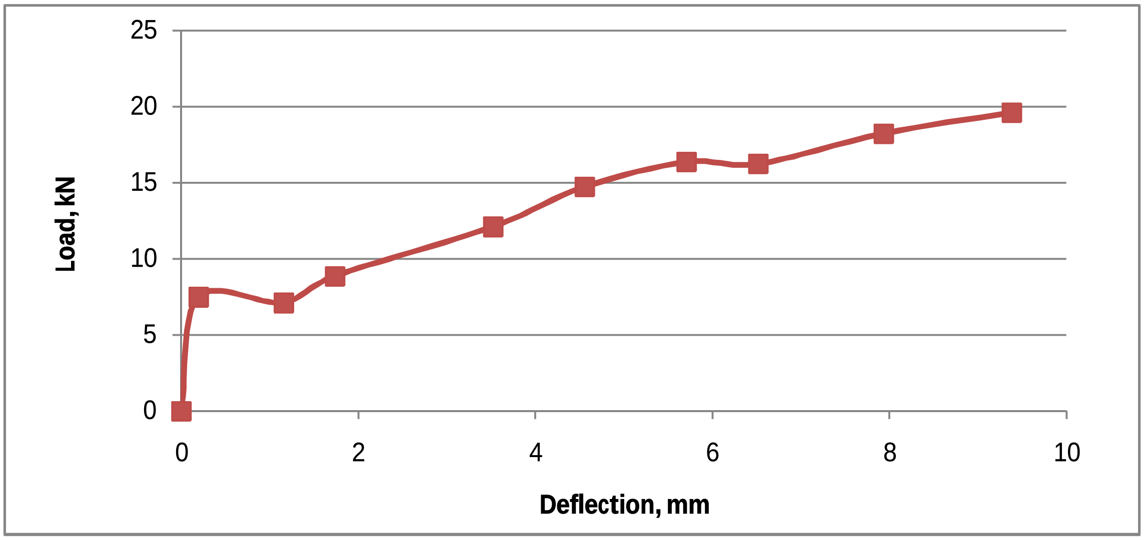

Figure 4.14: Load-deflection curve for L.B.805 beam

Plate 4.6: Failure pattern of L.B.805 beam

4.4.3.2 L.B 787 beam

In the case of the L.B 787 beam, it was observed that the first crack on concrete occurred at a load of 8.15 kN at a deflection of 0.9 mm. However, the ultimate load was 17.38 kN at a deflection of 12 mm. The maximum bending stress achieved was 2.5 N/mm^2. The results from the test were as shown in table 4.12.

From the results in figure 4.15, it can be observed that there was minimal strain in both concrete and reinforcement until when the loading reached 4 kN. At a load of 4 kN the concrete strain started to increase up to when a load of 6.5 kN was attained at which the reinforcement strain started increasing with increase in loading till when the first crack occurred at a load of 8.15 kN. After the first crack load, the reinforcement strain increased even when loading decreased but thereafter the reinforcement strain increased with load increase up to a load of around 12.5 kN. At 12.5 kN load the reinforcement strain increased with decreased in load before it increased with increase in load. This increase in reinforcement strain with decrease in loading can be attributed to failure in concrete, hence transferring the resistance to the reinforcement. It can be observed that there was a relaxation just before the reinforcement strain gauge stop recording.

Consequently, in figure 4.16, it can be observed that the concrete strain kept increasing even after the first crack at a load of 8.15 kN. The negative gradient of the curve line corresponds to either crack occurrence or crack widening. The reinforcement strain gauge stopped recording because it peeled from the reinforcement. From figure 4.17, it can be observed that there was minimal deflection at initial stages up to when the load reached around 6.5 kN when it started increasing with increase in load till the ultimate load was reached at 17.38 kN. After the first crack occurrence, the crack widen progressively to the point when the ultimate load was reached at 17.38 kN. The first crack occurred at the bottom mid-span of the beam due to bending as shown in the plate 4.7.

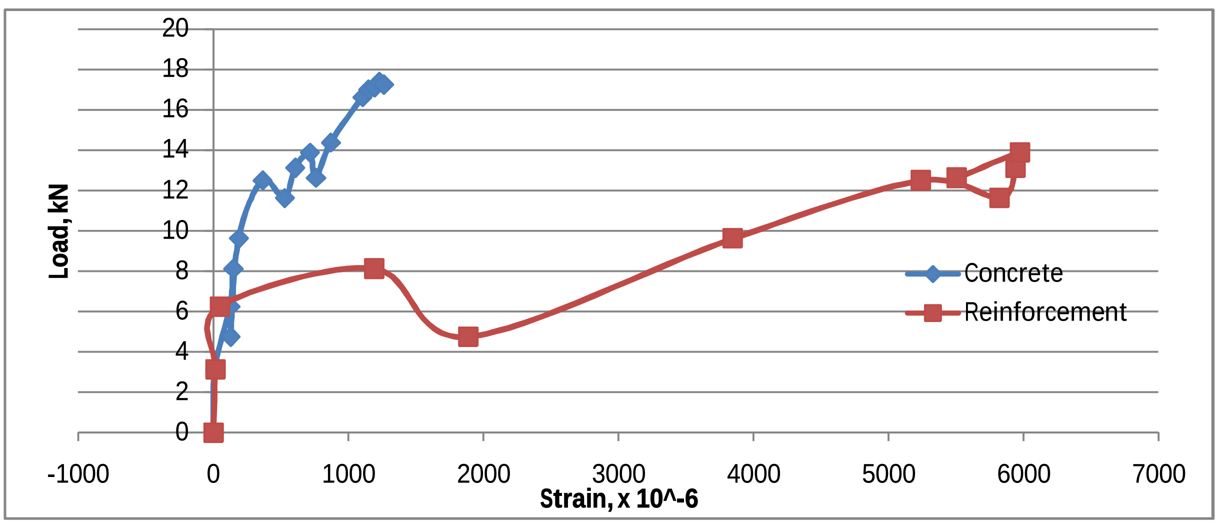

Figure 4.15: Load-strain curves for L.B.787 beam

Table 4.12: Results from experimental loading for L.B.787 beam

L.B.787 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, x10^{-6}**	**Concrete strain , x10^{-6}**	**Bending stress, N/mm^2**
0	0	0	0	0
3.13	0.04	15.5	11.73	0.45
6.25	0.05	48.1	123.94	0.9
8.13	0.91	1190.1	146.48	1.17
4.75	1.11	1884.9	126.76	0.68
9.63	2.12	3845.8	187.79	1.39
12.50	3.20	5238.7	362.91	1.80
11.63	4.20	5820.3	527.23	1.67
13.13	4.93	5938.7	605.16	1.89
13.88	5.64	5972.6	714.55	2.00
12.63	6.66	5503.3	757.75	1.82
14.38	7.29		868.55	2.07
16.63	9.56		1102.35	2.39
17.00	10.34		1146.01	2.45
17.13	11.18		1192.02	2.47
17.38	12.02		1227.70	2.50
17.25	12.89		1260.09	2.48

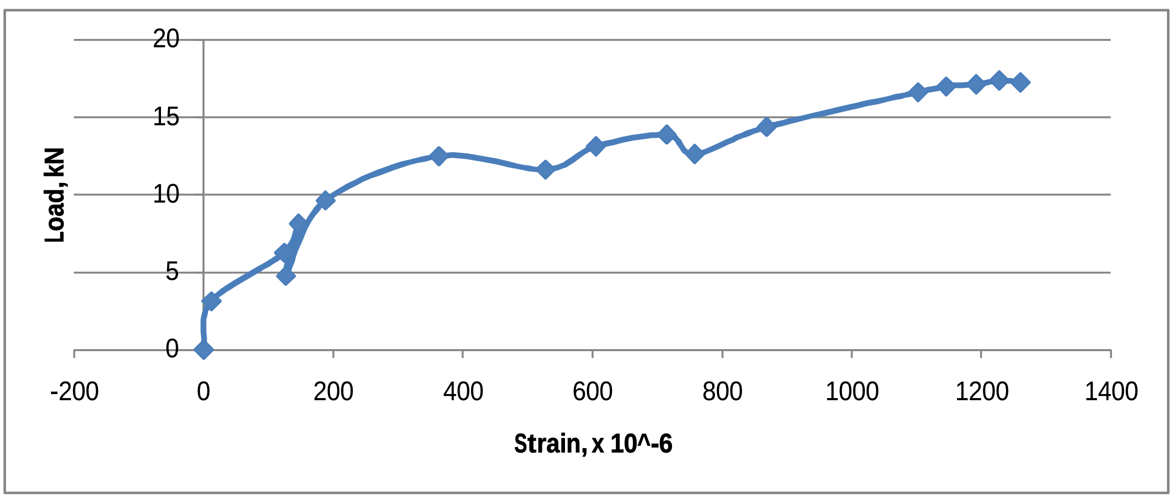

Figure 4.16: Concrete load-strain curve for L.B.787 beam

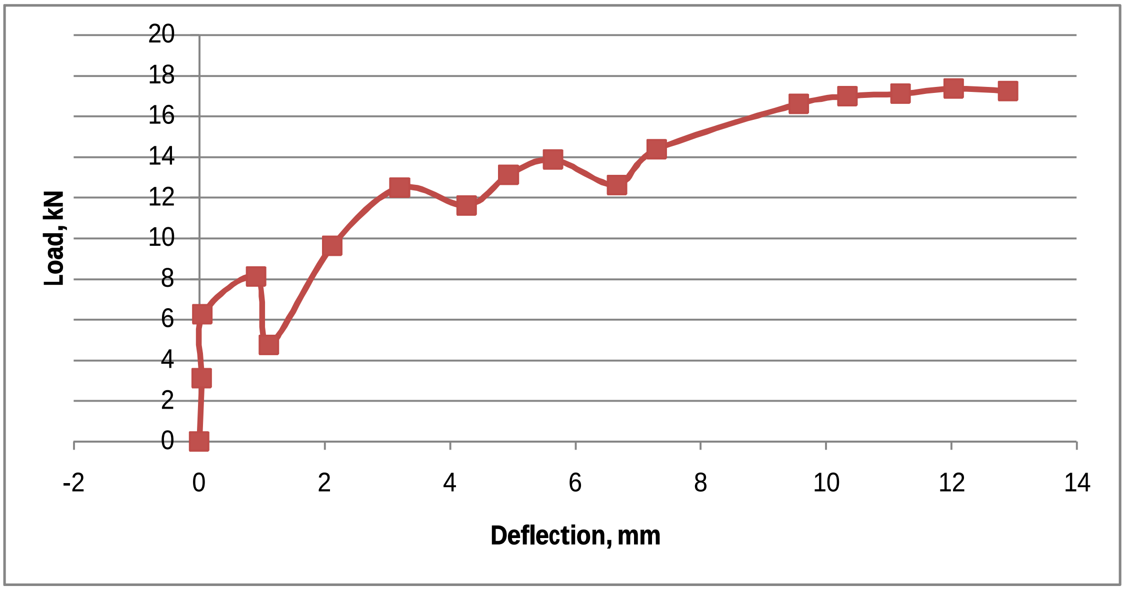

Figure 4.17: Load-deflection curve for L.B.787 beam.

Plate 4.7: Failure pattern of L.B.787 beam

4.4.3.3 L.B. 741 beam

The results based on this indicated that the first crack on concrete occurred at a load of 8.5 kN at a deflection of 0.28 mm while the second crack on concrete occurred at a load of 10.0 kN at a deflection of 9.2 mm. The ultimate load was however found to be 13 kN at a deflection of 16.18 mm with maximum bending stress attained was 1.87 N/mm^2. The results from the test were as shown in table 4.13.

It is clear from the findings in figure 4.18 that there was minimal strain on both concrete and reinforcement up to the point when the load reached around 5.5 kN where the concrete strain started increasing rapidly until when the first crack occurred at a load of 8.5 kN. After the first crack the reinforcement strain increased rapidly even if there was decrease in load because there was failure in concrete thus transferred the resistance to the reinforcement. The reinforcement strain underwent relaxation strain gauge stopped recording by peeling off from the reinforcement. The concrete strain after the first crack underwent relaxation (figure 4.19) before it started increasing with increase in load up to when the load reached around 10 kN a point which the second crack occurred. After the 10 kN load, concrete strain relaxation occurred and continued till when the ultimate load was reached. Conversely, from figure 4.20, it can be seen that at there was minimal deflection at initial stages before the first crack occurred at a load of 8.5kN after which the deflection increased rapidly even when the load decreased. Thereafter deflection increased with increase in load up to when the second crack occurred at a load of 10 kN. At the point of second crack load, the deflection increased proportionally with decrease in load before it started increasing with increase in load up to the point when the ultimate load was attained. Concurrently, the first crack widen progressively until when the second crack on concrete occurred. However, the first crack occurred at the bottom mid-span of the beam due to bending while the second crack occurred just before the failure at around 100mm from the mid-span as shown in the plate 4.8.

