THE USE OF ENCASED TIMBER AS ALTERNATIVE *TO* REINFORCED CONCRETE COLUMNS

BY

ENGR. VINCENT 0. U. ADEYEMI

B.ENG. CIVIL (A.B.U) MNSE

A THESIS SUBMITTED TO TEE POSTGRADUATE

SCHOOL, AHMADU BELLO UNIVERSITY, ZARIA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF MASTER OF SCIENCE DEGREE

IN STRUCTURAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

AHMADU BELLO UNIVERSITY

ZARIA, NIGERIA

SEPTEMBER, 1990

DECLARATION

I hereby declare that this dissertation has been prepared by myself and it is a record of my own research work. It has not been accepted in any previous application for a higher degree. All sources and information are specifically acknowledged by means of references.

13/09/90

VINCENT O. U. ADEYEMI

DEDICATION

This work is dedicated to my beloved wife, Mrs. Bola Adeyemi and the pursuit of excellence in structural engineering practice.

CERTIFICATION

This thesis entitled "The Use of Cased Timber as Alternative to Reinforced Concrete Columns" by Vincent O. U. Adeyemi meets the regulations governing the award of the Degree of Master of Science (Structural Engineering))of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

Mr. S. S. LETCHA
Chairman, Supervisory Committee

Date: 14/1/91

Prof. A. O. ABATAN
Member, Supervisory Committee

Date: 5/2/91

Prof. A. O. Abatan
Head of Department

Date: 5/2/91

Dean, Postgraduate School
Ahmadu Bello University
Zaria

Date: 12/4/91

ACKNOWLEDGEMENT

My sincere gratitude goes to my Supervisors, Mr. S. S. Letcha and Prof. A. O. Abatan for their tiredless efforts in making sure that this thesis is completed and to the standard required. I actually consider myself privileged to have worked with Mr. S. S. Letcha and Prof. A. O. Abatan.

I am also grateful to Prof. O. Adebisi for his great efforts in encouraging me to come for this postgraduate course. I also thank the concrete laboratory staff members for their assistance. My special thanks go to Mr. Bello, Alhaji Shonibare , Mallam Abdullahi and others.

I am thankful to my wife for her endurance and patience coupled with her moral and financial supports given to me during the preparation of this thesis.

I wish to express my sincere appreciation to Mr. Sam. Imande for his contribution to the success of this thesis.

Finally, I am grateful to God and the Lord Jesus Christ for granting me the opportunity to run this course. I am also grateful to the Civil Engineering Department non-academic staff members for their precious co-operations at all time.

13/09/90
VINCENT O. U. ADEYEMI

ABSTRACT

The cost of steel rods and cement has become so high that it is becoming increasingly difficult for an average Nigerian to build reinforced concrete houses. Even though coarse aggregates could be got locally, the cost of production is also high. Thus, fewer houses are being erected nowadays. It will not augur well for the country if this trend is allowed to continue since the population of Nigeria is increasing daily. Therefore, there is a need to look for alternative building materials. Timber is one of such materials that can be used. Timbers that are good enough for structural elements are readily available in many parts of Nigeria and relatively cheaper when compared with the cost of steel rods. The fear of the prospective users of timber in this fashion is the fire hazard.

This work therefore investigated the use of timber encased with concrete as an alternative to the conventional steel renforced concrete columns in buildings, especially in low cost housing schemes. The objective is to cut down the cost of columns in buildings, since timbers are available and cheaper. A lot of saving will also be made in the amount of concrete used.

The results indicate that encased timber columns have adequate load bearing capacity for use in conventional residential buildings. It has been found from test results also that the bond strength between concrete and timber is adequate (averagely $1.95N/mm^2$) for achieving composite action in the encased timber column. The failure loads of the columns were also computed theoretically and. the results agreed favourably with the test results.

TABLE OF CONTENTS

PAGE

LIST OF TABLES

LIST OF FIGURES

LIST OF PLATES

NOTATIONS

ρ_t	:	Density of Timber
m	:	Moisture Content of Timber
σ_p	:	Compressive stress parallel to the grain of the Timber
E_t	:	Modulus of Elasticity of Timber
ν_t	:	Poison's ratio for Timber
fst	:	Bond Strength betwee Concrete and Timber
σ_c	:	Strength of concrete from cube Test
M_t	:	Bending Moment carried by Timber
I_t	:	Second moment of area of Timber Section
R	:	Radius of Curvature
Mc	:	Bending Moment carried by Concrete
I_c	:	Second moment of area of Concrete Section
E_c	:	Modulus of Elasticity of Concrete
M_s	:	Bending Moment Carried by Steel
I_s	:	Second Moment of area of the Steel Section
E_s	:	Modulus of Elasticity of Steel
I_e	:	Equivalent second moment of area
σ_{cs}	:	Bending stress in a fibre of Concrete
ε	:	Strain
y	:	Distance from the axis to the fibre under consideration
σ_{ss}	:	Bending stress in a fibre of Steel
σ_{ts}	:	Bending stress in a fibre of Timber
Ac'	:	Equivalent Concrete area for Timber
Ac"	:	Equivalent Concrete area for Steel
ε_t	:	Strain in Timber
ε_c	:	Strain in Concrete
ε_s	:	Strain in Steel

A_t	:	Cross-sectional area of Timber
Ac	:	Cross-sectional area of Concrete
A_s	:	Cross-sectional area of Steel
L	:	Length of Column
P	:	Thrust on Column
V	:	deflection of Column
M	:	bending moment at any section of column
M_t	:	bending moment at any section of timber column
M_c	:	bending moment at any section of concrete column
K_s	:	Ratio of thrust to bending stiffness
P_e	:	Critical load on column
σ_e	:	Critical stress
A	:	Cross-sectional area of column
r	:	Radius of gyration
E	:	Young's Modulus
$\bar{x}$	:	Mean of results
σ	:	Standard deviation
V_f	:	Coefficient of variation

CHAPTER ONE

INTRODUCTION

1.1 PREAMBLE AND PROBLEM STATEMENT

The use of reinforced concrete material in the building industry has been in existence for some years now. A lot of researches which aimed at the best use of steel reinforced concrete had been carried out. Reinforced concrete is an artificial substance formed by mixing cement, water and aggregate together with an insertion of steel rods in such a fashion that when the mixture is set, it forms a stone-like material. In Nigeria today, the cost of production of reinforced concrete has gone out of the reach of the masses. Steel rod used as reinforcement is so expensive since part of materials used for steel production is imported, it is the same with cement. Even though aggregates are got locally, the production cost is so high that less use of reinforced concrete is becoming inevitable. Therefore, there is the need to look for alternatives. One of the local material that can be got very readily and averagely cheaper in Nigeria is Timber. There are in existence buildings constructed mainly of timbers. In recent time, works on timber - concrete composite structures have been carried out, especially in construction of bridge elements such as decks, T-beams and girders.

The idea of using timber-concrete composite columns in buildings and especially in low-cost housing is particularly welcome, because the fear of clients using timber in housing, is eliminated since timber is now encased, hence preventing it from fire hazards.

1.2 THE OBJECTIVE OF THIS RESEARCH

The objective of this research is to investigate the possibility of using a concrete cased timber as alternative to the conventional steel reinforced concrete columns in buildings. Recalling from the introduction, works have been carried out on Concrete-Timber Composite elements such as decks, T-beams and girders. In the first instance, the use of timber in this way will greatly reduce the overall constructional cost of buildings when compared with the conventional steel reinforced concrete buildings. This can be said confidently when one considers the availability of timbers good enough for structural elements and their cost in Nigeria on one hand, and the high cost of the manufactured steel and cement on the other hand. Less mass of concrete will be used in the case of concrete cased timber column.

THE SCOPE OF WORK

1.3 Tests on small, clear specimens will be performed on the timber itself. The preparation of the specimens and the testing procedures shall follow the British Standard Specifications.

The full description and the testing procedures shall be presented in chapter three of this thesis. The following tests will be carried out:

(a) Test to determine the density of the timber.

(b) Test to determine the moisture content of the timber at use.

(c) Compression test parallel to the grain.

(d) Test to determine the modulus of Elasticity of the Timber Using Ultrasonic Pulse Velocity (PUNDIT).

(e) Test to determine the bonding strength between the timber and the concrete.

(f) Concrete cubes for every mix will be prepared and tested to determine the concrete strength.

The investigation to be carried out in this research shall consist of an experimental and a theoretical phase. The experimental phase shall be concerned with the test to find the load bearing capacity of the concrete cased timber columns, loaded axially. The column specimens shall have different slenderness ratios. The mode of failure shall be observed. The theoretical phase of the study shall consist of the prediction of the ultimate load-bearing capacity of each of the specimens by analysis based on method of transformed sections.

Twenty-five numbers of concrete cased timber column specimens shall be prepared. A steel fabric reinforcement with steel average size of 2.95mm both ways and pitch of 50mm x 50mm will be placed in the covering concrete of these specimens to check any cracking that may occur.

The specimens shall have the same cross-sectional area of timber and the same concrete cover thickness. The specimens will comprise of five sets. The sets will be different from one another by varying the slenderness ratio from 16 to 5.3. Another six specimens similar to the ones above but without steel fabric will be prepared to see if the specimens will not crack without the steel reinforcement since the timber will be anticipated to absorb water and swell when the concrete is wet and shrink when the concrete is dry.

CHAPTER TWO

LITERATURE REVIEW

2.1.0 TIMBER

The use of Timber as a compression member (columns, posts etc.) has been in existence for many years. It has been known that timber can resist compressive loads appreciably and that when properly preserved and seasoned, its loading capacity can be high and very durable too. Generally, timber is both a trustworthy traditional building material and one which is suitable for the application of advanced engineering and production techniques aimed at greater economy in the field of industrialised house-building. In Nigeria, timber is a building material that can be got readily in many parts of the country. It is disheartening that fire hazard has posed a serious problem to the common use of timber. Although, there are fire preservative chemicals, they are too costly for an average income earner in Nigeria.

2.2.0 SOME PROPERTIES THAT INFLUENCE THE STRENGTH OF TIMBER

2.2.1 Direction of the Grain

The direction of the grain has some influence on the strength of timber. When the strength of timber is the primary consideration, it is usual to specify that it shall be straight grained. It has been found out that there is a reduction of about 4% in bending strength when the slope of the grain is 1:25. With slope of 1:20, the reduction is 7%, according to Desch(1977).

2.2.2 Moisture Content

Moisture in the timber affects greatly the strength of the timber. The timber of living trees and freshly felled logs contains a large amount of water, which usually constitute a greater proportion by weight than the solid material itself. Water has a profound influence on the properties of timbers. It affects the weight, the strength, the shrinkage and the liability to attacks by fungi or insects of timbers. Therefore, it is important to know the moisture content of a timber before use. When the moisture content of a timber is about 12%, average increase or decrease in value of various strength properties like bending strength, tensile strength, compressive strength and so on, is affected by decreasing or increasing moisture content by 1%, Wangaard (1950) and Desch (1977). Shrinkage or swelling of timber due to changes in the moisture content is called "working" or movement.