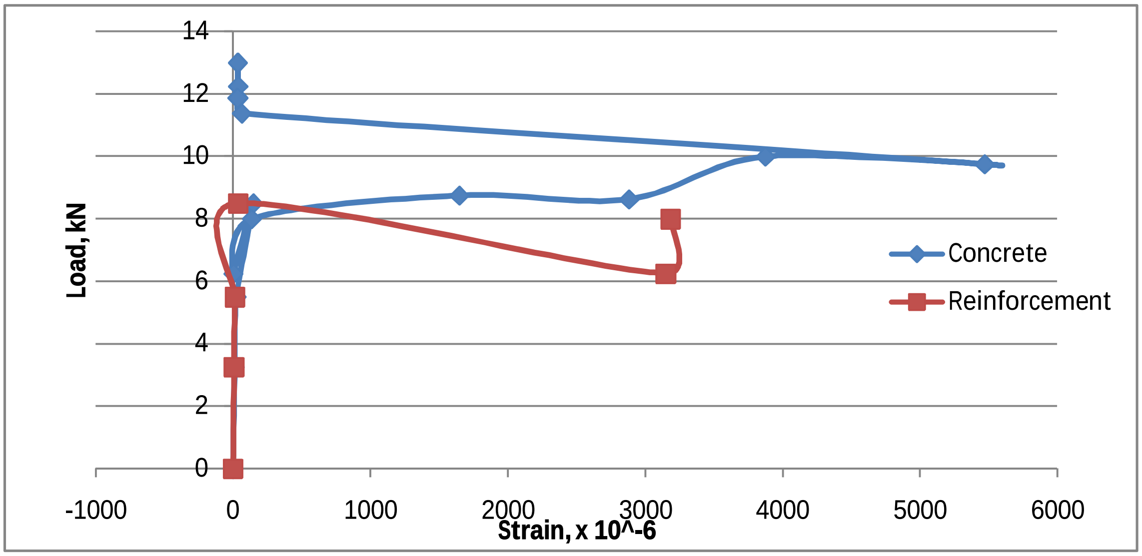

Figure 4.18: Load-strain curves for L.B.741 beam

Table 4.13: Results from experimental loading for L.B.741 beam

L.B.741 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, $x10^{-6}$**	**Concrete strain, $x10^{-6}$**	**Bending stress, N/mm^2**
0	0	0	0	0
3.25	0.08	6.6	11.7	0.45
5.50	0.16	13.2	25.8	0.79
8.50	0.28	35.8	147.9	1.22
6.25	2.45	3152	2.3	0.90
8.00	4.66	3187	135.7	1.15
8.75	6.18		1646	1.26
8.63	7.65		2882	1.24
10.00	9.29		3874	1.44
9.75	11.17		5470	1.40
11.38	12.85		62.9	1.64
11.88	13.70		40.8	1.71
12.25	14.35		37.5	1.76
13.00	16.18		34.7	1.87

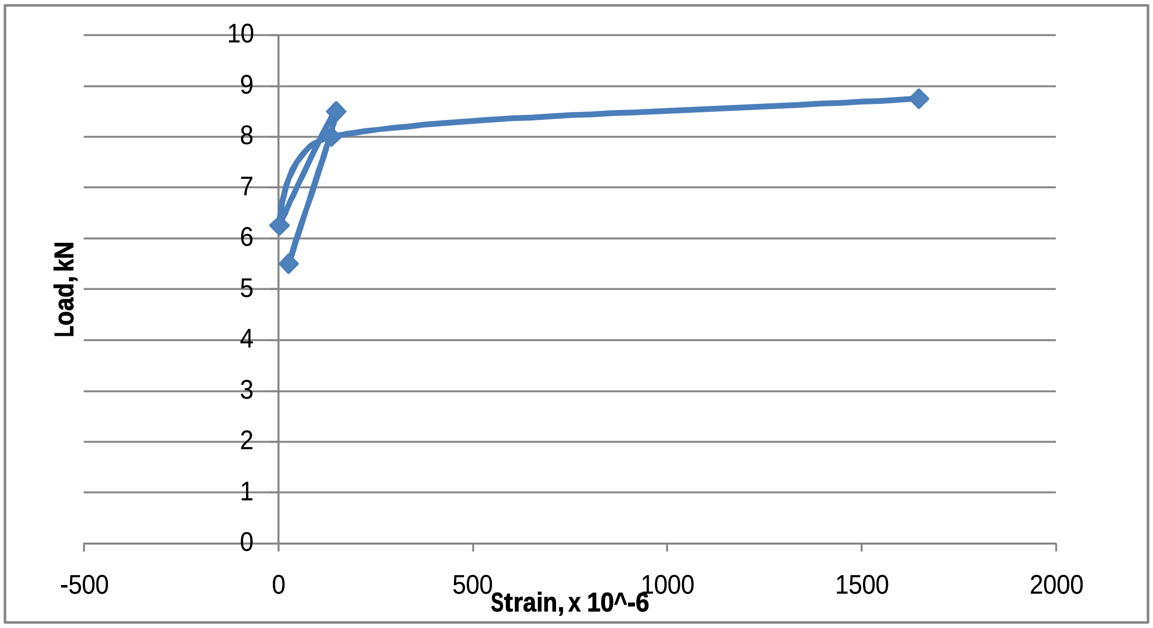

Figure 4.19: Concrete load-strain curve for L.B.741 beam

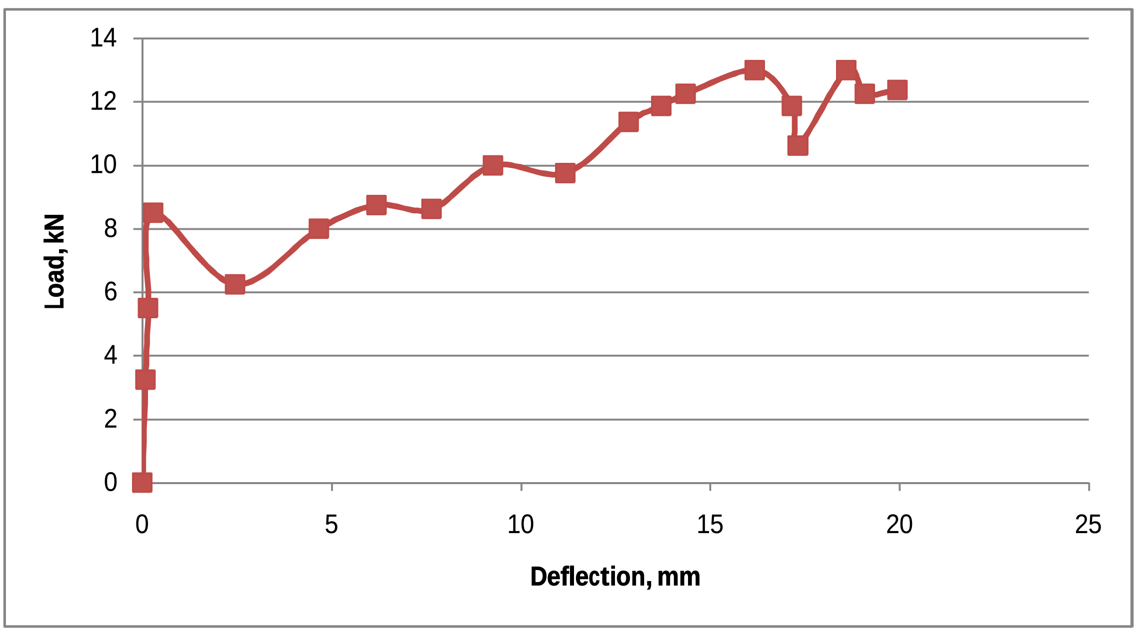

Figure 4.20: Load-deflection curve for L.B.7.87 beam.

Plate 4.8: Failure pattern of L.B.741 beam

4.4.3.4 L.B. 710 beam

The results from the test were obtained and presented as shown in table 4.14 which showed that the first crack on concrete occurred at a load of 7.4 kN and the ultimate load was found to be at 15.5 kN. In addition, the maximum bending stress achieved for this beam was 2.23 N/mm^2. However, the deflection was not recorded by the instrument for this test because the measuring equipment was faulty.

In this case, it was observed that there were minimal strains in both concrete and reinforcement until when the first crack occurred at a load of 7.3kN (see figure 4.21). After which the strain in reinforcement increased with decrease in load and is thus attributed to failure in concrete. The failed concrete perhaps transferred the load resistance to the strain in reinforcement causing it to increase. However, upon occurrence the first crack, the reinforcement strain increased with increase in load to the level when the strain gauge failed. It can be observed in figure 4.22, that there was relaxation of concrete strain after first crack load was reached. Thereafter the concrete strain increased and underwent relaxation till when the ultimate load was reached. On the other hand, the first crack widen gradually as the load was increased. Consequently, the first crack occurred at the bottom mid-span of the beam due to bending as shown in plate 4.9.

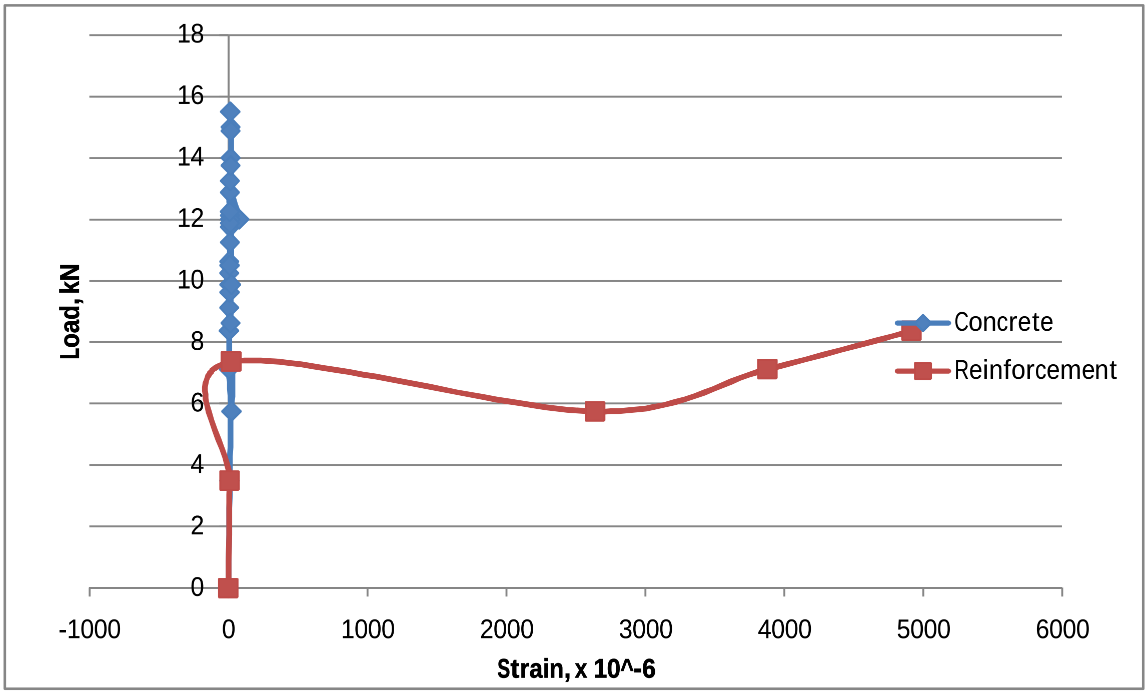

Figure 4.21: Load-strain curves for L.B.710 beam

Table 4.14: Results from experimental loading for L.B.710 beam

L.B.710 Beam			
Load, kN	**Reinforcement strain, x10-6**	**Concrete strain, x10^{-6}**	**Bending stress, N/mm^2**
0	0	0	0
3.50	8.1	8.4	0.50
7.38	20.2	24.1	1.06
5.75	2640.1	23.1	0.83
7.13	3878.3	4.2	1.03
8.38	4914.2	4.7	1.21
9.13		5.6	1.31
9.88		7.1	1.42
10.63		6.1	1.53
10.50		6.6	1.51
10.25		5.2	1.48
9.63		9.0	1.39
10.50		9.9	1.51
11.75		11.3	1.69
12.00		11.8	1.73
11.25		12.3	1.62
12.25		13.2	1.76
12.00		77.4	1.73
12.89		11.3	1.85
12.13		10.8	1.75
12.00		14.2	1.73
8.63		14.2	1.24
11.88		12.3	1.71
12.25		9.9	1.76
14.00		16.5	2.01
9.89		20.8	1.42
13.25		12.3	1.91
13.75		16.0	1.98
15.00		16.0	2.16
15.50		13.7	2.23
14.875		16.0	2.14
	CAUTION: Reinforcement strain appeared to be defective or damaged on this results		

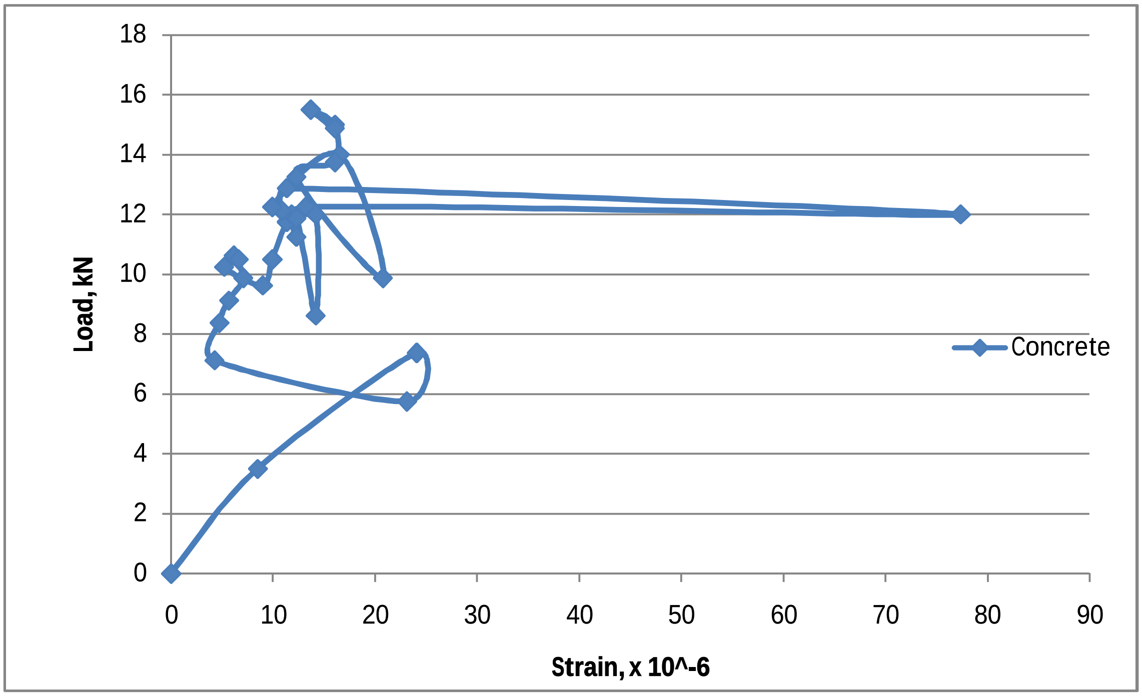

Figure 4.22: Concrete load-strain curve for L.B.710 beam

Plate 4.9: Failure pattern of L.B.740 beam

4.4.3.5 L.B. 419 beam

It can be seen from the test results in table 4.15 that there was an occurrence of the first crack on concrete at a load of 11.9 kN which was also the ultimate load. The maximum bending stress achieved for this beam was 1.71 N/mm^2. However, there no deflection test recorded for this beam because the measuring equipment was defective.

From figure 4.23, it can be seen that the strain was the same for both reinforcement and concrete up to the failure load. Both strain gauges failed to read once the once the ultimate load was reached. The first crack expanded increasingly with increased in load. The first crack occurred at the bottom mid-span of the beam as a result of bending as shown in plate 4.10.

Table 4.15: Results from experimental loading for L.B.419 beam

L.B.419 Beam			
Load, kN	**Reinforcement strain, x10^{-6}**	**Concrete strain, x10^{-6}**	**Bending stress, Nmm^2**
0	0	0	0
7.8	20.7	20.8	1.11
11.9	35.4	36.3	1.71

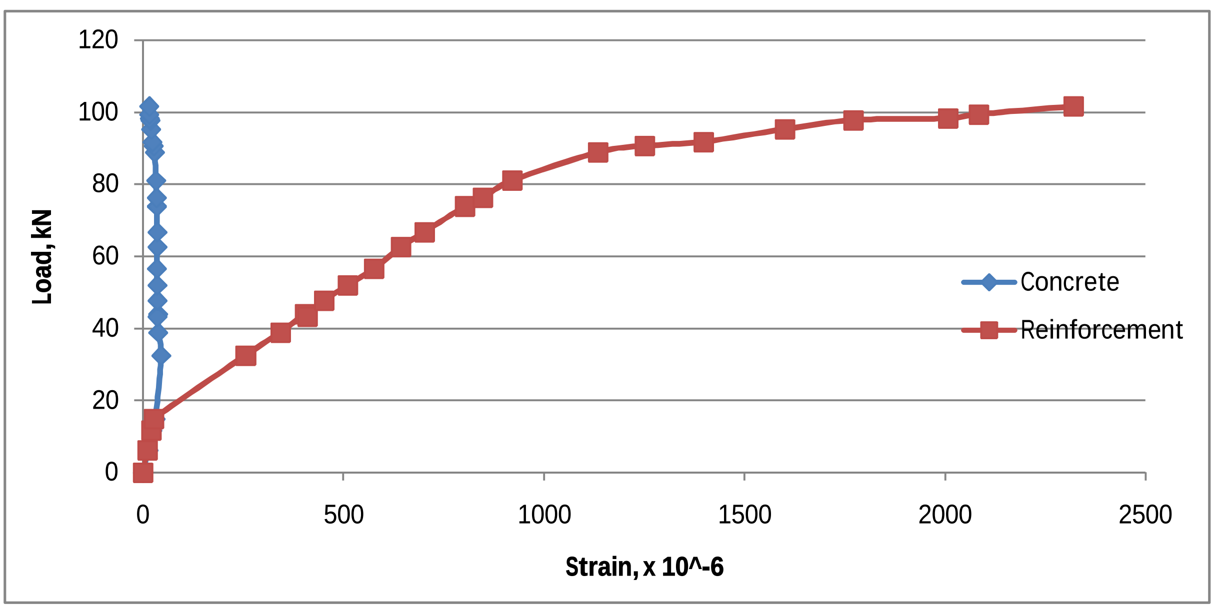

Figure 4.23: Load-strain curves for L.B.419 beam

Plate 4.10: Failure pattern of L.B.419 beam

4.4.3.6 L.B 399 beam

The initial crack on concrete occurred at a load of 11 kN which was also found to be the ultimate load with 1.5 N/mm^2 maximum bending stress attained for this beam. In this test also, there was no deflection recorded since the measuring equipment used was flawed. The results from the test were as presented in table 4.16.

From the results in figure 4.24, it can be observed that the strain in concrete at the beginning was more than strain in reinforcement which can be attributed to concrete resisting the failure more than the reinforcement. Further, it can be seen that the strain in concrete underwent relaxation between 3-8.5 kN load just before the ultimate load of 11 kN was reached. The first crack widened gradually with increase in load and the first crack was observed to have occurred at the bottom mid-span of the beam owing to the bending as displayed in plate 4.11.

Table 4.16: Results from experimental loading for L.B.399 beam.

L.B.399			
Load, kN	**Reinforcement strain, x10^{-6}**	**Concrete strain, x10^{-6}**	**Bending stress, N/mm^2**
0	0	0	0
2.88	1.4	4.2	0.41
8.38	6.1	2.8	1.21
11.00	22.1	17.0	1.58

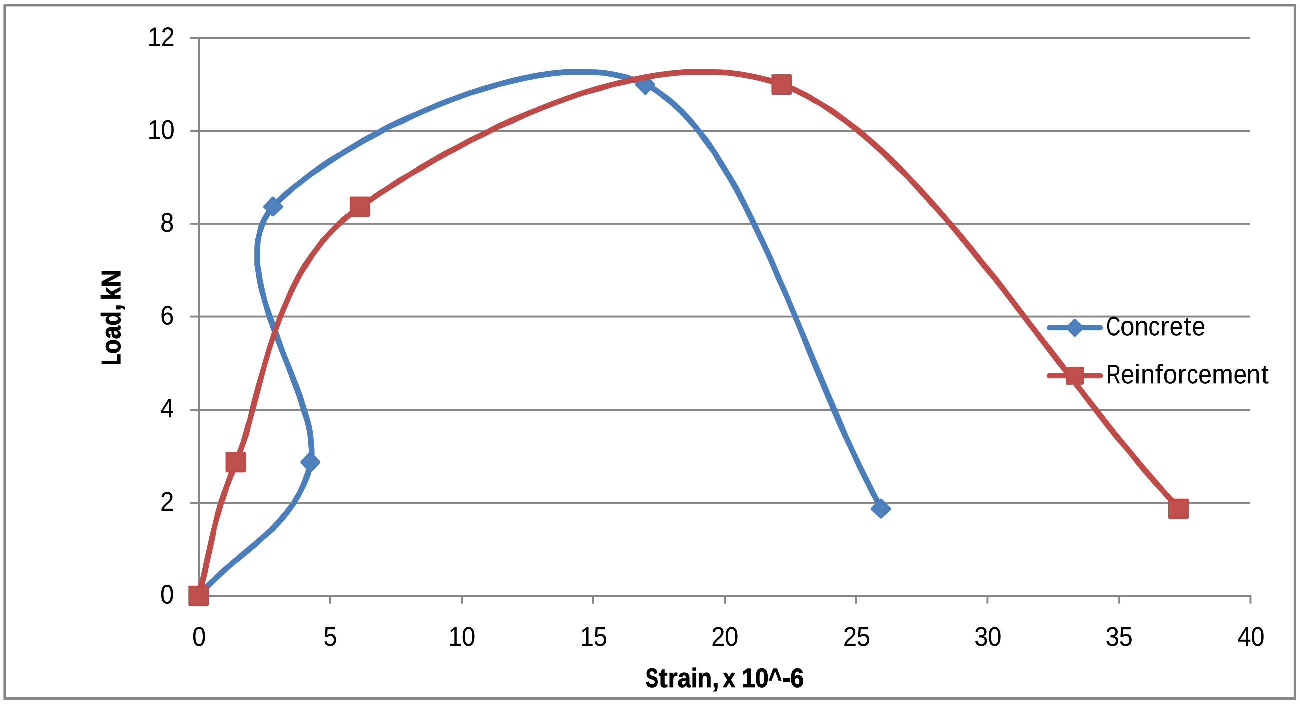

Figure 4.24: Load-strain curves for L.B.399 beam

Plate 4.11: Failure pattern of L.B.399 beam

4.4.3.7 Steel 101 beam

Based on the test results in regards to steel 101 beam, (see table 4.17), it is clearly seen that the first crack on concrete occurred at a load of 56.25 kN which was also the ultimate load at a deflection of 14.94 mm. The maximum bending stress achieved for this beam was 8.1 N/mm^2. The deflection for this beam was not recorded due faulty measuring instrument.

It can be observed that the strain in the reinforcement was more than in concrete (figure 4.25). This means that the reinforcement was the one resisting the failure. In addition, from figure 4.26, it can be seen that the strain in concrete increased as the loading increased but underwent major relaxation when load was at around 29 kN. From the test results in figure 4.27, the strain the reinforcement surpassed strain in concrete when the load exceeded 2.5 kN. Consequently, in figure 4.28, it is observed that the deflection increased linearly as the load increased during initial stages up to load of around 50kN after which the curve became non-linear till failure. However, as the load increased the first crack was observed to increase gradually as well. The results also showed just like in the other test that the first crack occurred at the bottom mid-span of the beam due to bending as seen in plate 4.12.

Table 4.17: Results from experimental loading for Steel 101 beam

Steel 101 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, x10^{-6}**	**Concrete strain, x10^{-6}**	**Bending stress, N/mm^2**
0	0	0	0	0
2.88	0.11	6.1	7.0	0.41
21.13	1.38	541.5	47.9	3.04
24.50	1.61	614.2	88.7	3.53
28.63	1.95	732.6	94.3	4.12
32.88	2.30	905.2	3.2	4.73
36.75	2.70	1082.1	9.8	5.29
42.00	3.29	1467.9	10.3	6.05
45.50	3.87	2104.7	2.3	6.55
47.38	4.37	2907.1	63.3	6.82
49.63	5.24	2024.1	68.0	7.15
49.50	6.23	1133.5	40.8	7.13
50.38	7.05	964.2	133.	7.25
51.25	7.90	707.6	134.7	7.38
52.00	8.64	549.5	132.4	7.49
53.38	8.93	522.6	127.2	7.69
54.50	9.82	500.9	131.9	7.85
54.38	10.62	315.6	135.7	7.83
54.00	11.30	317.5	133.3	7.78
55.25	12.35	330.2	135.2	7.96
55.10	13.11	433.0	131.9	7.94
56.00	14.13	475.9	133.3	8.06
56.25	14.94	504.7	129.1	8.10