It can not be eliminated completely but can be minimized. Timber hardly shrinks at all along the grain. In the radial and tangential directions, however, movement is appreciable. Tangential shrinkage is always more than the radial shrinkage. Even in different pieces of the so-called air-dry-seasoned timbers, some variations do occur in moisture content. The moisture content at a given time depends on the atmospheric conditions. Drying does not continue indefinitely. There is normally a stage when there will be no more interchange of moisture between the timber and the air. This is the stage of equilibrium which can be up set easily when there is a change in the surrounding temperature.

2.2.3 Density

Density is the best single criterion that can be used when considering the strength of a piece of timber. Density is the ratio of mass of timber to its volume. A thoroughly dry timber consists of solid material of the cell walls. The cell cavities contain air and small quantities of gum. The relative density (specific gravity) of the solid material of the walls has been found to be similar in all timbers (around 1.5). Timbers generally weigh between 160 to 1250 kilogrammes per cubic metre. One very interesting thing is that, besides the variation in density occuring in timbers of different species, there is an appreciable variation in density between different samples of the same species, which contain the same amount of water, expressed as a percentage of the dry weight of timber in the sample. This variation can also occur between the timber of different trees, and in timber from different parts of one tree.

2.2.4 Defects

The presence of defects in timber affects its strength. The relative abundance of the different kinds of tissue and the arrangement of the individual elements in relation to one another bring differences in the mechanical properties of the timbers. Defects in timber could be irregularities of grain, splits and checks that develop during seasining, the presence of rot and the abnormalities in anatomical structure. The weakening effect of defects in timber depends on the position of such a defect in relation to the piece of the timber as a whole and the proposed use of the timber. 'Spongy heart' even though it can reduce the strength of timbers, if its presence is not much, it may have no significant effect at all on timbers used as short columns and light effect on beams. All forms of warpping reduce the strength of timbers. Initial bending of timbers used as long columns say 1:1000 may reduce the strength of the timber by about 20%. Splits and checks which are due to ruptures of the tissues reduce the shear resistance of timbers.

2.3.0 SEASONING OF TIMBER

Seasoned timber is superior for all practical purposes than unseasoned timber. A seasoned timber is always dry and has less or no movement. If the moisture content of a timber is more than 20%, most fungi can grow in it and destroy the timber. Therefore, seasoning arrests the development of incipient of sound timber. Seasoned timber has structural advantage of less weight. The cost of transportation is also reduced. The major difficulty in seasoning is to prevent the outer layers to dry out too rapidly than the interior. If this is not prevented, stresses will be developed and can cause surface splits or checks. There are two common methods of seasoning, namely: air seasoning (natural seasoning) and kiln seasoning (artificial seasoning).

2.3.1 Air Seasoning

Air seasoning makes use of the prevailing winds and the sun, while protecting the timbers from rain. Winds circulate the air, hence disallow the air from being saturated with moisture absorbed from the timber, while the sun raises the air temperature and lowers its relative humidity. Control of air circulation is always necessary and this may best be achieved by pilling the timber in properly constructed stacks. Some form of end covering should be adopted to prevent rapid loss of moisture at the ends which can cause end splits.

2.3.2 Kiln Seasoning

Kiln drying is effected in a closed chamber, providing maximum control of air circulation, humidity, and temperature. This makes regulation of drying possible so that shrinkage if occurs at all will be minimum. Lower moisture contents can be reached than are possible with air seasoning. Kiln seasoning has more advantages because it is rapid, adaptable easily and more precise. It can be used at any time of the year. A timber seasoned by kiln has the likelihood of having a constant moisture content throughout its length.

2.4.0 CONCRETE

Concrete refers to a stone-like material produced by mixing cement, coarse aggregate (chippings), fine aggregate (sand) and water when set. The foundation of the compound is the cement, which when mixed with water forms a paste that bonds the aggregates together. This material has been in use for many years and many researches have been carried out to improve the use of concrete. The essential properties of hardened concrete are durability and strength. These properties are affected by the voids and capillaries in the concrete which are caused by incomplete compaction or excess of water in the mix.

2.4.1 Cement

Cement is a clinker, formed by heating limestone and clay, rich in calcium silicates and is ground to a fine powder with a small proportion of gypsum. Gypsum is usually added to regulate the rate of setting when the cement is mixed with water. Ordinary Portland cement is often used for general concrete constructions when there is no exposure to sulphates. Ordinary Portland Cement will therefore be used for this research.

2.4.2 Aggregates

The term 'aggregates' is used to describe gravels, crushed stones and other materials which are mixed with cement and water to make concrete.

Aggregate forms the bulk of the volume of concrete. Good aggregate should be hard and clean and preferably angular in shape. It should not contain materials that can decompose. It should not contain clay. For workability purpose, the aggregate's sphericity should be low, since the lower the sphericity, the higher the workability. The average size of aggregates to be used for concreting depends very much on the use of the concrete, for example, high size of aggregate will be required in foundation and mass concrete works. Also, spacing of reinforcements is a determining factor for aggregate size. Aggregate which has high percentage of large particles is referred to as being coarsely graded and one which has high proportion of small particles is referred to as being finely graded.

2.4.3 Water

Water is used to mix relevant materials such as cement and aggregates together to produce concrete. Its quantity affects workability and strength. Its addition brings reactions (chemical) within cement particles, and between cement and aggregates. Water used for concreting must be very clean and should not contain dissolved salts, suspended solids and organic matters. These affect the setting and hardening of concrete.

Salt can cause corrosion of steel in reinforced conrete. The amount of water added to cement affects the strength of concrete. If the water is not sufficient, incomplete chemical reactions will take place and proper bonding of the materials will be prevented. The water-cement ratio is the combined weight of the mixing water and water in the aggregates divided by the weight of cement. Water-cement ratio usually used in ordinary concreting ranges from 0.40 to 0.60. The drier the mix, the stronger the concrete will be as long as it can be fully compacted. Too much of water can reduce the concrete strength significantly. Greater size of aggregate particles lowers water-cement ratio but increases workability and strength.

2.5.0 REINFORCED CONCRETE

In the ancient times, mass concrete was used for structural elements. It was then found out that for these structural elements to carry their intended loads, the quantity of concrete needed was always excessive, hence the elements were always too bulky and transfered more weights to the foundations. Therefore, there was a need to reinforce the concrete. After many researches, steel reinforcements were introduced and were very effective. More studies and researches brought about the better use of steel reinforcement and its curtailment.

As the whole world is under a crumbling economy, steel becomes too expensive for people to buy especially in Nigeria. Therefore, there is a need to find adequate alternatives to the use of steel to produce reinforced concrete used for structural elements in building industry. The use of Concrete-Timber Composite is one of such alternatives. Many studies and researches have been directed to this area.

2.5.1 Composite action of timbers joined together

It is possible that the need to join timbers together in preparing timber-concrete composite elements may arise, especially when the needed length is more than the length of the timber piece and also when it is necessary to increase the strength of the element. When this is done, composite action must be made to take place by providing rigid interconnections. Nails, bolts and screws can be used as connectors. The degree of composite action in timber systems and even in timber-concrete composite depends on the type and number of connectors used to attach the component parts and on the characteristics of the timber itself. Interlayer slip may results in incomplete composite action, therefore the effects of interlayer movements should be checked. Interlayer connection is always used to improve the performance of the timber beams but where the interconnection is not rigid, the deflection is always very high.

In case of columns, buckling will be high when the interconnection is not rigid or even separation between the timber pieces joined together when subjected to loading.

The use of timber-concrete composite columns in building industry in Nigeria has not been introduced. Therefore, publications on this are not available to me.

2.5.2 Timber - Concrete Composite

Mechanically connected timber-concrete composite - systems have been in use in Nigeria of recent, especially for bridge elements such as decks, T-beams and girders. The design criteria must include recognition that the allowable stresses in tension for the timber girders or beams and in compression for columns or posts and also at the interfaces between the timber and concrete must not be exceeded. Transformed - section concepts can be used in the analysis. It is necessary that there should be continued research to assess the effects of long-term loading, fatique, temperature changes to allow these new systems to be fully utilized.

Composite action in timber-concrete systems is no longer being ignored by researchers and designers. Composite timber-concrete elements have been used effectively in bridge constructions.
Composite timber-concrete construction in use today commonly consist of one of two types given below:-

(1) T - beams: timber stringer stems attached to concrete slab flanges.

(2) Slab - decks: Nailed-laminated plank deck over which concrete is cast monolithically.

It is known that cased timber beams could be used to support concrete floors (slabs) and could be designed as T-beams or L-beams depending on the position of the beams. The concrete therefore forms the flange. To ensure good composite action shear developers can be used. A series of nails (say 32.5mm long) can be nailed to the sides of the timber say at an angle of 45° (25mm into the timber) while the remaining length of these nails (12.5mm) will be in the concrete to improve the bond (or composite action) between the timber and concrete. The nails are slantened so that they can check horizontal shear and vertical separation. Where appreciable horizontal shear is expected, timber should be sawed down a few millimetres into the top surface if used as beams or all the sides if columns or posts (in case shear along the column length is anticipated) at say 300mm to 400mm intervals and the alternate blocks cut out to create a castellated pattern to develop the shear.

2.5.3 Design Considerations

According to Furlong (1979), truly concentric loads can exist only instantaneously for composite columns. Under the action of a concentric compression force, a composite column should shorten in length, and it seems logical to assume that all particles of each cross-section experience the same amount of shortening per unit length.

From the analysis of steel-concrete composite, the analysis of timber-concrete composite could be deduced. For axially concentric column load, the force-strain graphs are given below (Steel-Concrete Composite Column).

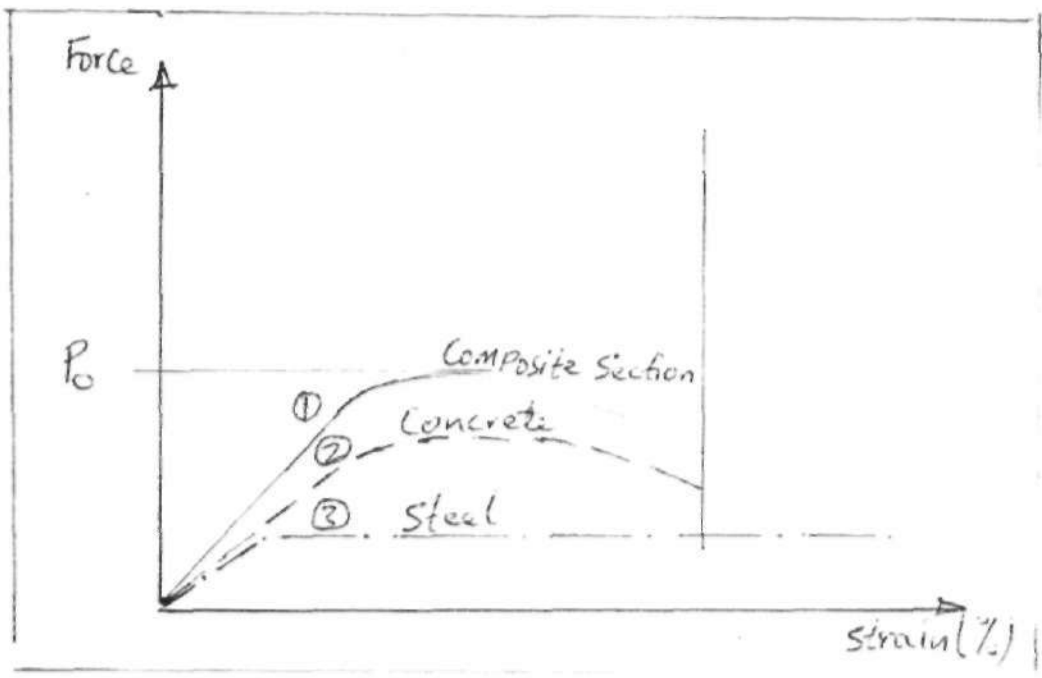

Fig. 2.1 Force-strain graphs for axially concentric column load.

The peak value of force, that is Po, can be evaluated reasonably well by adding together the separate capacities of concrete and steel. This will also hold for timber and concrete. The maximum compression strength

$Po = A_t f_t + 0.85 f_c A_c$ .... (1)

factor 0.85 has been applied to modify the material control values of Concrete Compression Strength.

A_t = Cross-sectional area of timber

f_t = Compressive strength of timber

A_c = Cross-sectional area of concrete

f_c = Compressive strength of concrete

A composite tangent-modulus form of strength-slenderness behaviour represents a lower limit to column strength P_c. A buckling load P_c can be expressed by

$$P_c = \frac{\pi^2 EIa}{(Kl)^2} \qquad (2)$$

according to Knowles and Park (1969). Kl represents the effective length of the compression member and EIa represents an effective tangent - modulus stiffness for the cross-section. EIa value can be estimated from characteristic stress-strain properties of timber and concrete. For purposes of design, stress-strain functions for timber and for concrete, once expressed in analytic form, can be differentiated to obtain the the slope of each as a function of strain. These slopes represent tangent-moduli for each material. Then for any strain level

$$EIa = E_t I_t + \tfrac{1}{2} E_c I_c \qquad (3)$$

only half of the moment of inertia of concrete is suggested here because it should be assumed that something less than the full concrete cross-section can remain uncracked in pure flexure. At the same level of strain for which EIa is evaluated, the corresponding thrust

$$P_{cr} = f_t A_t + f_c A_c \qquad (4)$$

Therefore after EIa and P_{cr} are evaluated, the effective length Kl can be determined thus

$$Kl = \pi \sqrt{\frac{EIa}{P_{cr}}} \qquad (5)$$

Using many values of strain at different levels, and evaluating equation (5) an adequate strength-slenderness design curve can be ploted.

It is to be noted that if P_{cr} is less than $0.5P_o$, long-column behaviour should be assumed, and $EI = E_t I_t + \frac{1}{2} E_c I_c$ should be taken as EIa.

E_c can be estimated from BRS research (76) formula

$$E_c = 9.1\ (\sqrt[3]{f_{cu}}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Therefore, the effective length Kl_c at which long column action commences can be computed from

$$Kl_c = \pi \sqrt{\frac{EI}{0.5P_o}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

In general therefore,

$$P_{cr} = P_o \left(1 - \tfrac{1}{2} \left(\frac{Kl}{Kl_c}\right)^2\right), \quad Kl < Kl_c \ldots\ldots\ldots\ldots \quad (8)$$

and $$P_{cr} = \pi^2 \frac{EI}{(Kl)^2}, \quad Kl > Kl_c \ldots\ldots\ldots\ldots\ldots \quad (9)$$

CHAPTER THREE

PREPARATION, TESTING PROCEDURE OF THE SPECIMENS AND RESULTS

3.1.0 TEST ON PHYSICAL PROPERTIES

3.1.1 Determination of Density of the Timber

Density is defined as the mass of unit volume, and is therefore obtained by dividing the weight by the volume. The density of timber is of practical interest because it is the best single criterion to predict the strength. A relationship exists between the specific gravity and strength because these properties depend much on the thickness of the walls of individual cells and the proportions of the different kinds of tissue in the timber.

Procedure: Three numbers 20mm x 20mm pieces were cut out near the middle point of the timber length. Each was weighed using a balance called OERTLING. Using the weights from the results, the density of the specimen was determined as shown below:

$$\text{Density } (\rho_t) = \frac{\text{Weight (kg)}}{\text{Volume } (m^3)} = \frac{7.41 \times 10^{-3}}{(20 \times 20 \times 20) \times 10^{-9}}$$

$$= 926.3 kg/m^3$$

Table 1 : The Density (ρ_t) of the Timber

Sample	Weight (kg)	Volume (m^3)	Density (kg/m^3)
1	7.41×10^{-3}	8×10^{-6}	926.3
2	7.35×10^{-3}	8×10^{-6}	918.8
3	7.5×10^{-3}	8×10^{-6}	937.5

Average Density Value = $927.5 kg/m^3$

3.1.2 Determination of the Moisture Content of the Timber

Oven-dry method was used to determine the moisture content of the timber. In this method, the moisture content of the moment was obtained as follows:

$$100 \times \frac{\text{Initial weight of sample - dry weight of sample}}{\text{Dry weight of sample}}$$

The initial weight of sample is the actual weight at the time of test, and the dry weight is the weight of the sample after the moisture has been expelled.

Sampling: Three numbers 20mm x 20 mm x 20mm pieces were cut out from the timber length. One at about 600mm from one end of the timber and another at the same distance from the other end. The third sample was cut out from middle.

Apparatus: The apparatus used were a simple balance called OERTLING to weigh the samples, and a drying oven that could be maintained at a constant temperature.

Procedure: Immediately the samples were cut out to shape, they were weighed in turn. This is to minimize the chance of a sample picking up or losing moisture to the outside atmosphere. The samples were then transfered immediately into the oven which has been kept runing at a temperature of 60^{oc} for some hours to prevent the moisture in the centre of the samples from being sealed in, as a result of case-hardening. The temperature in the oven was then raised to 102^{oc}. The samples were left in the oven overnight. The following morning, the samples were re-weighed and put in the oven again for some further hours and re-weighed. The weighings were done very rapidly. The weights got then were close to the former values. These last values were taken as the dry weights of the samples.

The calculation of the moisture content is shown below:

$$\text{Moisture Content}(m) = \frac{\text{Initial weight} - \text{dry weight}}{\text{dry weight}} \times 100$$

Sample 1 : Moisture Content(m) $= \frac{7.39 - 6.9}{6.9} \times 100 = 7.1\%$

Table 2 : Moisture Content(m) of the Timber

Sample	1st (Initial) weight (g)	2nd weight (g)	3rd (dry) weight (g)	Moisture Content (%)
1	7.39	6.95	6.90	7.1
2	7.55	7.10	6.95	8.6
3	7.61	7.16	7.10	7.2

Average Moisture Content of the Timber = 7.6%

3.2 TEST ON MECHANICAL PROPERTIES OF TIMBER

3.2.1 Determination of Compressive Strength Parallel to Grain

It is known that high strength in compression parallel to timber's grain implies that the timber is good for columns, posts etc. The sample taken here is large in cross-sectional area compare to the length because in practice the column's lengths are always large compare to the cross-sectional sizes and therefore failure is more likely in bending.

Sampling: Four numbers 60mm x 20mm x 20mm pieces of the timber were cut out. The 60mm length was along the grain.

Machine: Ivery Denison Compression Testing Machine Model T.I.B./MC and maximum capacity of 2500KN was used to crush the specimens. Unfortunately, this machine could not plot graphs nor give deflection values. The only reading that could be recorded was the failure load.

Procedure: The test piece was placed on end of the flat surface bearing of the machine, and the load was then applied through a plate acting on the full sectional area of the test piece, and parallel to the grain of the timber. The specimen was loaded to failure and the maximum failure load was recorded. The calculations of the stresses are as follows:

$$\text{Compressive stress } (\sigma_p) = \frac{\text{Load}}{\text{Cross-Sectional area}}$$

$$\text{Sample 1} : \frac{7.2 \times 10^3}{20 \times 20} = 18 \text{ N/mm}^2$$

Table 3 : Compressive Stress Parallel to the grain of the Timber

Sample	Sizes (mm)	Failure Load (N)	Compressive Stress (N/mm^2)
1	60 x 20 x 20	7200	18.00
2	60 x 20 x 20	7100	17.75
3	60 x 20 x 20	8000	20.00

Average Compressive Stress (σ_p) = 19.04 N/mm^2

3.2.2 Determination of Modulus of Elasticity (Dynamic) of Timber Using Ultrasonic Pulse Velocity Equipment (PUNDIT)

Pundit equipment is an electrical equipment used to determine the modulus of Elasticity of a material. The equipment sends electrical waves through the specimen and the time in micro-seconds taken the waves to pass through is recorded by the equipment, from which velocity can be calculated.

Sampling: Four pieces of the timber with different lengths were cut out of the plank. The ends of each timber piece were greased. Each piece was then weighed.

Procedure: The ends of the two transducers of the equipment were greased. The ends of the reference metal bar were also greased and the transducers were placed one at one end of the bar and the other at the other end. The equipment was then adjusted to a reference value. The specimen was then greased at both ends and the transducers were placed one at one end of the specimen and the other at the other end of the specimen. The machine was then put on and the time was recorded by the machine. The calculation of the modulus of Elasticity is shown below:

For Sample 1:

Specimen size = 38mm x 39mm x 506mm long.

Weight of the specimen = 561.69(g)

Time recorded = 101 μsec.

$\therefore$ Density of Specimen $(\rho_t) = \dfrac{651.6}{38 \times 39 \times 506} = 7.479 \times 10^{-4}\, g/mm^3$

$$\text{Velocity} = \frac{\text{Length}}{\text{Time}} = \frac{506}{101 \times 10^{-3}} = 5009.9 mm/sec.$$

From the following formular, modulus of elasticity can be calculated

$$\text{Velocity} = \sqrt{\frac{E_t(1 - V_t)}{\rho_t\,(1 + V_t)\,(1 - 2V_t)}}$$

where V_t = Poisons Ratio = 0 for wood

ρ_t = density

E_t = modulus of Elasticity

$$\therefore \quad 5009.9 = \sqrt{\frac{E_t(1 - 0)}{7.49 \times 10^{-4}(1 + 0)(1 - 2 \times 0)}}$$

$E_t = 18799 N/mm^2$

Table 4 : Modulus of Elasticity of the Timber by Pundit Test

Sample	Length (mm)	Time (μsec.)	Cross Section mm x mm	Weight (g)	Velocity mm/sec	Density g/mm^3	Modulus of Elasticity (N/mm^2)
1	506	101	38 x 39	561.6	5009.9	7.49×10^{-4}	18799
2	510	104	39 x 39	690.4	4903.8	8.9×10^{-4}	21402
3	510	101	39 x 39	564.5	5049.5	7.28×10^{-4}	18562

Average Modulus of Elasticity E_t = $19.93 KN/mm^2$

3.2.3 Determination of bonding strength between the timber and the Concrete

This test is necessary because of the influence the bond between the timber and concrete has on the overall strength of the specimen. If composite action does not take place, the concrete will separate from the timber easily under loading.

Sampling: Four pieces of timber were cut out of the plank. The average cross-sectional area of the timber piece was 24mm x 24mm.