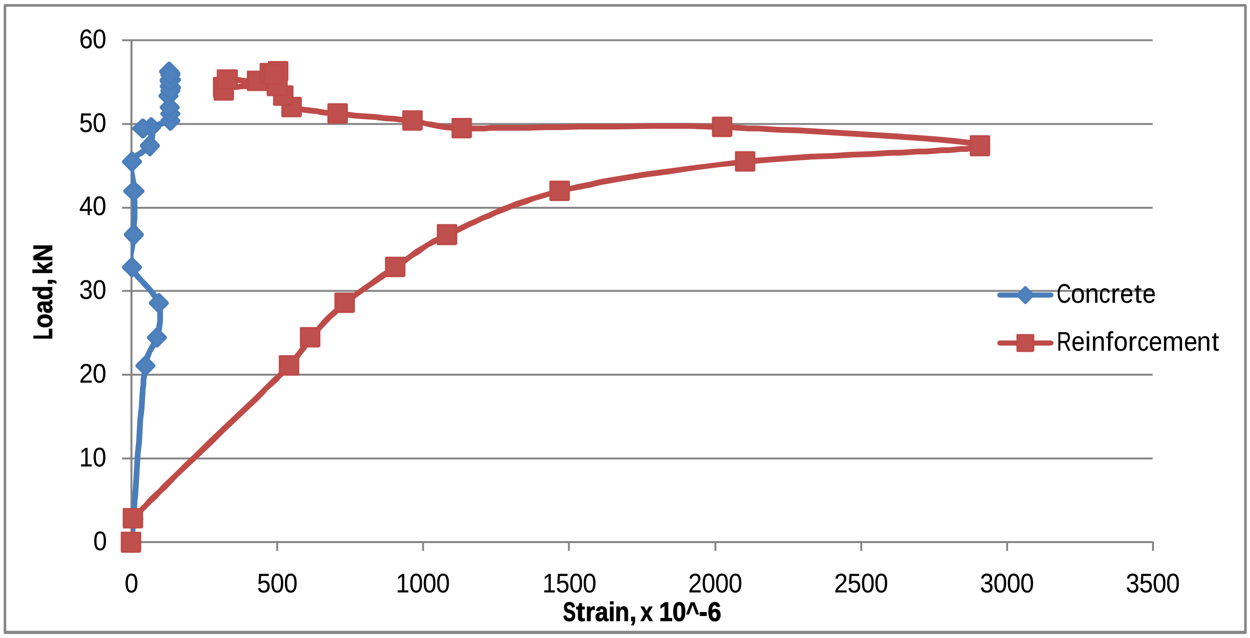

Figure 4.25: Load-strain curves for Steel 101 beam

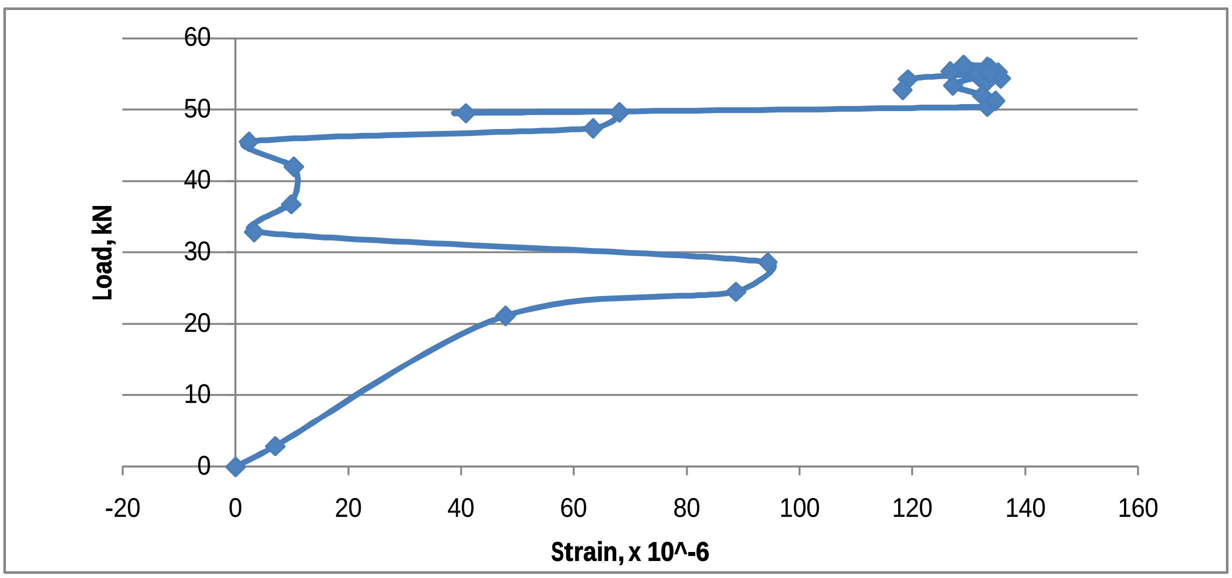

Figure 4.26: Concrete load-strain curve for Steel 101 beam

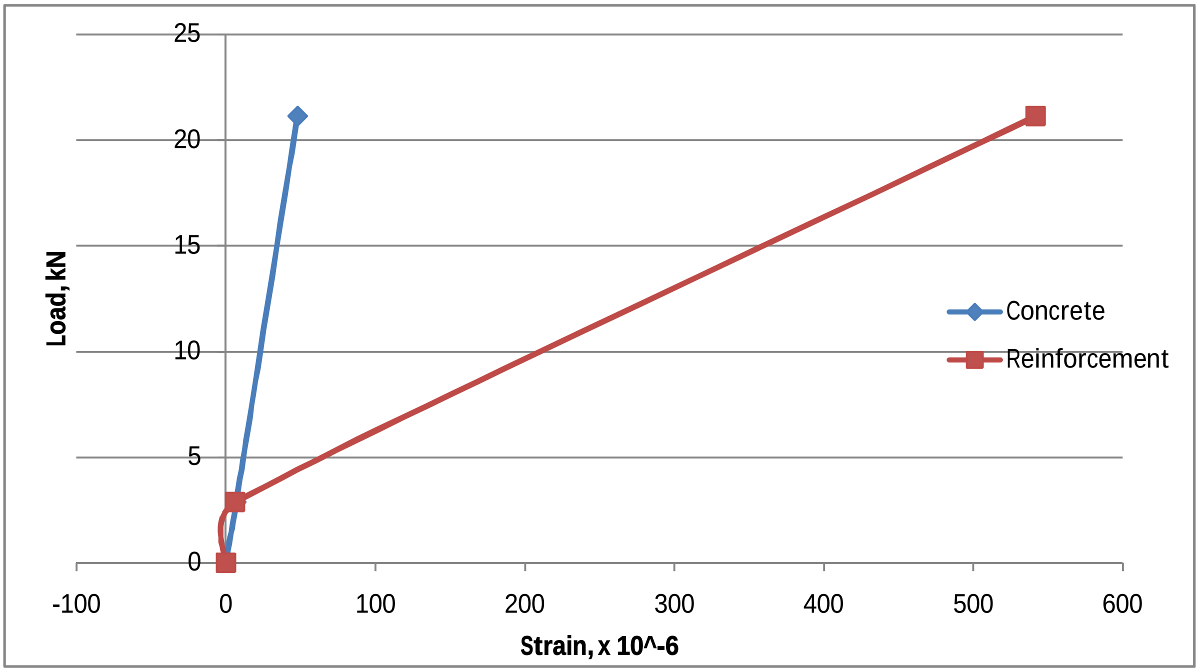

Figure 4.27: Load-strain curves for Steel 101 beam at initial stages

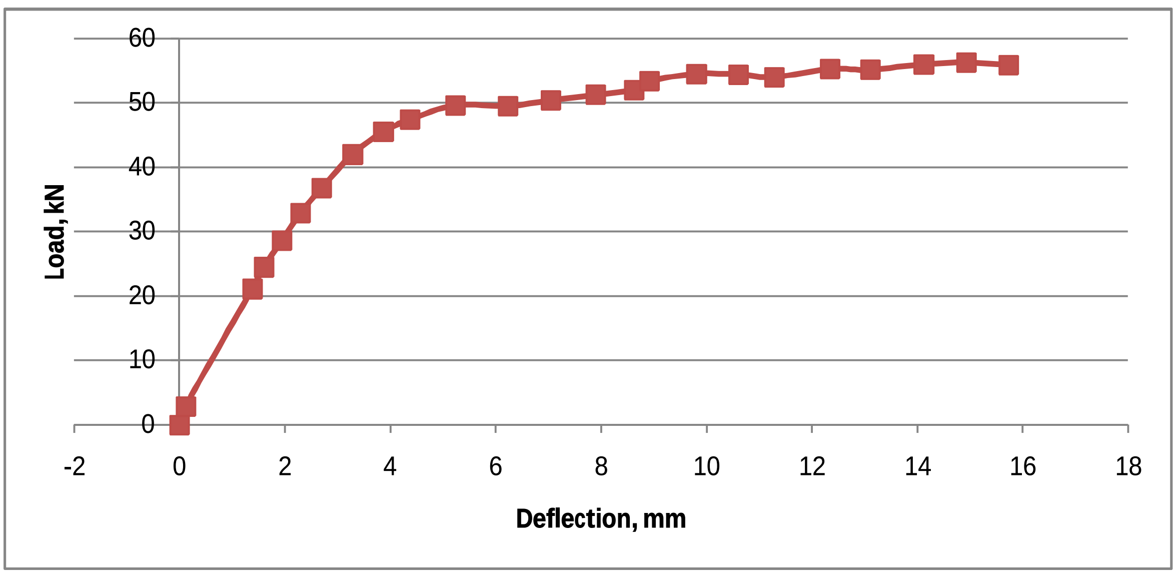

Figure 4.28: Load-deflection curve for Steel 101 beam.

Plate 4.12: Failure pattern of Steel 101 beam

4.4.4 *Landolphia buchananii* as shear reinforcement beams

There were four beams with *Landolphia buchananii* as shear reinforcement, the size of the shear reinforcement was varied from each beam. The following notations have been applied for ease identification of beams in this section:

L.B.1x6: Beam with one *Landolphia buchananii* shear link of diameter 6mm.

L.B.1x8: Beam with one *Landolphia buchananii* shear link of diameter 8mm.

L.B.2x10: Beam with two *Landolphia buchananii* shear link of diameter 10mm.

L.B.2x13: Beam with two *Landolphia buchananii* shear link of diameter 13mm.

Steel 1x6: Beam with one steel shear link of diameter 6mm.

4.4.4.1 L.B.1x6 beam

The test results in relation to this beam showed that, the first crack on concrete occurred at a load of 34.13 kN at a deflection of 1.35 mm where the ultimate load was 58.38 kN at a deflection of 2.9 mm. The maximum bending stress achieved was at 8.4 N/mm^2 while the maximum shear stress achieved was at 0.78 N/mm^2 as presented in table 4.18.

The findings in figure 4.29, clearly shows that the strain increased linearly on the reinforcement up to when a load of 13 kN was reached. The findings also shows that between 13 kN and 31.43

kN load there was strain relaxation and thereafter increased and underwent relaxation again before it reached the ultimate load. As shown in plate 4.13, it can be seen that the shear reinforcement did not snap even after the ultimate failure load was reached. Conversely, from the figure 4.30, it can be observed that the load-deflection curve is generally linear from when the loading started till failure. The deflection increased with increase in load to the level when failure load was reached at 58.38 kN. The first crack however, expanded progressively with increase in load. The first crack occurred at angle of around 45 degrees from the point of loading to the support as shown in plate 4.14.

Table 4.18: Results from experimental loading for L.B.1x6 beam.

L.B.1x6 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, N/mm^2**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**
0	0	0	0	0
3.75	0.09	0.4	0.54	0.05
8.00	0.36	1.4	1.15	0.11
13.00	0.54	2.3	1.87	0.17
25.38	0.95	1.4	3.65	0.34
34.13	1.35	1.8	4.91	0.46
33.00	1.41	4.2	4.75	0.44
42.50	1.92	2.8	6.12	0.57
58.38	2.90		8.41`	0.78

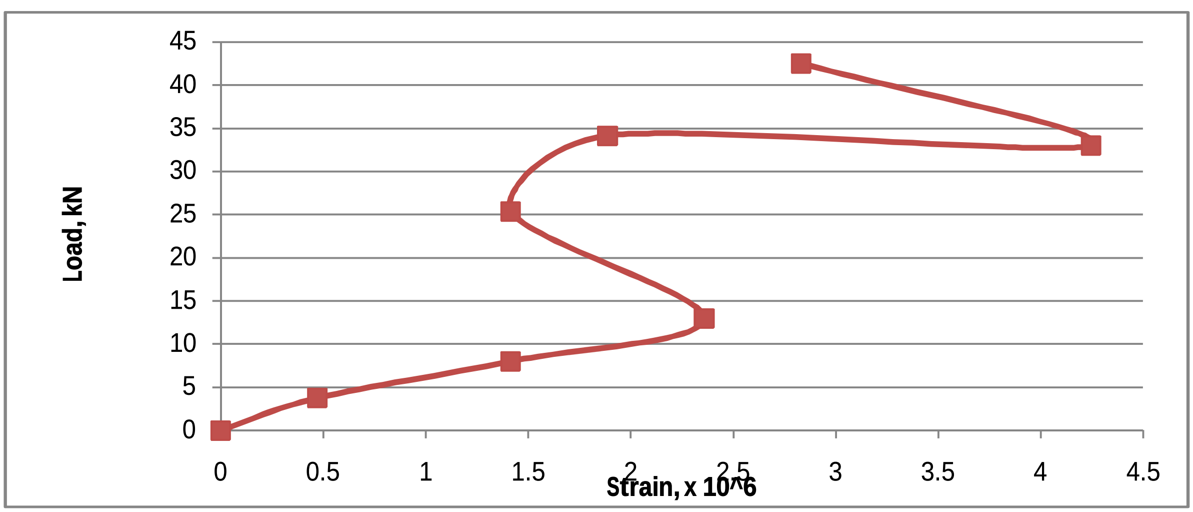

Figure 4.29: Reinforcement load-strain curve for L.B.1x6 beam

Plate 4.13: unbroken shear reinforcement for L.B.1x6 beam

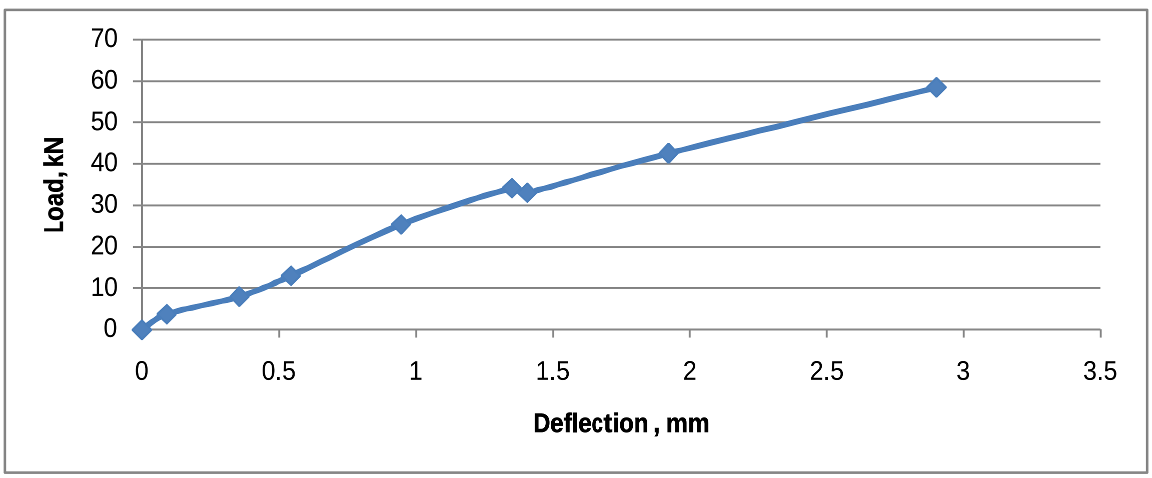

Figure 4.30: Load-deflection curve for L.B.1x6 beam.