Machine: Universal Testing Machine Model 7104 DCJ and maximum capacity of 1000KN was used for the tests.

Procedure: Fresh concrete of characteristic strength of $22.5N/mm^2$ (1:2:4 mix) used for preparing the timber-concrete column specimens was poured into metalic cylinderical mould into which the timber piece was already placed centrally and vertically. The concrete was poured to different levels, so that the length of the timber in the concrete varied from cylinder to cylinder. The following day, the concrete with the timber at the centre was carefully removed from the mould and placed in water for curing in the curing room.

Water level was just very close to the brim of the concrete so that water would not penetrate through the interface of the timber and concrete from the top. On the 28th day, the specimens were removed from water for testing. The upper jaw of the machine held the timber securely while the lower jaw held the concrete firmly and pulling force was applied by the machine until the timber was pulled out of the concrete. The pulling force was recorded. The bond strength was calculated as follows:-

For sample 1: Cross-section of the timber = 25mm x 25mm

Length of timber in concrete = 300mm

Concrete-timber contact area = 4(25 x 300)

= $30000mm^2$

Pulling force = 58.8KN

$$\text{Bond Stress (fct)} = \frac{58.8 \times 10^3}{30000} = 1.96N/mm^2$$

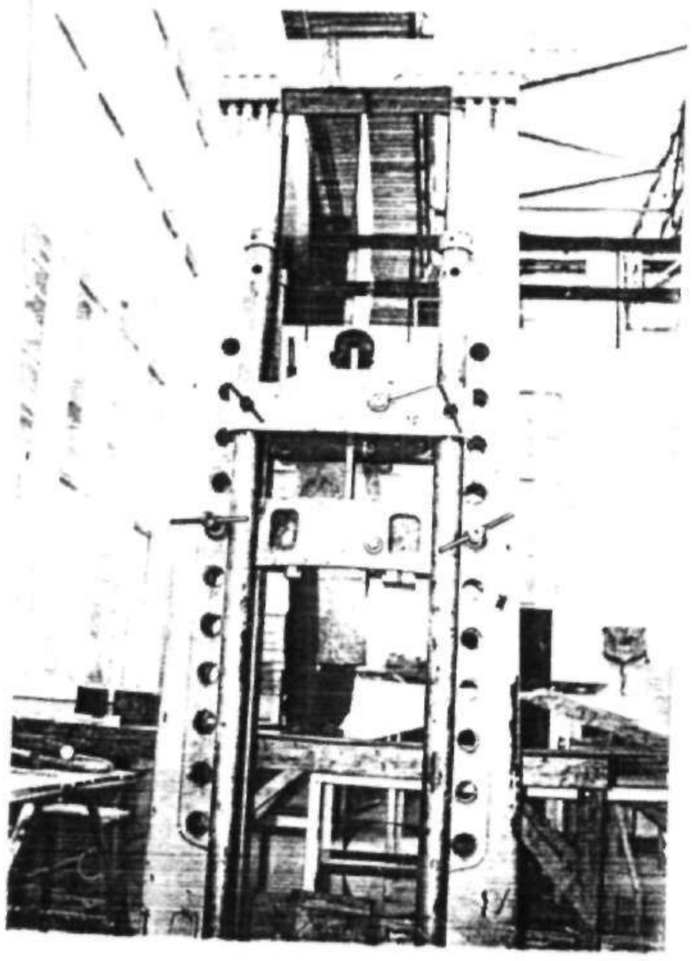

Plate i: Experimental test for bond strength between Concrete and Timber.

Table 5 : Bond Strength between the Timber and Concrete

Sample	Timber Size (mm x mm)	Length of Timber in the Concrete (mm)	Timber–Concrete Contact area (mm^2)	Pulling Force (KN)	Bond Stress (N/mm^2)
1	25 x 25	300	30,000	58.80	1.96
2	24 x 24	300	28,800	55.87	1.94
3	24 x 24	225	21,600	43.20	2.00
4	21 x 21	150	12,600	24.00	1.90

Average Bond Strength (fct) = 1.95 N/mm^2

3.2.4 Determination of the Concrete Cube Strength

Below is the table of the Concrete Cube Strength for each Specimen:

Table 6 : Strength of Concrete from Concrete Cube Tests

Sample	Size of Cube (mm)	Age of Cube(days)	Weight of Cube (kg)	Failure Load(KN)	Strength of Concrete(N/mm^2)
1_A	150x150x150	36	7.8	550	24.4
1_B	150x150x150	36	7.5	560	24.9
1_C	150x150x150	36	7.5	530	23.6
2_A	150x150x150	33	7.5	515	22.9
2_B	150x150x150	33	7.4	513	22.8
2_C	150x150x150	33	7.5	516	22.9
3_A	150x150x150	32	7.6	502	22.3
3_B	150x150x150	32	7.5	513	22.8
3_C	150x150x150	32	7.5	506	22.5
4_A	150x150x150	34	7.8	475	21.1
4_B	150x150x150	34	7.8	470	20.9
4_C	150x150x150	34	8.0	470	20.9

Sample	Size of Cube (mm)	Age of Cube (days)	Weight of Cube (kg)	Failure Load (KN)	Strength of Concrete (N/mm^2)
5_A	150x150x150	32	7.8	502	22.3
5_B	150x150x150	32	7.6	516	22.9
5_C	150x150x150	32	7.6	495	22.0

3.3 PREPARATION OF THE TIMBER-CONCRETE COLUMN SPECIMENS

3.3.1 Timber

Mahogany from dry zone was used. The timber was already seasoned. Moisture content tests were carried out and recorded in this chapter (Table 2). From the Nigerian Code of Practice on Nigerian Timbers, this type of mahogany has the following properties:

Botanical Name	Density at 18% Moisture Content (kg/m^3)	Strength Group	Natural Durability	Resistance to Impregnation	Movement	Shrinkage
Khaya Senegaless	816	N_3	Durable	Very Resistance	Small	Medium

Boreholes in the timber piece are about 5 in $1000cm^2$ and have diameters between 1.5mm to 3.0mm. There is no decayed part in the piece. Checks are about $\frac{1}{2}$ thickness of member. Therefore, the timber is in grade 63.

The timbers were sawn into cross-section 38mm x 38mm and of various lengths as shown below:

(A) 5 No x 38 x 38 x 1200mm

(B) 5 No x 38 x 38 x 1000mm

(C) 5 No x 38 x 38 x 800mm

(D) 5 No x 38 x 38 x 600mm

(E) 5 No x 38 x 38 x 400mm

The timber pieces were not smoothened to improve the bond strength between the concrete and the timber. Grooves were not made on the timber pieces and nails were not applied as means of improving the bond because the timber size is small; these may weaken the timber if applied.

3.3.2 Concrete

The coarse aggregates are crushed aggregates, most of which are angular in shape. The aggregates are hard and without dirts. It was well graded. The maximum size was 9mm. Dusts were sieved out of the aggregates. The fine aggregates were sharp sands and sieved with B. S. sieve 4.76. The slump was between 75mm and 90mm. Mechanical concrete mixer was used. Ashaka Ordinary Portland Cement was used. The quality of the cement could be said to be alright because there were no traces of caked cement.

3.3.3 Steel Reinforcements

A steel fabric reinforcement was used to envelope the timber piece. This is to check any cracking that may occur and to increase the cover concrete strength. The first twenty-five specimens have this steel fabric reinforcement. Another three specimens were prepared without steel fabric reinforcement but with the same curing method as used for the twenty-five specimens. A set of another three specimens were also prepared without steel fabric reinforcement, but different curing procedure. These last three samples were not wetted with water as others but sacks were spread on them in the curing room for the usual 28 days. Both longitudinal and transverse steel, making the mesh are averagely 2.95mm in size and the pitch which is square is 50mm x 50mm centre to centre of the steel. The steel fabric was bent by the machine to the required shape.

3.3.4 Formworks

18.mm thick plywood was used to prepare the forms. The forms for every set of five column specimens were seated on planks levelled enough for that purpose. The column's forms were then braced together at intervals from bottom to the top. Plumb was used to make sure that the forms were vertically erected. Apart from bracing the column forms together, each form was braced squarely at the bottom, middle and top positions. Steel clamps were also employed when necessary.

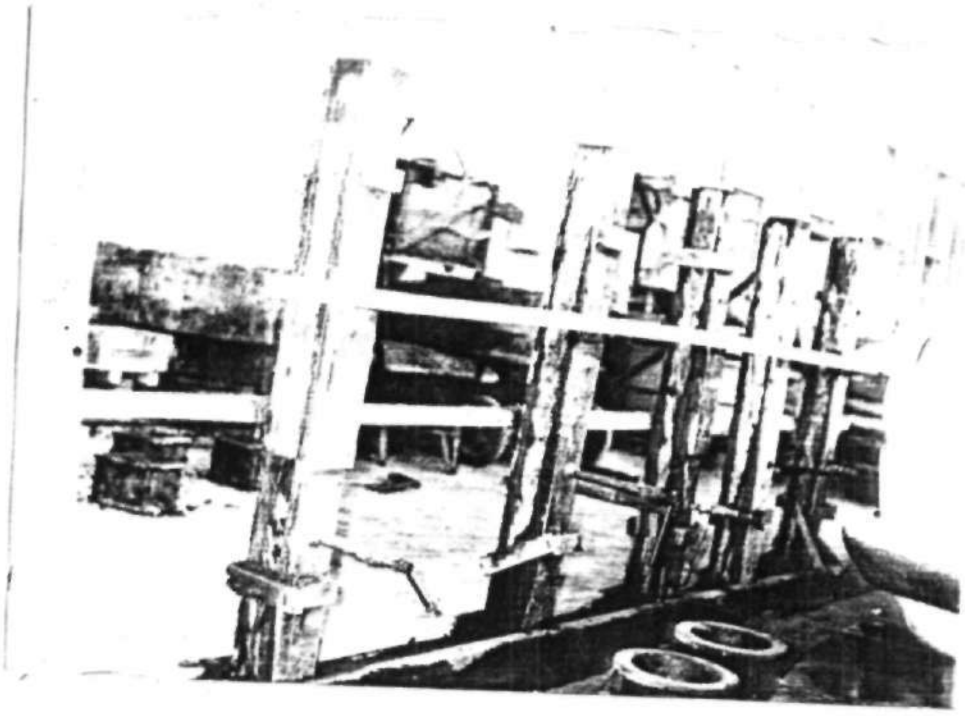

Plate ii: Formworks prepared for the column specimens.

3.3.5 Concrete Placement

The central point of each timber, cross-sectionally, was located at each end and a small nail was nailed at this position and passed through the form's end cover to keep the timber at the centre of the form. Then the steel fabric was placed and lapped properly in between the timber and the inner face of the form. The five forms were oiled. When the concrete was thoroughly mixed and slump tests were done, the concrete was then poured into the form gradually and external vibrator employed until the form was half filled. This procedure was followed for all the five forms. Then starting with the first one again the remaining half of the height of the form was filled and vibrated well enough to eradicate honey combs. This system was chosen so as to minimize any difference that may arise within the five samples due to concrete placement and to make sure that the concrete for the five samples is uniform.

3.3.6 Curing

The specimens were left in their forms till the third day and then removed to curing room. The samples were watered and covered with soaked sacks. This curing continued for at least 28 days. There was a set made up of three specimens prepared in the same way as the first twenty-five specimens but without steel fabric.

The curing was in the same pertern. Another three specimens were prepared without steel fabric reinforcement but were just covered by sacks without wetting them with water.

3.4 Testing of the Timber-Concrete Column Specimens and Test Results

3.4.1 Test of columns with Steel Fabric

The first three sets of the columns were taken out and painted with white paint, ready for testing. It was not possible to test each set at the 28 days because the machine to be used was on repair. After the paint had been allowed to dry off, the former numbers of the samples were re-printed on them. The first set of five samples were numbered 1_A to 1_E and the second set 2_A to 2_E, likewise other sets. Universal testing machine was used for the tests. The machine could record the failure load and plot the graph of load against strain. All the column specimens were loaded axially and compressive failure force recorded. The base on which the sample was placed could rotate freely (Pin jointed) to remove or minimize shearing effects. Celotex boards were placed on both ends of the sample during the test to allow uniform distribution of the loads on the column. Plumb was also used to make sure that the column was vertical, so as to eliminate or minimize eccentricity. Table 7 shows the results of all the columns tested.

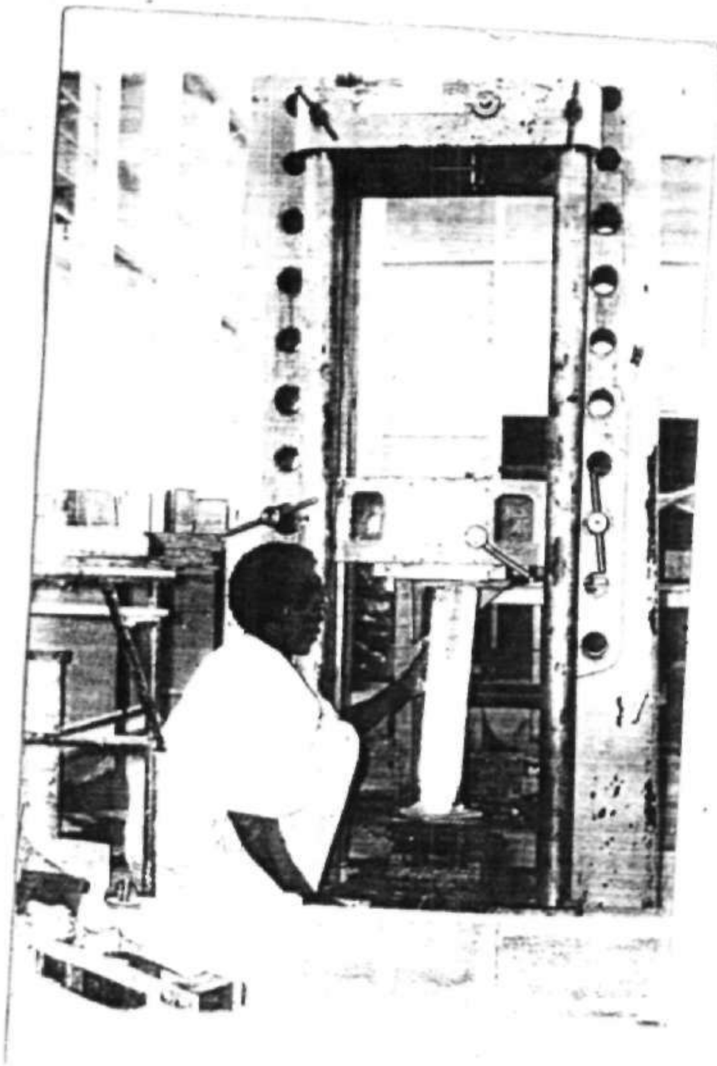

Plate iii: Columns Specimen on test

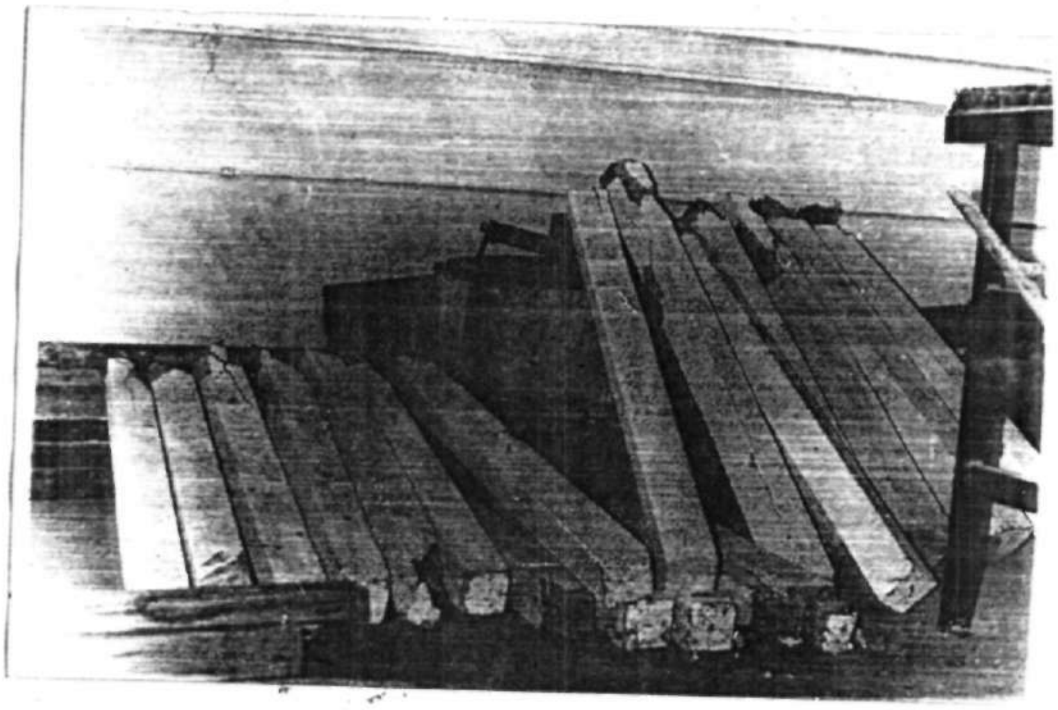

Plate iv: Column Specimen after test

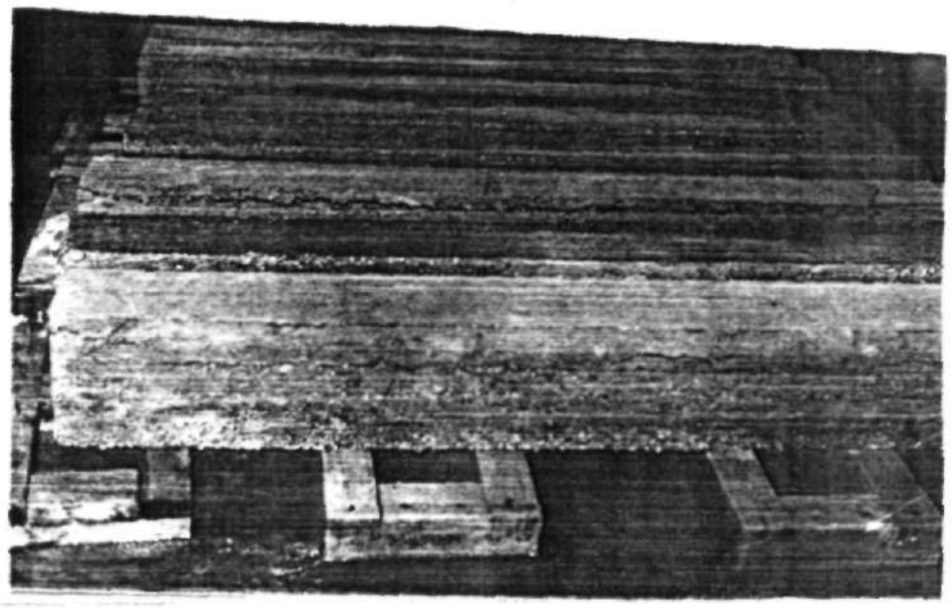

Plate v: Column specimens without steel fabric cracked before test.

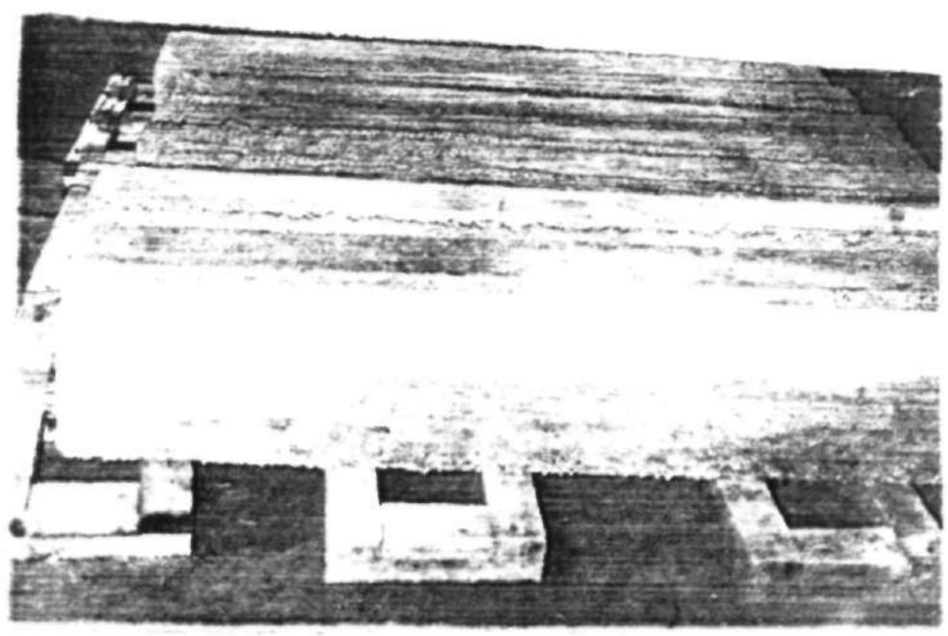

Plate vi: Column specimens without steel fabric cracked before test.

3.4.2 Test of Columns without Steel Fabric

The remaining specimens without steel fabric reinforcements had serious cracks, even those three specimens that were not wetted with water but just covered by sacks. This was counted as failure and therefore they were not tested.