Plate 4.14: Failure pattern of L.B.1x6 beam

4.4.4.2 L.B. 1x8 beam

The test results based on L.B. 1x 8 beam were presented in table 4.19. it was observed from the test results that the first crack on concrete occurred at a load of 47.5 kN at deflection point of 0.15 mm. In this case, the ultimate load was at 59.5 kN at a deflection of 0.69 mm. However, the maximum bending stress achieved was 8.6 N/mm^2 while the maximum shear stress achieved was 0.79 N/mm^2

It is observed from the test results in figure 4.31 that, the load-strain curve was non-linear at initial stages before a load of 13 kN was attained. Thereafter the load-strain curve became linear as the strain increased with increase in load till when the ultimate load was reached. As shown in plate 4.15, it can be seen that the shear reinforcement did not snap even after the ultimate failure load was reached. From figure 4.32, it can be seen that the load-deflection curve was generally linear from when the loading started till when the first crack load was reached at 47.5 kN. Subsequently, the deflection increased up to the point when the ultimate load was reached at 59.5 kN. In addition, the test results showed that the first crack widened increasingly until when the failure load was reached and that this crack occurred at angle of about 45 degrees from the point of loading to the support as seen in plate 4.16.

Table 4.19: Results from experimental loading for L.B.1x8 beam

L.B.1x8 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, x10^{-6}**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**
0	0	0	0	0
13.0	0.15	2.36	1.87	0.17
47.5	0.49	16.98	6.84	0.63
59.5	0.69	24.06	8.57	0.79

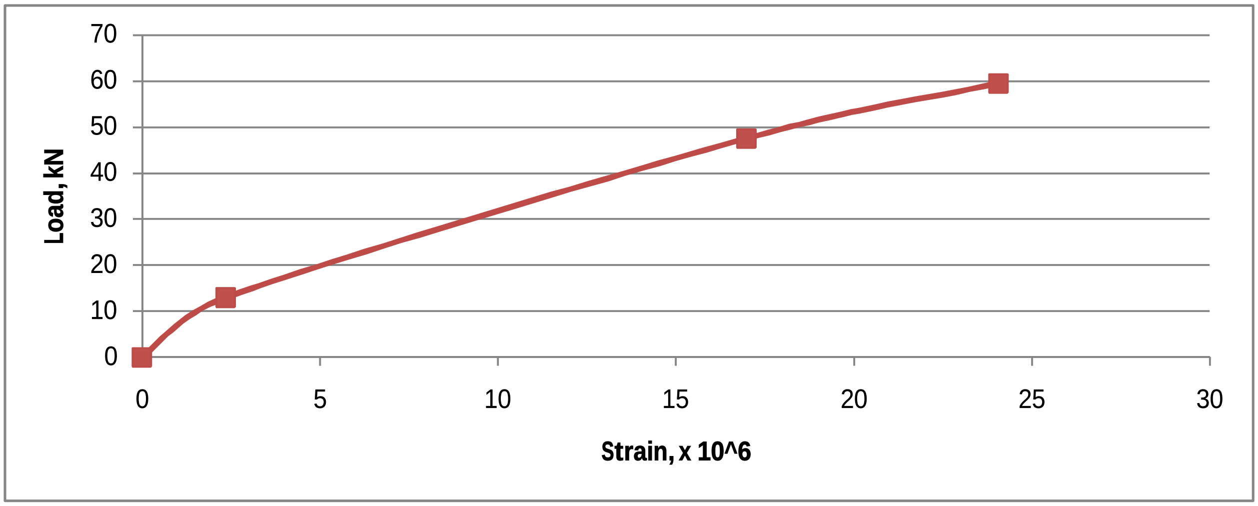

Figure 4.31: Reinforcement load-strain curve for L.B.1x8 beam

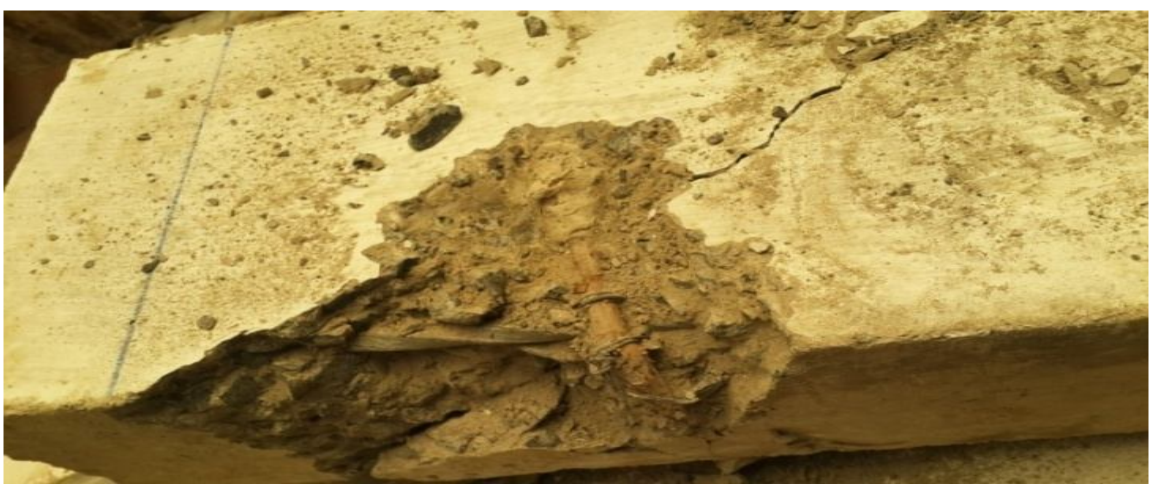

Plate 4.15: unbroken shear reinforcement for L.B.1x8 beam

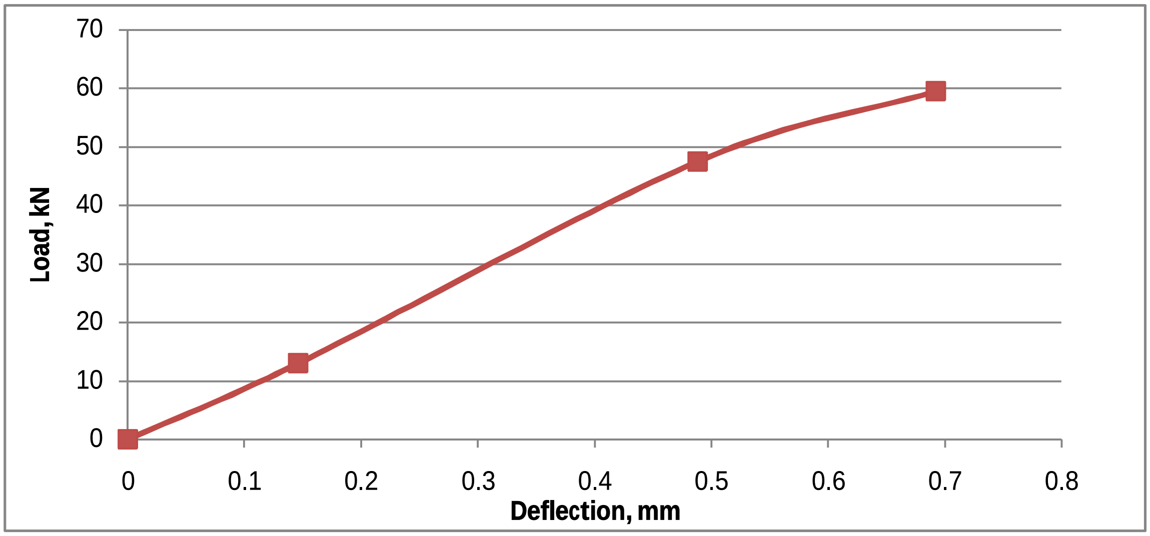

Figure 4.32: Load-deflection curve for L.B.1x8 beam.

Plate 4.16: Failure pattern of L.B.1x8 beam

4.4.4.3 L.B. 2x10 beam

The results from the test were as presented in table 4.20. The first crack on concrete occurred at a load of 31.2 kN at a deflection of 2.94 mm and the ultimate load was 79.8 kN at deflection of 5.9 mm. The maximum bending stress achieved was 11.5 N/mm2 while the maximum shear stress achieved was 1.06 N/mm^2.

From the test result in figure 4.33, it can be seen that the load-strain curve was non-linear. There were series of increment and relaxation of strain on the shear reinforcement as the loading increased till the ultimate load was reached at 79.8kN. Largest relaxation occurred at first crack load at around 31 kN. From plate 4.17, it was observed that the shear reinforcement did not snap even after the ultimate failure load was reached. From figure 4.34, it is clear that the load-deflection curve is non-linear throughout the loading. The deflection increased with increase in load to the level in which failure load was reached at 79.8 kN. The first crack widen increasingly with load increase to failure. The first crack occurred at bottom of the beam around 20 cm after the mid-span which progressed as it widened to the point of loading. At around 44 kN load a second crack occurred at 10 cm before the mid-span at the bottom of the beam going up to the point of loading at the top of the beam as shown it plate 4.18.

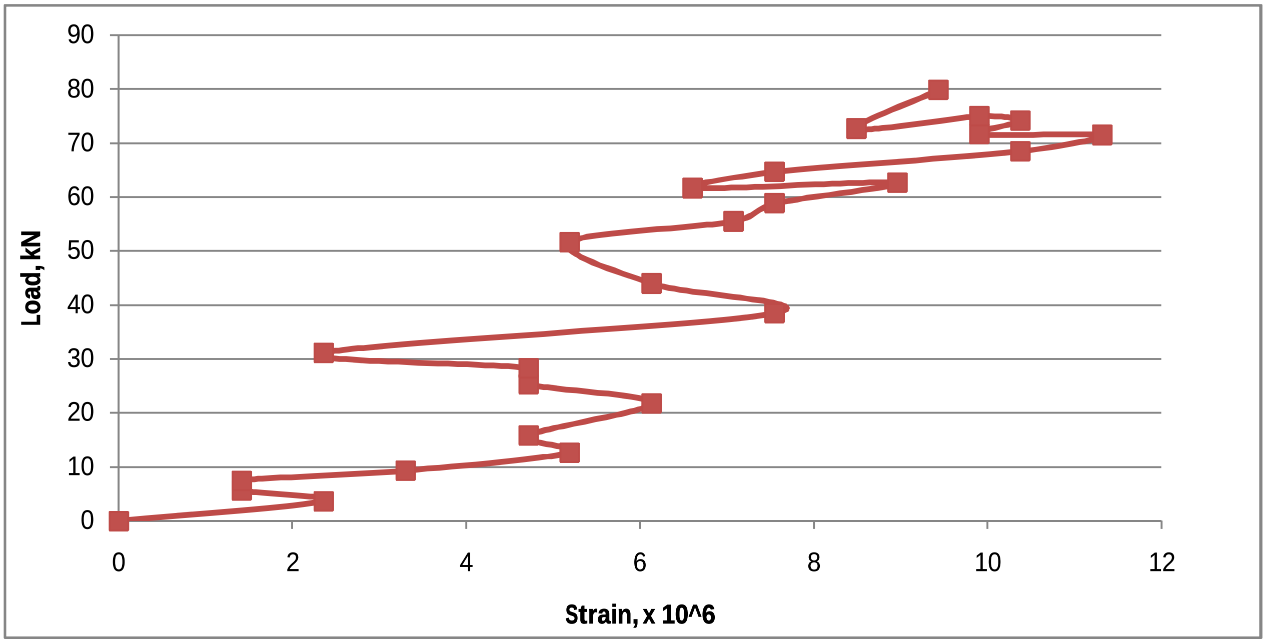

Figure 4.33: Reinforcement load-strain curve for L.B.2x10 beam

Table 4.20: Results from experimental loading for L.B.2x10 beam

L.B.2x10 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, $x10^{-6}$**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**
0	0	0	0	0
3.67	0.22	2.4	0.53	0.05
5.67	0.39	1.4	0.82	0.08
7.50	0.57	1.4	1.08	0.10
9.33	0.79	3.3	1.34	0.12
12.67	1.19	5.1	1.82	0.17
15.88	1.60	4.7	2.28	0.21
21.84	2.13	6.1	3.15	0.29
25.33	2.25	4.7	3.65	0.34
28.34	2.36	4.7	4.08	0.38
31.17	2.52	2.3	4.49	0.42
38.51	2.67	7.5	5.55	0.51
44.01	2.94	6.1	6.34	0.59
51.68	3.19	5.1	7.44	0.69
55.51	3.37	7.0	7.99	0.74
58.85	3.53	7.5	8.47	0.79
62.68	3.73	8.9	9.03	0.84
61.68	3.81	6.6	8.88	0.82
64.68	4.05	7.5	9.31	0.86
68.51	4.23	10.3	9.87	0.92
71.51	4.47	11.3	10.30	0.95
71.68	4.65	9.906	10.32	0.96
74.18	4.95	10.38	10.68	0.99
75.02	5.32	9.906	10.8	1.00
72.68	5.50	8.491	10.47	0.97
79.85	5.93	9.434	11.5	1.07

Plate 4.17: unbroken shear reinforcement for L.B.2x10 beam

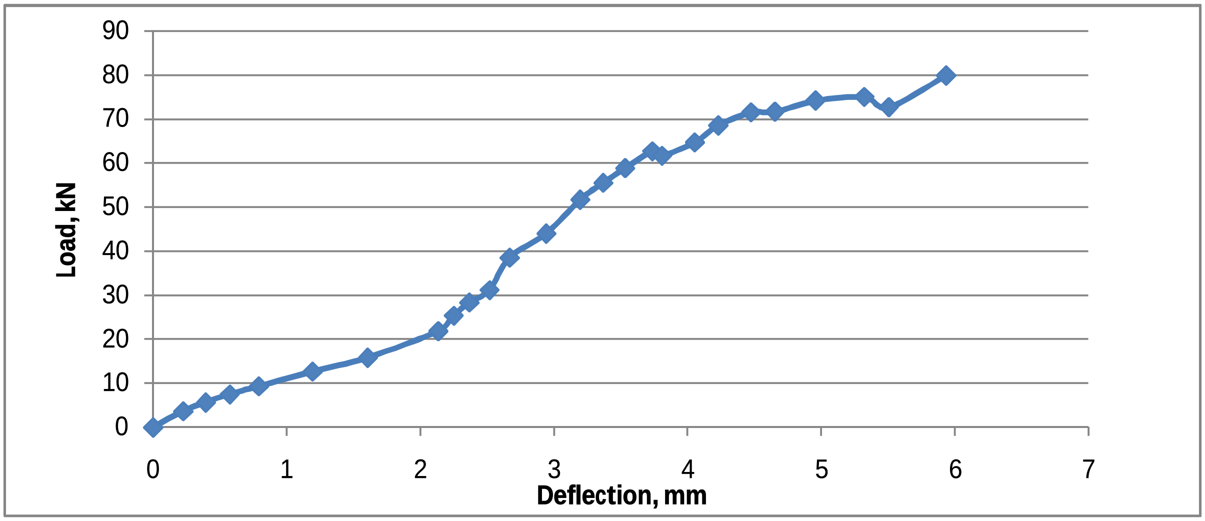

Figure 4.34: Load-deflection curve for L.B.2x10 beam.

Plate 4.18: Failure pattern of L.B.2x10 beam

4.4.4.4 L.B. 2x13 beam

The test result was carried out and it was observed that the first crack on concrete occurred at a load of 61.7 kN at a deflection of 1.98 mm and the ultimate load was 76.5 kN at deflection of 3.36 mm. The maximum bending stress achieved was 11.01 N/mm^2 while the maximum shear stress achieved was 1.02 N/mm^2. The test results based on this beam specimen were as presented in table 4.21.

In the study results as in figure 4.35, it can be seen that the load-strain curve was generally linear until when the first crack load was reached at about 61kN. The curve then became non-linear up to a point when the ultimate load was reached at 76 kN. As shown in plate 4.19, it can be observed that the shear reinforcement did not snap even after the ultimate failure load was reached. From the figure 4.36, it is clear that the load deflection curve was non-linear throughout the loading though deflection increased with increase in load. The first crack occurred at angle of around 45 degrees from the horizontal as shown in plate 4.20. However, the first crack expanded gradually up to the level where the failure load was reached.

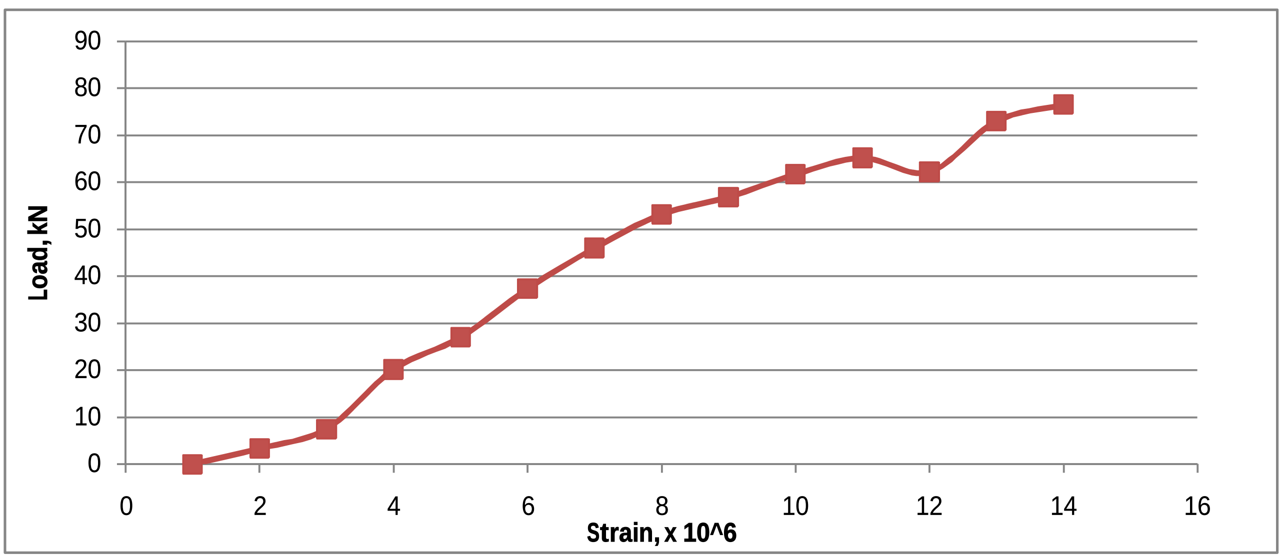

Figure 4.35: Reinforcement load-strain curve for L.B.2x13 beam

Table 4.21: Results from experimental loading for L.B.2x13 beam

L.B.2x13 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, x10^{-6}**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**
0	0	0	0	0
3.33	0.03	27.36	0.48	0.04
7.50	0.11	28.77	1.08	0.10
20.17	0.34	27.36	2.90	0.27
27.01	0.50	27.36	3.89	0.36
37.34	0.82	29.25	5.38	0.50
46.01	1.16	31.13	6.63	0.61
53.18	1.56	32.54	7.66	0.71
56.84	1.81	33.96	8.19	0.76
61.68	1.98	35.85	8.88	0.82
65.18	2.28	38.21	9.39	0.87
62.18	2.44	44.34	8.95	0.83
73.01	2.84	49.53	10.51	0.97
76.52	3.36	55.66	11.02	1.02

Plate 4.19: unbroken shear reinforcement for L.B.2x13 beam

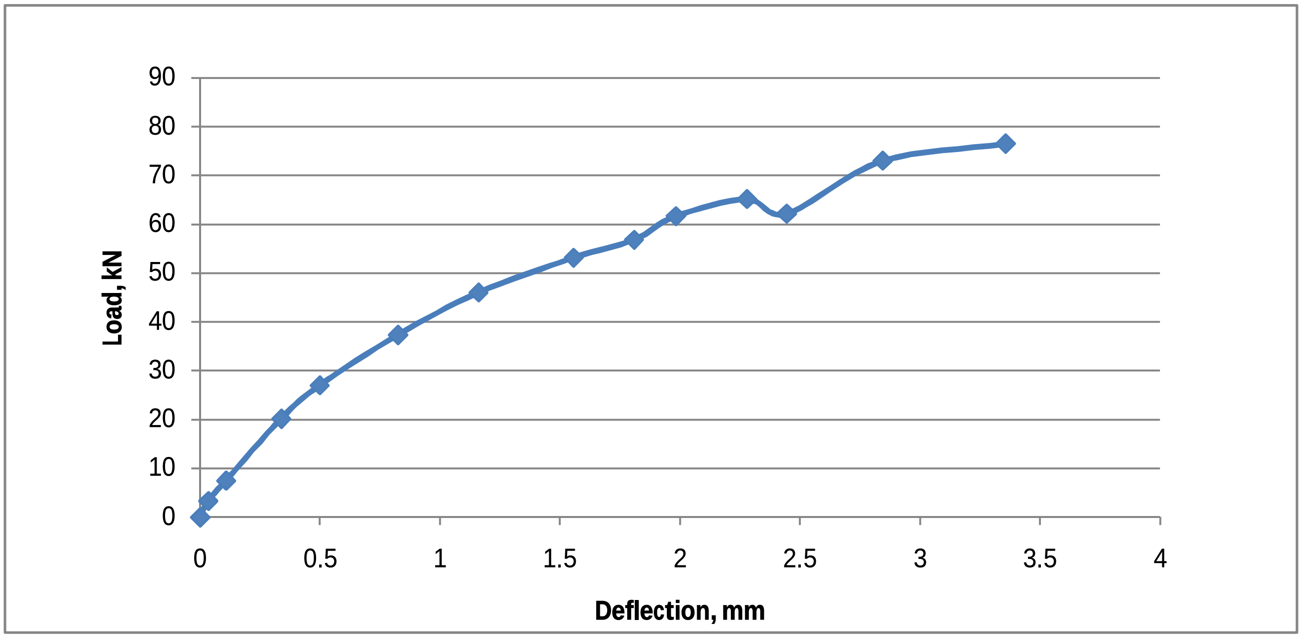

Figure 4.36: Load-deflection curve for L.B.2x13 beam.

Plate 4.20: Failure pattern of L.B.2x13 beam

4.4.4.5 Steel 1x6 beam

The test was conducted and the results obtained presented as shown in table 4.22. In this test, the study observed that the first crack on concrete happened at a load weight value of 32.2K kN at about 0.9 mm point of deflection. In addition, it can be seen that the ultimate load was 81.25 kN at around 8.11 mm deflection point. The maximum bending stress attained on this case was 11.7 N/mm^2 while the maximum shear stress achieved was 1.08 N/mm^2.

It can be seen that the load-strain curve was in linear in nature from the starting point to the point when the load reached around 62 kN (see figure 4.37). From then on the strain on the reinforcement increased up to when the ultimate load was reached. From the findings in plate 4.21, it can be seen that the steel shear reinforcement failed by snapping. Conversely, in figure 4.38, it is clear that the load-deflection curve was non-linear throughout the loading. The deflection increased with increase in load till when failure load was reached at 81.25 kN. The first crack widen as well until when the failure load was reached. The first crack was observed to have occurred at bottom around 10cm from the mid-span which progressed as it widened to the point of loading as shown in plate 4.22.

Table 4.22: Results from experimental loading for Steel 1x6 beam.

Steel 1x6 beam				
Load, kN	**Deflection, mm**	**Reinforcement strain, x10^{-6}**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**
0	0	0	0	0
2.62	0.01	0.47	0.38	0.09
6.38	0.19	0.94	0.92	0.09
17.00	0.40	2.83	2.45	0.23
32.25	0.92	4.71	4.64	0.43
40.5	1.05	6.13	5.83	0.54
55.00	1.73	8.96	7.92	0.73
62.63	2.12	10.85	9.02	0.84
72.13	3.69	14.15	10.39	0.96
75.75	4.64	15.09	10.91	1.01
75.75	5.20	16.04	10.91	1.01
78.63	6.08	16.51	11.32	1.05
78.13	6.69	16.98	11.25	1.04
78.63	7.24	17.45	11.32	1.05
81.25	8.11	17.45	11.7	1.08