TABLE 7 : TEST RESULTS OF ENCASED TIMBER COLUMNS LOADED AXIALLY

COLUMN SET	OVERALL SIZE (mm x mm)	TIMBER SIZE (mm x mm)	CONCRETE COVER (mm)	SLENDERNESS RATIO	DATE CAST	DATE TESTED	CONCRETE AGE DAYS	CONCRETE MIX	WEIGHT OF COLUMN (kg)	FAILURE LOAD KN	COMMENT
1_A	75 x 75	38 x 38	18.5	16	19/03/1990	23/04/1990	36	1:2:4	15.0	118.5	
1_B	75 x 75	38 x 38	18.5	16	19/03/1990	23/04/1990	36	1:2:4	15.2	122.0	
1_C	75 x 75	38 x 38	18.5	16	19/03/1990	23/04/1990	36	1:2:4	15.7	120.5	
1_D	75 x 75	38 x 38	18.5	16	19/03/1990	23/04/1990	36	1:2:4	14.8	126.0	
1_E	75 x 75	38 x 38	18.5	16	19/03/1990	23/04/1990	36	1:2:4	14.6	119.5	
2_A	75 x 75	38 x 38	18.5	13.5	22/03/1990	23/04/1990	33	1:2:4	12.6	138.4	
2_B	75 x 75	38 x 38	18.5	13.5	22/03/1990	23/04/1990	33	1:2:4	13.2	132.0	
2_C	75 x 75	38 x 38	18.5	13.5	22/03/1990	23/04/1990	33	1:2:4	13.5	136.0	
2_D	75 x 75	38 x 38	18.5	13.5	22/03/1990	23/04/1990	33	1:2:4	13.1	129.5	
2_E	75 x 75	38 x 38	18.5	13.5	22/03/1990	23/04/1990	33	1:2:4	13.5	79.0	Steel Fabric Splited at lap.
3A	75 x 75	38 x 38	18.5	10.7	23/03/1990	23/04/1990	32	1:2:4	10.2	89.6	Steel fabric splited at lap
3_B	75 x 75	38 x 38	18.5	10.7	23/03/1990	23/04/1990	32	1:2:4	10.5	158.2	
3_C	75 x 75	38 x 38	18.5	10.7	23/03/1990	23/04/1990	32	1:2:4	10.6	163.0	

TABLE 7 CONTINUED

COLUMN SET	OVERALL SIZE (mm x mm)	TIMBER SIZE (mm x mm)	CONCRETE COVER (mm)	SLENDERNESS RATIO	DATE CAST	DATE TESTED	CONCRETE AGE DAYS	CONCRETE MIX	WEIGHT OF COLUMN (kg)	FAILURE LOAD KN	COMMENT
3_D	75 x 75	38 x 38	18.5	10.7	23/03/1990	23/04/1990	32	1:2:4	10.6	164.0	
3_E	75 x 75	38 x 38	18.5	10.7	23/03/1990	23/04/1990	32	1:2:4	10.6	168.0	
4_A	75 x 75	38 x 38	18.5	8.0	28/03/1990	30/04/1990	34	1:2:4	7.3	184.0	
4_B	75 x 75	38 x 38	18.5	8.0	28/03/1990	30/04/1990	34	1:2:4	7.6	190.0	
4_C	75 x 75	38 x 38	18.5	8.0	28/03/1990	30/04/1990	34	1:2:4	7.6	189.1	
4_D	75 x 75	38 x 38	18.5	8.0	28/03/1990	30/04/1990	34	1:2:4	7.2	182.3	
4_E	75 x 75	38 x 38	18.5	8.0	28/03/1990	30/04/1990	34	1:2:4	7.6	188.0	
5_A	75 x 75	38 x 38	18.5	5.3	30/03/1990	30/04/1990	32	1:2:4	5.5	198.0	
5_B	75 x 75	38 x 38	18.5	5.3	30/03/1990	30/04/1990	32	1:2:4	5.4	200.00	
5_C	75 x 75	38 x 38	18.5	5.3	30/03/1990	30/04/1990	32	1:2:4	5.6	209.1	
5_D	75 x 75	38 x 38	18.5	5.3	30/03/1990	30/04/1990	32	1:2:4	5.5	196.0	
5_E	75 x 75	38 x 38	18.5	5.3	30/03/1990	30/04/1990	32	1:2:4	5.6	169.7	The concrete around the top end removed

CHAPTER FOUR

ANALYSIS AND COMPARISON OF RESULTS

4.1 COLUMN SPECIMENS

The test results of all the timber-concrete composite columns were given in Table 7 in chapter three. Here, the analysis of the columns theoretically will be carried out and the corresponding failure loads will be presented and compared to the values got from the tests. Method of transformed sections will be used for the analysis. There are two groups of the columns, one being short columns and the other slender columns.

4.1.1 THEORETICAL ANALYSIS

4.1.2 Method of Transformed Section

When two materials are involved in a section, for the purpose of analysis only, one material can be converted to the other by replacing it with an equivalent area, so that the section can be analysed as if it were made up of a single material. This method is known as method of transformed section. This method can only be useful when there is composite action between the two materials. Considering the section of the specimen shown in Fig4.1 below, the section is assumed to be under the action of a bending moment M applied about say CX.

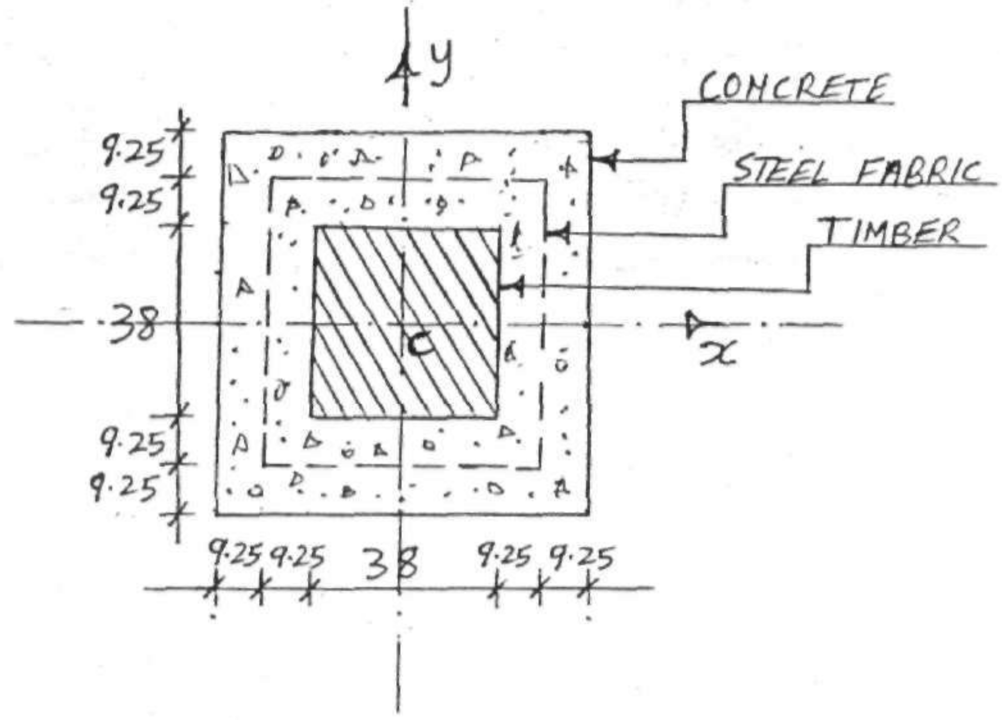

Fig 4.1 : Cross-section of the Specimen (Dimensions in mm)

If the timber is bent into a curve of radius R, then, from simple theory of bending, the bending moment carried by the timber is

$$M_t = \frac{(EI)_t}{R} \quad \ldots\ldots\ldots\ldots \quad (1)$$

Where $(EI)_t$ is the bending stiffness of the timber. If the concrete with the steel fabric reinforcement is bonded to the timber securely (this is true considering the bond strength test results in table 5), composite action will take place and therefore, the concrete with the steel fabric will bend to the same radius of carvature R as the timber.

The bending moments carried by the concrete and the steel fabric on both sides are then

$$M_c = \frac{(EI)_c}{R} \quad \text{and} \quad M_s = \frac{(EI)_s}{R}$$

Where $(EI)_c$ is the bending stiffness of the concrete and $(EI)_s$ is the bending stiffness of the steel fabric.

The total bending moment is then

$$M = M_t + M_c + M_s = \frac{1}{R}\left[(EI)_t + (EI)_c + (EI)_s\right]$$

This gives $$\frac{1}{R} = \frac{M}{(EI)_t + (EI)_c + (EI)_s} \quad \ldots\ldots\ldots \quad (2)$$

Clearly, the section behaves as though the total bending stiffness EI were

$$EI = (EI)_t + (EI)_c + (EI)_s \quad \ldots\ldots\ldots \quad (3)$$

If E_t, E_c and E_s are the values of Young's Modulus for timber, concrete and steel respectively, and if I_t, I_c and I_s are the second moments of area about cx of the timber, concrete and steel respectively, then

$$EI = (EI)_t + (EI)_c + (EI)_s = E_t I_t + E_c I_c + E_s I_s \quad \ldots \quad .(4)$$

Then $$EI = E_c\left[\frac{(E_t)}{(E_c)} I_t + I_c + \frac{(E_s)}{(E_c)} I_s\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

If I_t and I_s are multiplied by $\frac{(E_t)}{(E_c)}$ and $\frac{(E_s)}{(E_c)}$

respectively, which are the ratio of Young's moduli for timber and concrete, and steel and concrete respectively, then from equation (5), we can see that the composite section may be treated as wholly concrete, having an equivalent second moment of area

$$I_e = I_c + \frac{(E_t)}{(E_c)} I_t + \frac{(E_s)}{(E_c)} I_s \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

The bending stress σ_{cs} in a fibre of the concrete care of the section a distance y from the neutral axis is

$$\sigma_{cs} = \frac{M_c y}{I_c}$$

but $M_c = \frac{(EI)_c}{R}$, and $M = \frac{1}{R}\left((EI)_c + (EI)_t + (EI)_s \right)$

on eliminating R

$$M_c = \frac{M}{1 + \left(\frac{E_t I_t}{E_c I_c}\right) + \left(\frac{E_s I_s}{E_c I_c}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

Then

$$\sigma_{cs} = \frac{Mcy}{I_c\left(1 + \frac{E_t I_t}{E_c I_c} + \frac{E_s I_s}{E_c I_c}\right)}$$

$$\sigma_{cs} = \frac{Mcy}{I_c + \frac{(E_t)}{(E_c)} I_t + \frac{(E_s)}{(E_c)} I_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

The bending stresses in the concrete core are found therefore by considering the total bending moment M to be carried by the transformed concrete section.

The longitudinal strain at the distance y from the neutral axis CX is

$$\varepsilon = \frac{\sigma_{cs}}{E_c} = \frac{Mcy}{E_c I_c + E_t I_t + E_s I_s}$$

Then at the distance y from the neutral axis the stress in the timber is

$$\sigma_{ts} = E_t \varepsilon = \frac{Mty}{I_t + \frac{(E_c)}{(E_t)} I_c + \frac{(E_s)}{(E_t)} I_s} \quad \ldots\ldots\ldots\ldots \quad (9)$$

and the stress in the steel at this level is

$$\sigma_{ss} = E_s \varepsilon = \frac{Mty}{I_s + \frac{(E_c)}{(E_s)} I_c + \frac{(E_t)}{(E_s)} I_t} \quad \ldots\ldots\ldots\ldots \quad (10)$$

Since the strains in the timber, concrete and the steel are the same at the same distance y from the neutral axis. This condition of equal strains is implied in the assumption made earlier that the timber, concrete and steel components of the section are bent to the same radius of curvature R. It is also possible to replace the timber and the steel by an equivalent area of concrete respectively. To do this, it is necessary to obtain a force in the 'equivalent' concrete equal to that in the timber; an area of concrete A_c' must be used such that

$$\sigma_c A_c' = \sigma_t A_t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

Likewise it is necessary to obtain a force in the 'equivalent' concrete equals to that in the steel, an area of concrete, A_c'' must be used such that

$$\sigma_c A_c'' = \sigma_s A_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

but $E = \dfrac{\sigma}{\varepsilon} \Longrightarrow \sigma = E\varepsilon$

Therefore, from equation (11) substitute for σ_c and σ_t

$$\varepsilon_c E_c A_c' = \varepsilon_t E_t A_t$$

Therefore $$A_c' = \frac{\varepsilon_t}{\varepsilon_c} \frac{E_t}{E_c} A_t \quad \text{...............} \quad (13)$$

$$A_c'' = \frac{\varepsilon_s}{\varepsilon_c} \frac{E_s}{E_c} A_s \quad \text{...............} \quad (14)$$

However, since the equivalent concrete area for timber and for steel are to be placed at the same level as the timber and steel respectively, the strain in the equivalent concrete must equal to the strain in the timber; the same holds in the steel.