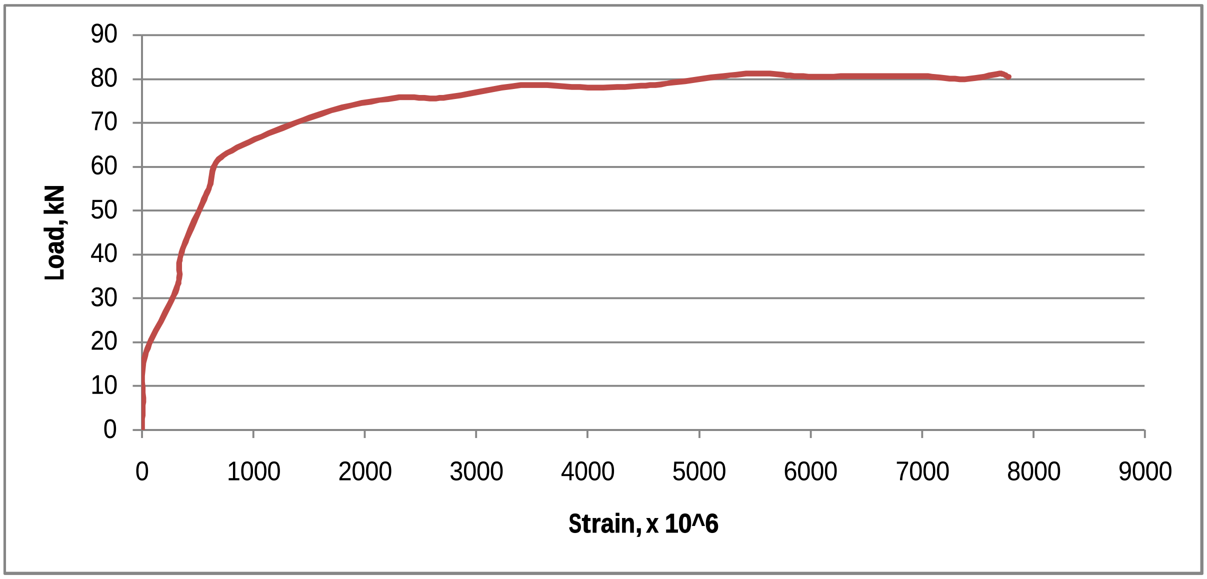

Figure 4.37: Reinforcement load-strain curve for Steel 1x6 beam

Plate 4.21: unbroken shear reinforcement for Steel 1x6 beam

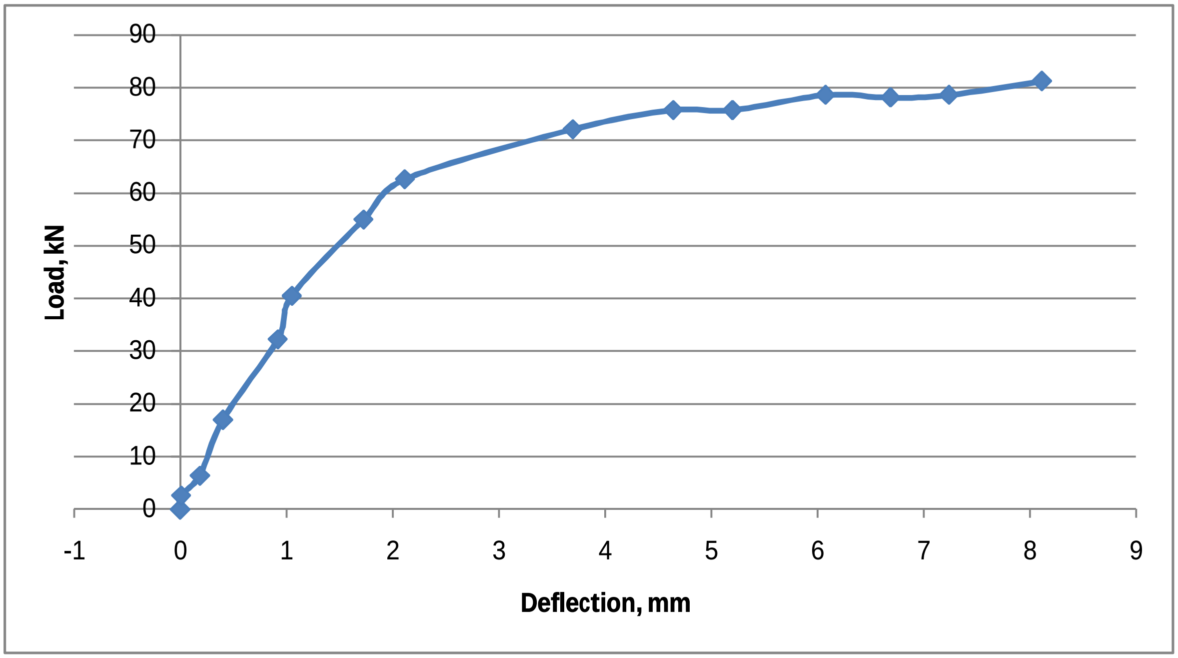

Figure 4.38: Load-deflection curve for Steel 1x6 beam.

Plate 4.22: Failure pattern of Steel1x6 beam

4.4.5 Comparison with of concrete reinforced beams

4.4.5.1 Tensile *Landolphia buchananii* and steel reinforced concrete beams

Figure 4.39 shows the load-deflection curves for steel and *Landolphia buchananii* reinforced concrete. The *Landolphia buchananii* curves s exhibit linearity for loads up to around 8kN or the first cracking load. At 8kN the slope of the curves changed and maintained a non-linear relationship up to failure. The steel reinforced concrete beams curves was linear till load o 42 kN was reached. The steel reinforced concrete curve exhibited the post elastic behavior but *Landolphia buchannii* reinforced concrete beams did not.

From the test results in table 4.23, the *Landolphia buchananii* reinforced concrete beams failed between 13 kN and 19.2 kN while the steel reinforced beam failed at 56.25 kN. Failure load coincide with the last point of the curves. The yield deflection for L.B. 805, L.B. 787, L.B. 741 and steel 101 was 9.4, 12, 16.2 and 19.2 mm while their first crack loads deflection was 1.74, 0.9, 0.28 and 19.2mm respectively. The failure loads for *Landolphia buchananii* reinforced concrete beams increased with increase in the area of reinforcement of *Landolphia buchananii* while their deflection at these failure loads decrease with increase of area of reinforcements.

It can be seen from the study findings that the bending stress of *Landolphia buchananii* reinforced concrete beam increased with increased in area of reinforcement of *Landolphia buchananii*. It can be seen that the bending stress of the *Landolphia buchananii* beam with the largest area of reinforcement (L.B. 805) was around 35% that of steel reinforced beam (Steel 101) as shown in figure 4.40

In accordance to BS 8110 beams are considered to have failed when deflection exceeds the ration span/360. Actual span of beams tested was 900 mm which by this code of practice complied at first crack which was 2.5mm.

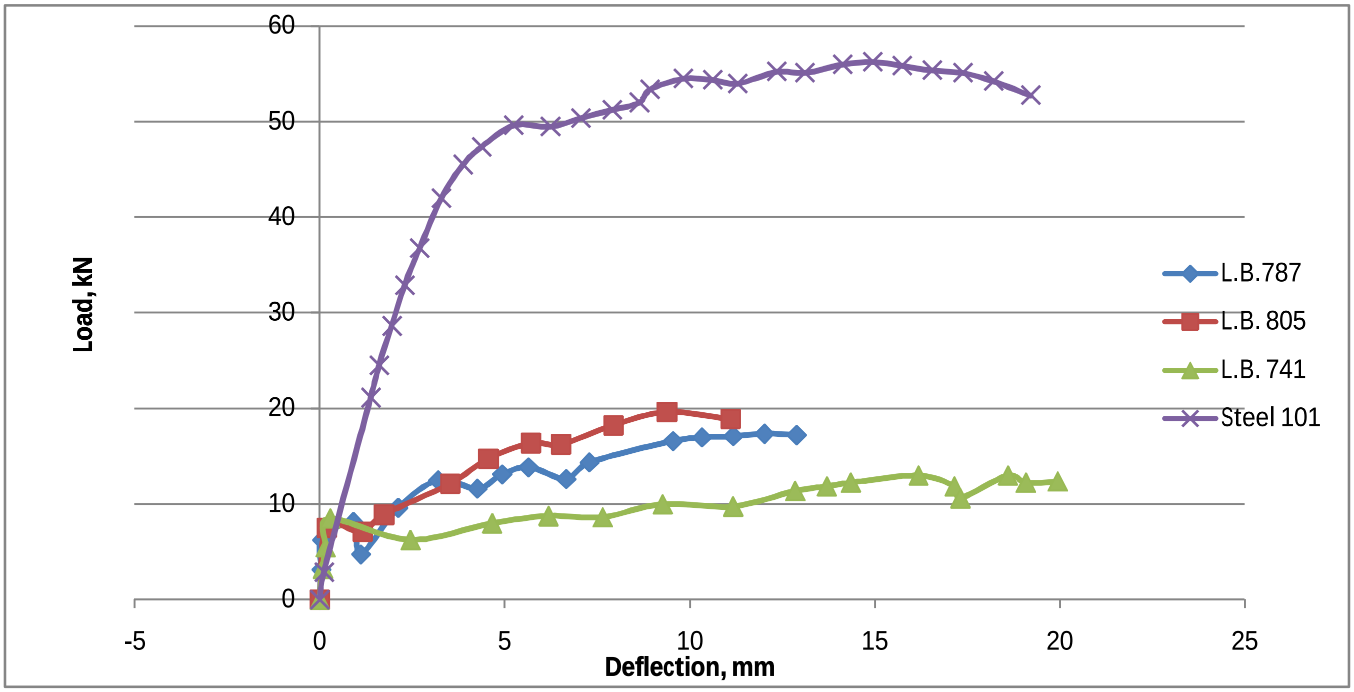

Figure 4.39: Load-deflection curves for tensile reinforced beams.

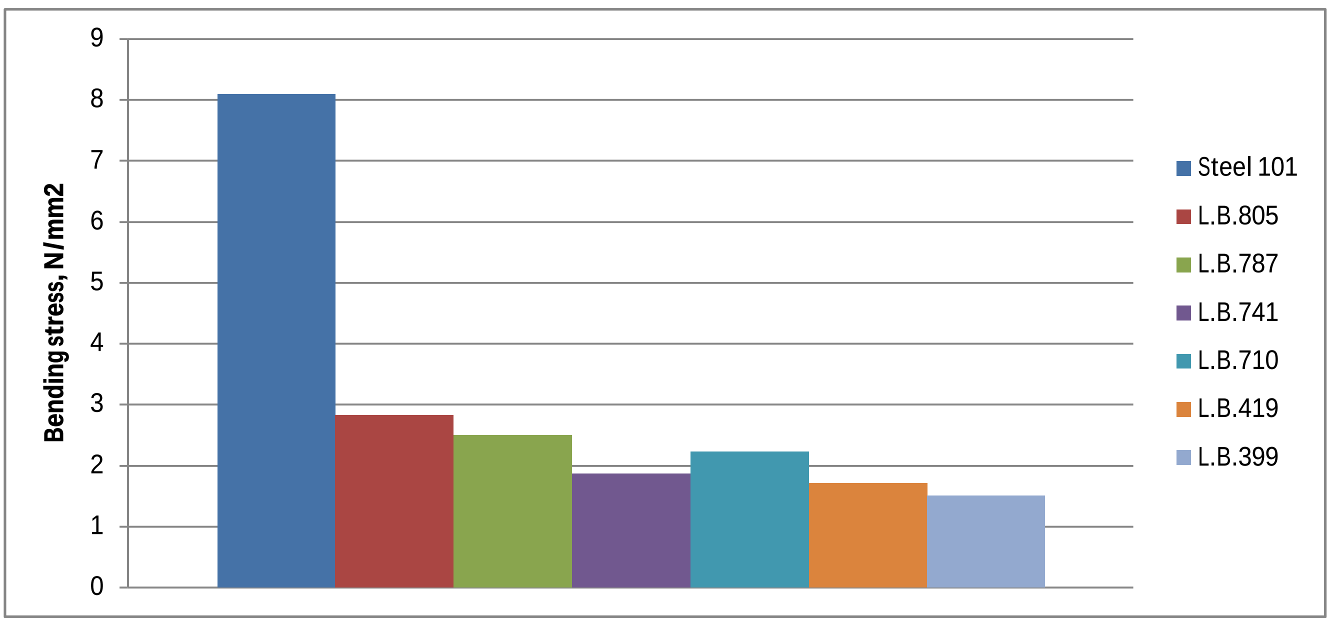

Figure 4:40: Comparison of bending strengths of tensile reinforced concrete beams

4.4.5.2 Shear *Landolphia buchananii* and steel reinforced concrete beams

The findings in figure 4.41 shows the load-deflection curves for steel and *Landolphia buchananii* reinforced concrete. All the curves generally exhibit non-linearity for all the loads to failure.

From table The *Landolphia buchananii* reinforced concrete beams failed between 58 kN and 79.8 kN. Steel reinforced beam failed at 81.3 kN. Failure load coincide with the last point of the curves. The yield deflection for L.B. 1x6mm, L.B. 1x8, L.B. 2x10, L.B. 2x13 and steel 1x6 was 2.9, 0.69, 5.9, 3.36 and 8.1 mm while their first crack loads deflection was 1.35, 0.49, 0.25, 1.98 and 0.9mm respectively. The failure loads for *Landolphia buchananii* reinforced concrete beams increased with increase in the area of shear reinforcement of *Landolphia buchananii*.

From table 4.24, it can be seen that the bending stress and shear stress of *Landolphia buchananii* reinforced concrete beam increased with increased in area of shear reinforcement of *Landolphia buchananii*. It can be seen that the bending stress and shear stress of the *Landolphia buchananii* beam with the largest area of shear reinforcement (L.B. 2x13) was around 94 % that of steel reinforced beam (Steel 1x6) as shown in figure 4.42 and 4.43.

In accordance to BS 8110 beams are considered to have failed when deflection exceeds the ration span/360. Actual span of beams tested was 900 mm which by this code of practice complied at first crack which was 2.5 mm.

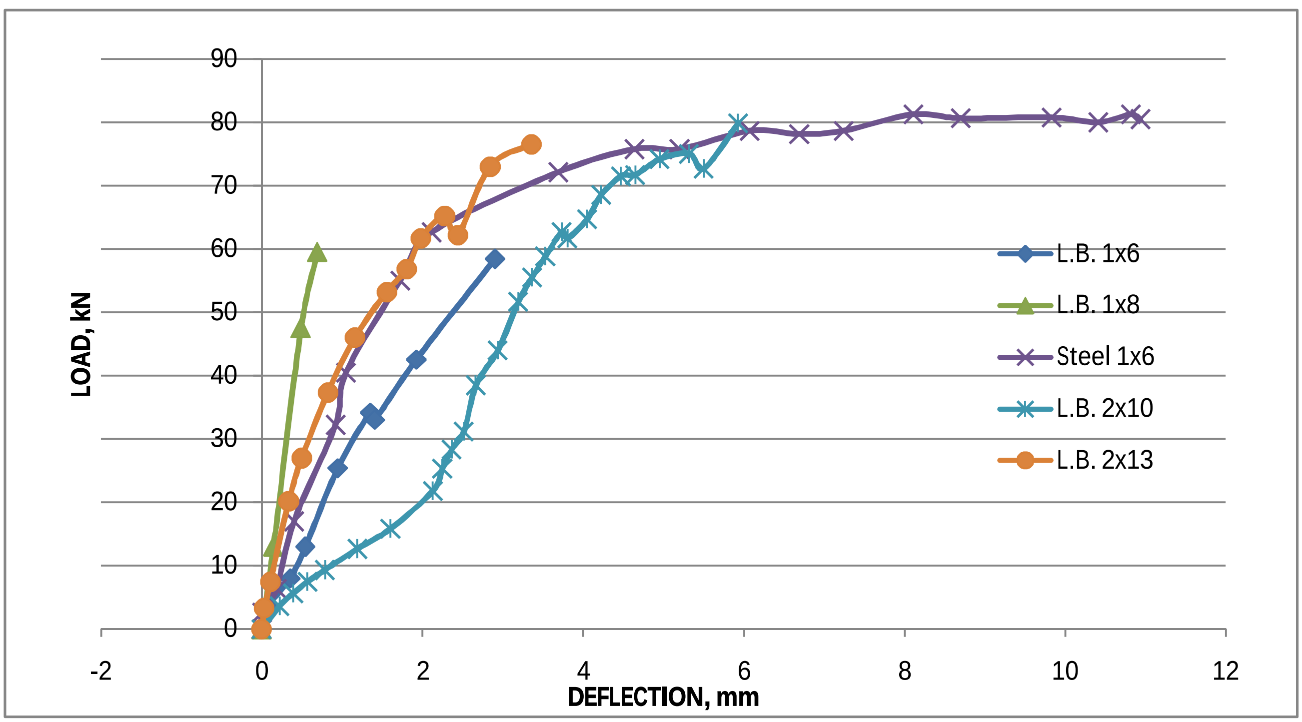

Figure 4.41: Load-deflection curves for shear reinforced beams.

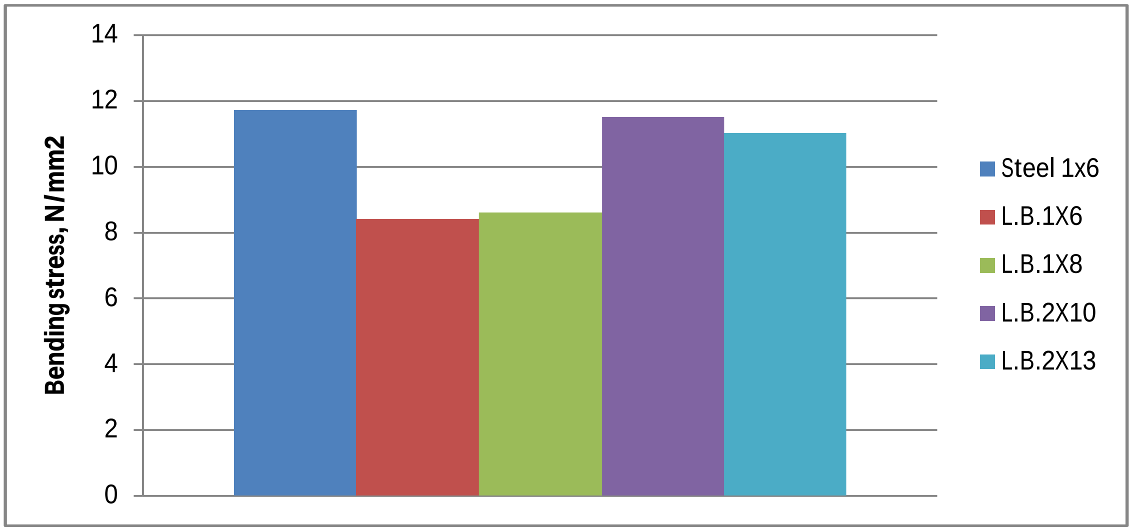

Figure 4.42: Comparison of bending strengths of shear reinforced concrete beams

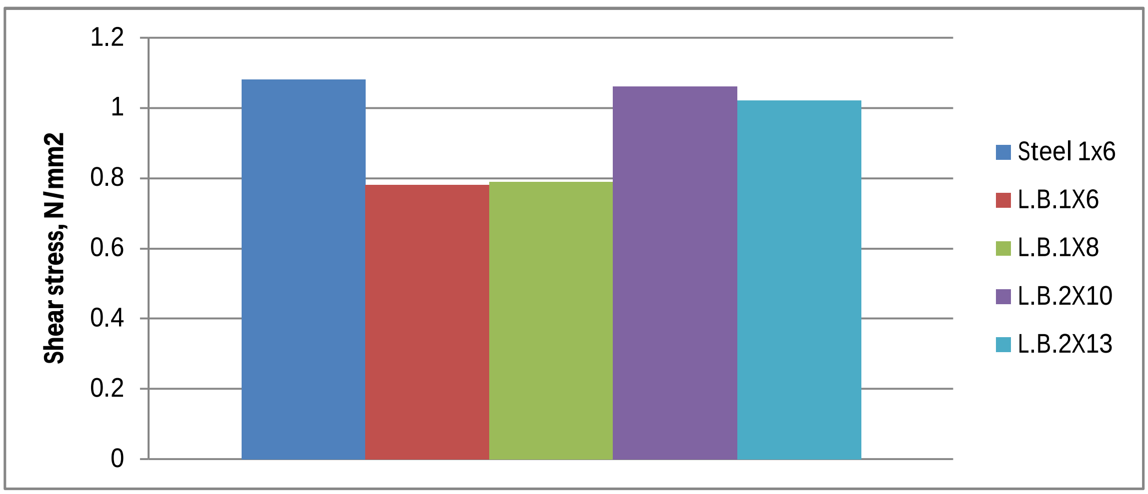

Figure 4.43: Comparison of bending strengths of shear reinforced concrete beams

4.4.6 Summary of reinforced concrete beam results

4.4.6.1 Landolphia buchananii as a tensile reinforcement beams

The summary of the results are presented in table 4.23. The results presented are first crack loads, ultimate loads, first crack load deflections, second crack load deflections and bending stresses of the reinforced concrete beams at concrete age of 28 days. From table 4.23, it can be seen that the load carrying capacity of the beam depended on the area of reinforcement: the larger the area of reinforcement the larger the load carrying capacity.

Flexural failure is noted in all the beams with crack gradually starting at mid span and enlarges as the loading was increased. From the loading deflection curves (figure 4.39), failure pattern *L. buchananii* reinforced beams (plate 4.6, 4.7, 4.8, 4.9, 4.10 and 4.11) and failure in the reinforcements (Plate 4.24 and 4.25) as compared to that of the steel reinforced beam (plate 4.12 and 4.23). It can be noted that bonding failure could be the probable cause of the premature flexural failure in the *L. buchananii* reinforced beams. It can be seen that the tensile reinforcement was still intact but had slipped through the concrete while steel tensile reinforcement snapped as shown in figure 4.23. When the region of failure crack was examined in figure 4.26 the *L. buchananii* stem seem unbroken, suggesting that there was a poor bond between *L. buchananii* and concrete which led to beam failing in bonding.

Plate 4.23: Snapped steel tensile reinforcement

Plate 4.24: Slipped *L. buchananii* tensile reinforcement

Plate 4.25: unsnapped *L. buchananii* tensile reinforcement

Table 4.23: Summary beam test results for *L. buchananii* as a tensile reinforcement

***L. buchananii* as tensile reinforcement**								
Beams	**Deflection, mm**			**Load, kN**				
	1st crack	**2nd crack**	**Max.**	**1st crack**	**2nd crack**	**Max.**	**Bending stress, N/mm²**	**Mode of failure.**
Steel 101	14.94	-	14.94	56.25	-	56.25	8.1	flexure
L.B.805	0.19	5.71	9.38	7.5	10.14	19.6	2.83	flexure
L.B.787	0.9	-	12	8.15	-	17.38	2.5	flexure
L.B.741	0.28	9.2	16.18	8.5	10	13	1.87	flexure
L.B.710	-	-	-	7.4	-	15.5	2.23	flexure
L.B.419	-	-	-	11.9		11.9	1.71	flexure
L.B.399	-	-	-	11		11	1.5	flexure

4.4.6.2 *Landolphia buchananii* as a shear reinforcement beams

The summary of the results are presented in table 4.24. The results presented are first crack loads, ultimate loads, first crack load deflections, second crack load deflections, shear stresses and bending stresses of the reinforced concrete beams at the age of 28 days. From table 4.24, it can be seen that the load carrying capacity of the beam depended on the area of shear reinforcement: the larger the area of shear reinforcement the larger the load carrying capacity..

Shear failure is noted in all the *Landolphia buchananii* reinforced concrete beam with crack gradually occurred from the bottom side of the beam at or near the support diagonally to or near the point of loading. From figure 4.41 the loading deflection curves and the failure pattern *L.buchananii* reinforced beams (plate 4.14, 4.16, 4.18, and 4.20) as compared to that of the steel reinforced beam (plate 4.22). It can be noted that the *L. buchananii* shear reinforcement did not snapped as compared to the steel though all the beams failed shear. When the region of failure crack was examined (plate 4.13, 4.15, 4.17 and 4.19) the *L.buchananii* stem seem unbroken, this can suggest that there was a poor bond between *L.buchananii* and concrete which led to beam failing in bonding or that the reinforcement was flexible enough to not to break. The flexibility of *Landolphia buchananii* shear reinforcements can be attributed to low modulus of elasticity.

Table 4.24: Summary beam test results for *L. buchananii* as a shear reinforcement.

***L. buchananii* as shear reinforcement**							
Beams	**Deflection, mm**		**Load, kN**				
	1st crack	**Max.**	**1st crack**	**Max.**	**Bending stress, N/mm^2**	**Shear stress, N/mm^2**	**Mode of failure**
Steel 1x6	0.9	8.11	32.2	81.3	11.7	1.08	Shear +flexure
L.B.1x6	1.35	2.9	34.1	58.4	8.4	0.78	Shear
L.B.1x8	0.15	0.69	47.5	59.5	8.6	0.79	Shear
L.B.2x10	2.94	3.9	31.2	79.8	11.5	1.06	Shear
L.B.2x13	1.98	3.36	61.3	76.5	11.0	1.02	Shear

CHAPTER 6: CONCLUSION AND RECCOMENDATION

6.1 Conclusion

From the analysis and discussion of the results, it can be concluded that the use of *Landolphia buchananii* as longitudinal and shear reinforcement in concrete elements is feasible. The strength tests of *Landolphia buchananii* obtained in the present work shows *Landolphia buchananii* to be fairly strong material that can be used as a substitute for structural steel reinforcement with reasonable service load. The modulus of elasticity of *Landolphia buchananii* was around 1.7 % that of steel while the tensile strength of *Landolphia buchananii* is around 19% that of steel. *Landolphia buchananii* that is to be used in the reinforcement of concrete elements should be dry or seasoned because the dry *Landolphia buchananii* have higher strengths than the green one by 50% to 350%. *Landolphia buchananii* can be used as shear reinforcement in the beams especially where shear strength is low. The tensile strength of dry *Landolphia buchananii* has been observed in the present work to be substantially high, ranging between 65 N/mm^2 and 112.6 N/mm^2. From reinforced beams deflections, possible influence of sustained loading and the mechanical strengths of *Landolphia buchananii* it can be concluded that *Landolphia buchananii* can be utilized as a reinforcement of beams under low loading regimes such lintels with load bearing walls.

6.2 Recommendation

This study showed that *Landolphia buchananii* can be used as tensile and shear reinforcement of concrete beams. However, the following recommendation should be considered when using it:

i. Characterization of mechanical properties of *Landolphia buchananii* needs to be done due to the large variability of properties observed.
ii. Light weight concrete should be used in order to have lower loading regime for the *L. buchananii* reinforced concrete beams.
iii. *Landolphia buchananii* stems should be lightly filed in order to profile the surface and short length of wire should be wound round it at reasonable interval. This will improve its

interfacial bond strength in concrete which in turn will improve the load carrying capacity of the beams.

6.3 Areas of further research

Although this research study was confined to what was studied, there are more areas to research on. The following are areas of further research:

i. Since the strength properties of *Landolphia buchananii* vary from one specimen to another, it should be investigated and graded for strength purposes.
ii. Long-term loading carrying capacity of the *Landolphia buchananii* reinforced concrete elements should be investigated because concrete elements are always designed for a specific load regime and time.
iii. Performance of *Landolphia buchananii* reinforced concrete beams should be studied with respect to fire to establish their behavior in fire or at elevated temperatures.
iv. The structural dynamic performance of *Landolphia buchananii* reinforced concrete beam should be investigated to establish their viability in high wind and earthquake zones.
v. The durability of *Landolphia buchananii* in concrete should be investigated in order to establish the lifespan of concrete beams reinforced with it.
vi. An extensive study should be conducted to evaluate the behavior of other species of *Landolphia*. The literature shows that there is a significant variation of material properties within the same species of a plant, hence there is need to conduct an extensive study on *Landolphia* species in order to establish the species with best desirable material properties.

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APPENDICES

Aggregate sieve analysis test results

BS sieve size	Percentage passing, %	Percentage retained, %	Cumulative retained, %
Course aggregate			
31.7 mm	100	0	0
20 mm	100	0	0
14 mm	51.6	48.4	48.4
10 mm	23.6	28	76.4
5 mm	6.5	17.1	93.5
Pan	0	6.5	
Fine aggregate			
5 mm	100		
2.36 mm	99.2	0.8	0.8
1.18 mm	82.1	17.1	17.9
600 µm	48.2	34	51.9
300 µm	11.9	36.3	88.2
150 µm	2.5	9.3	97.5
Pan		2.5	