Therefore $$A_c' = \frac{E_t}{E_c} A_t \quad \text{....................} \quad (15)$$

and $$A_c'' = \frac{E_s}{E_c} A_s \quad \text{....................} \quad (16)$$

To find the position of the neutral axis which may be considered as passing through the centroid of the section, in this case, the section under consideration is the transformed section; it is only necessary to determine the centroid of the section, by equating the first moments of area of the transformed section above and below the neutral axis. For all the specimens of this project, the neutral axis is still passing through the centre of the section after transformation.

4.1.3 Elastic Buckling of the Column Specimen

When a structural member is under compressive stresses, the member may develop relatively large distortions under certain critical loading conditions. Such structural member is said to buckle, or become unstable, at these critical loads.

A perfectly straight element of uniform cross-section has two axes of symmetry CX and Cy shown below (Fig 4.3 c) The element is assumed to be pin-jointed at both ends. It should be noted that the neutral axis of our section (specimen) passes through the centre point of the section and also, during the testing of the specimens, each one was pin-jointed at both ends. The end thrusts P are applied along the centroidal axis CZ of the column. Supposing L is the length of the column and EI is the flexural stiffness of the transformed section for bending about CX. Since the section is square, the question of major axis and minor axis does not arise. Therefore, bending can take place in the yZ or XZ plane. It will be assumed here also that the contribution of the steel (vertical rods) in carrying the axial load is negligible because the pitch of the steel fabric is 50mm and therefore not more than two vertical rods can be at each face of the specimen and also the rod's diameter is only 2.9mm.

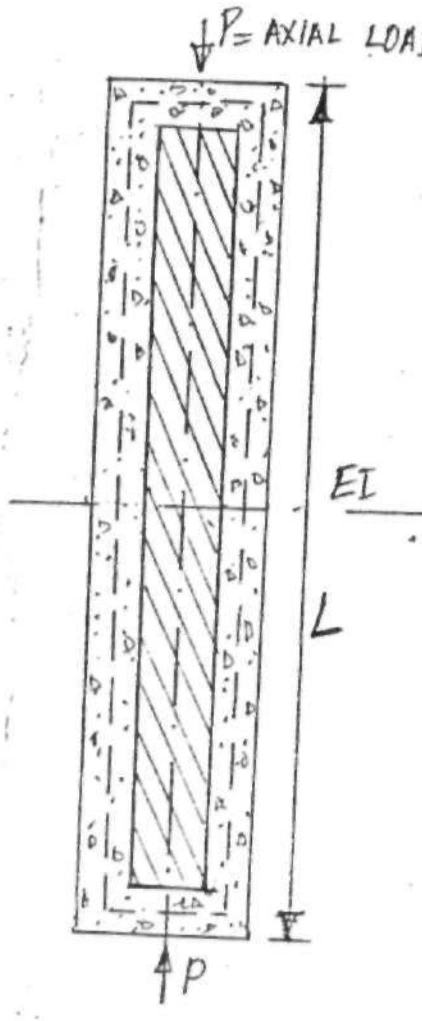

Fig.4.2(a)
Longitudinal Section
of the specimen

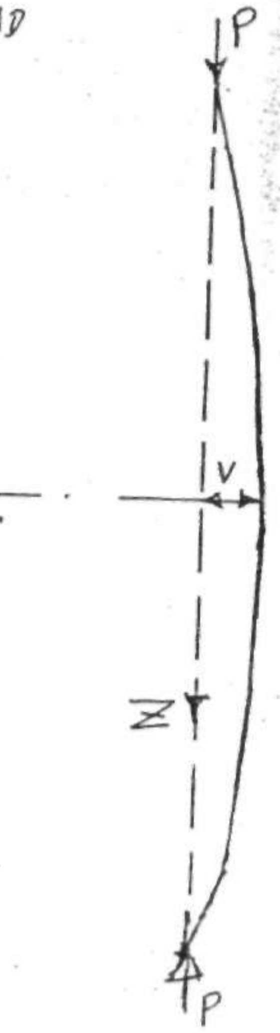

Fig. 4.2(b): Assumed deflected
specimen

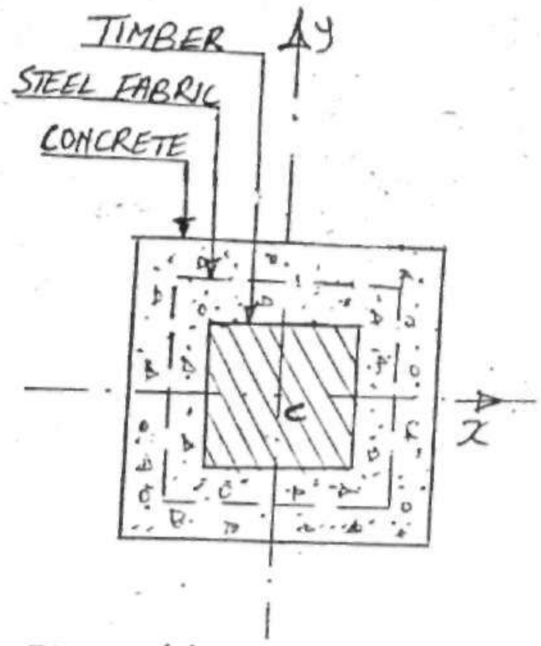

Fig. 4.2(c) : Cross-section
of the specimen

Consider the possibility that at some value of the end thrust P, the column can buckle laterally in the yZ - plane. There can be no lateral deflections at the ends of the column; suppose V is the displacement of the centre line of the column parallel to Cy at any point. Note also that at the end supports, there should be no bending moment since they are pin-jointed. The force P is also assumed to maintain its original line of action after the onset buckling. The bending moment at any section is:

$$M = PV \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

Where M is the total bending moment on the section i.e $M = M_t + M_c$ which is a sagging moment in relation to the axes Cx and Cy. But, the moment curvature relation for the beam at any section is

$$M = -EI\frac{d^2v}{dZ^2} \quad \ldots\ldots\ldots \quad (18)$$

Provided the deflection v is small.

$$\text{Therefore,} \quad -EI\frac{d^2v}{dZ^2} = PV \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (19)$$

$$\Longrightarrow \quad EI\frac{d^2v}{dZ^2} + PV = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

Let $\frac{P}{EI} = K_s^2$

$$\text{Then} \quad \frac{d^2v}{dZ^2} + K_s^2V = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

The general selution of this differential equation is

$$V = A\cos K_sZ + B\sin K_sZ \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (22)$$

Where A and B are arbitrary constants. We have two boundary conditions to satisfy:

at the ends $Z = 0$ and $Z = L$, $V = 0$

Then $A = 0$ and $B \sin K_s L = 0$ .... (23)

Considering the implications of the equation $B\sin K_s L = 0$, which is derived from the boundary conditions. If $B = 0$, then both A and B are zero, and obviously the column is undeflected. If, however, $\sin KL = 0$, B is indeterminate, and the column may assume the form

$$V = B\sin K_s Z \quad \text{..................} \quad (24)$$

This is the buckled condition of the column.

It can be seen that B is indeterminate when $K_s L = \pi$, 2π ...... etc.

We can not consider the solution $KL = 0$, which implies $K = 0$, since the solution of the differential equation is not trigonometric in form when $K = 0$. Instability occurs when

$$P = K_s^2 EI = \frac{\pi^2 EI}{2} \quad \text{etc.}$$

The fundamental mode occurs at the lowest critical load

$$P_e = \frac{\pi^2 EI}{L^2} \quad \text{.......................} \quad (25)$$

This is known as Euler formular and corresponds with buckling in a single longitudinal half-wave.

4.1.4 Limitations of Euler Theory

The assumptions made that the element is perfectly straight and of uniform cross-section and that the end load is applied axially through the centroid, ideally is not practicable. There is always some eccentricity and initial curvature present. There is always considerable uncertainty as to the values of these deviations. Therefore, it is necessary to apply an empirical formula for the analysis of these specimens. Due to these imperfections, the columns will sufer a deflection which increases with the load and consequently a bending moment is introduced which caused failure before the Euler load is reached. The fact is that failure is by stress rather than buckling. The deviation from Euler value is more marked as the slenderness ratio say l/r is reduced.

From Euler formular derived in 4.1.2, for pin-ended columns.

$$\sigma_e = \frac{P_e}{A} = \frac{\pi^2 EI}{AL^2}$$

$$= \frac{\pi^2 E}{(l/r)^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (26)$$

giving the curve shown below (Fig. 3)

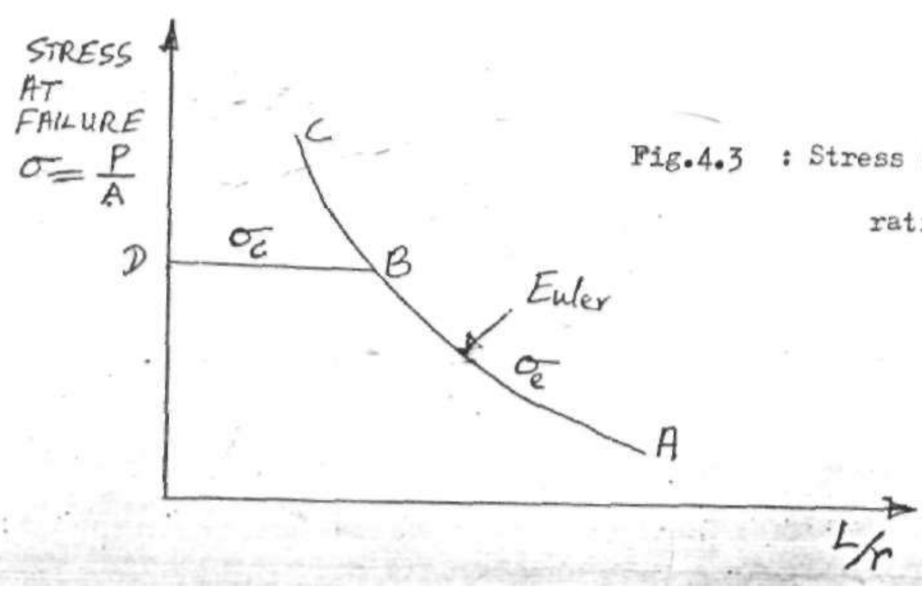

Fig.4.3 : Stress (σ) - Slenderness ratio (L/r) Curve

If σ_e exceeds σ_c, the elastic limit or yield stress in compression, the column must fail by crushing along the line BD (this is the region of short columns Allowing for imperfections of loading and column, actual values at failure must lie within and below ABD.

4.1.5 Rankine - Gordon Formula

Rankine and Gordon derived an expression for the failure load on columns which accounts for the imperfections that may occur due to initial bending of column or initial eccentricity. If σ is the actual stress to cause failure and σ_c and σ_e as defined in 4.1.3, the equation

$$\frac{1}{\sigma} = \frac{1}{\sigma_c} + \frac{1}{\sigma_e} \quad \text{...........} \quad (27)$$

will produce a curve which is tengential to σ_c as $L/r \rightarrow 0$, and tangential to σ_e as $L/r \rightarrow \infty$. This satisfies both limiting conditions, and for intermediate values will be less than both σ_c and σ_e, from equation (27)

$$\sigma = \frac{\sigma_c \sigma_e}{\sigma_c + \sigma_e} = \frac{\sigma_c}{1 + \frac{\sigma_c}{\sigma_e}}$$

For a pin-ended column, from equation (26)

$$\sigma_e = \frac{\pi^2 E r^2}{L^2}$$

$$\therefore \sigma = \frac{\sigma_c}{1 + \frac{(\sigma_c)}{(\pi^2 E)} \left(\frac{L}{r}\right)^2} \quad \text{..............} \quad (28)$$

$\frac{\sigma_c}{\pi^2 E}$ can be replaced by a constant say K. To make allowance for unknown imperfections, the value of K depending on the material and on the end conditions.

Rankine and Gordon then give the load as

$$P = \sigma A = \frac{\sigma_c A}{1 + K\left(\frac{L}{r}\right)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (29)$$

Where r = radius of gyration.

This formular will then be used to calculate all the failure loads on the columns theoretically and compared to the loads recorded from the tests.

4.2.0 Calculation of the failure loads for the Columns

$$P = \frac{\sigma_c A}{1 + K\left(\frac{L}{r}\right)^2}$$

σ_c (working stress) recommended for timber in using this formular = 35 N/mm^2 and $K = \frac{1}{3000}$

$A = A_t + A_c$ .... (30) (from transformed section see 4.1.1)

$$A = A_t + \frac{E_c}{E_t} A_c \quad \ldots\ldots\ldots\ldots \quad (31)$$

A_t (cross-sectional area of timber) = 1444mm^2

A_c (cross-sectional area of concrete) = 4181mm^2

Young's Modulus for concrete (E_c) will be estimated from BRS research (76) formula

$$E_c = 9.1\sqrt[3]{f_{cu}} \quad \ldots\ldots\ldots\ldots \quad (32)$$

The concrete strength will be taken from table 6 (average).

E_t (Young's modulus for timber) = 19.93 KN/mm^2

(average see table 4)

$$r \text{ (radius of gyration)} = \sqrt{\frac{I}{A}}$$

<u>Specimens 1_A to 1_E</u>

Length of columns = 1200mm

Strength of concrete (from table 6) = 24.3N/mm^2

$E_c = 9.1\ (24.3)^{0.33} = 26.1$ KN/mm^2

$E_t = 19.93$ KN/mm^2

$$\therefore\ A = A_t + \frac{E_c}{E_t}\ A_c$$

$$= 1444 + \frac{25.1 \times 10^3}{19.93 \times 10^3} \times 4181$$

$$= 6919.4\ \text{mm}^2$$

$\therefore$ height of section = width of section (square section)

$$= \sqrt{6919.4}$$

$$= 83.18\text{mm}$$

Therefore, the transformed section is drawn below, all timber. (Neglecting the steel fabric see 4.1.2).

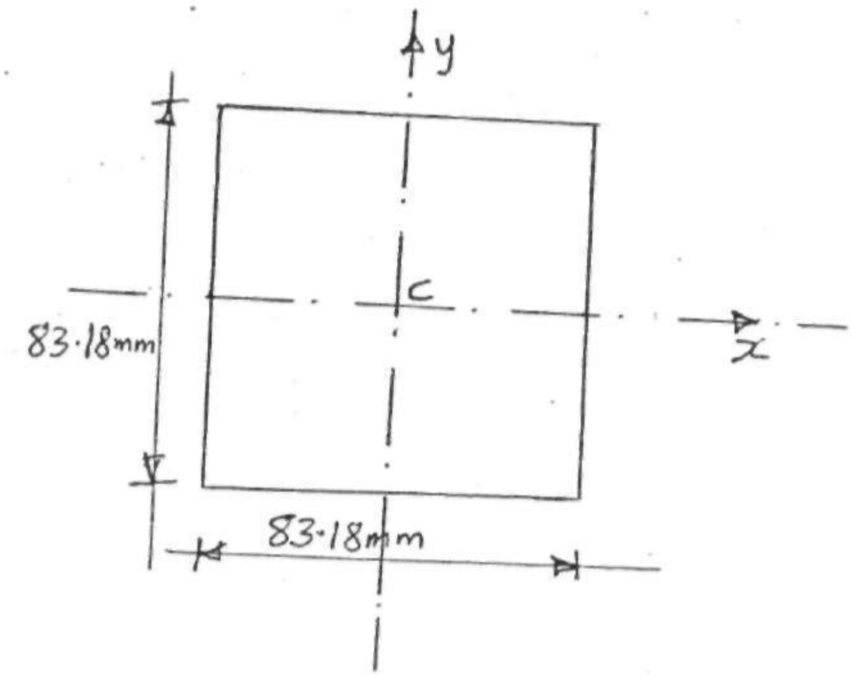

Fig 4.4 : Transformed Section, all timber

$$r = \sqrt{\frac{I}{A}}$$

$$I_{cx} = \frac{83.18 \times 83.18^3}{12} = 3.989 \times 10^6 \text{ (transformed section)}$$

$$r = \sqrt{\frac{3.989 \times 10^6}{6919.368}}$$

$$= 24.01\text{mm}$$

$$\therefore \quad P = \frac{35 \times 6919.368}{1 + \frac{1}{3000}\left(\frac{1200}{24.01}\right)^2}$$

$$= 132.1\text{KN.}$$

4.3.0 <u>The mean and other statistical values of the Column Test Results (From Table 7)</u>

<u>Specimens 1_A to 1_E</u>

Mean of results $\bar{X} = \dfrac{118.5 + 122.0 + 120.5 + 126.0 + 119.5}{5}$

$= 121.3KN$

Standard Deviation $\sigma = \sqrt{\dfrac{\Sigma(x-\bar{x})^2}{n-1}}$

$= \sqrt{\dfrac{34.3}{4}}$

$= \underline{\underline{2.93}}$

Coefficient of Variation $V_f = \dfrac{\sigma}{\bar{X}} \times 100$

$= \dfrac{2.93}{121.3} \times 100$

$= \underline{\underline{1.9\%}}$

Standard Error $= \dfrac{\sigma}{\sqrt{n}}$

$= \dfrac{2.93}{\sqrt{5}}$

$= \underline{\underline{1.31}}$

TABLE 8 : MEAN AND OTHER STATISTICAL VALUES OF THE COLUMN TEST RESULTS AND THE THEORETICAL FAILURE LOADS

Sample Set	Mean Failure Load (TEST) (KN)	Failure Load (THEORETICAL) (KN)	Standard Deviation (TEST) σ	Coefficient of Variation V_f(TEST) (%)	Standard Error (TEST)
1	121.3	132.1	2.93	1.9	1.3
2	134.0	150.0	15.87	11.9	7.9
3	163.3	172.3	16.9	10.4	8.5
4	186.6	192.1	11.8	6.3	5.3
5	200.8	216.5	32.9	16.4	16.5

4.4.0 DISCUSSION ON RESULTS

The results of the tests clearly indicate the possibility of using encased timber column as an alternative to conventional steel reinforced concrete columns, especially in low cost housing schemes. The results are however, lower than the theoretical values. This may be due to many factors, for example, the compaction of concrete in the specimen may not be the same with that of the concrete cube whose crushing value was used in the computation of the theoretical failure load; assumption of purely vertical and pin-jointed ends of the column at test may not be wholly achieved, likewise other unattainable theoretical assumptions.

The moisture content of the timber used was low (7.6% averagely), therefore, the timber absorbs water from the fresh concrete and swells, but shrinks when the concrete dries, this may lead to tensile stress being developed in the concrete and then plastic crack results. This may be the state of condition of the six specimens which were without steel fabric but cracked before testing.

4.5.0 COST ANALYSIS FOR A TYPICAL COLUMN (225mm x 225mm x 3000mm)

ITEM	DESCRIPTION	QTY	UNIT	RATE	N	K
	BILL NO. 1					
	STEEL REINFORCED CONCRETE COLUMN					
	Concrete column reinforced with 4 numbers 12mm diameter mild steel bars.					
A	Mass Concrete	0.151	CuM	650.0	98	20
B	12mm diameter mild steel bars	9.5	Kg	6.5	61	75
C	8mm diameter mild steel bars stirrups	7.8	Kg	5.6	43	68
D	PLANKING AND STRUTTING					
	Wrought formwork to sides of column	2.7	sq.M	22.0	59	40
	To Summary				263	03
	Add 25% for labour				65	76
	Grand Total				₦328	79

ITEM	DESCRIPTION	QTY	UNIT	RATE	₦	K
	<u>BILL NO. 2</u>					
	<u>TIMBER - CONCRETE COMPOSITE COLUMN</u>					
A	Mass Concrete	0.105	CuM	650.0	68	00
B	125mm x 125mm x 300mm Mahogany	3.0	Lin.M	4.9	14	70
C	Steel Fabric reinforcement size 3.0mm both ways and pitch 50mm x 50mm c/c	2.1	sq.M	14.5	30	45
D	<u>PLANKING AND STRUTTING</u>					
	Wrought formwork to sides of column	2.7	Sq.M	22.0	59	40
	To Summary				₦172	55
	Add 20% for labour				34	51
	Grand Total				₦207	06

The grand total for Bill number two (Timber-concrete composite column) was less by One hundred and twenty-one Naira seventy-five Kobo (₦121.75) (about 37%) than that of Bill number one (Steel reinforced concrete column). Therefore, considerable amount of money will be saved if timber-concrete composite columns are used for the construction of buildings.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

It can be observed that the results from the test agreed favourably with their corresponding theoretical values. The use of Euler's formula results to very high failure loads when the slenderness ratio is low. This is not surpirising, as a review from literature has indicated that Euler's formula is generally valid for high slenderness ratios. Thus the Rankine - Gordon formula was used to compute the theoretical failure loads since the specimens slenderness ratios range from 16.0 to 5.3.

In practice, the column sizes for low cost houses could be taken as 225mm x 225mm and say a height of 3000mm. Using the laws of similitude, for an example, taking specimen (2), (75mm x 75mm x 1000mm long) the scaling factor would be 3. Hence, failure load would be 402KN (i.e 134 x 3) and from experience, the total load on any such column in this type of houses is usually less than 400KN. This shows that timber columns encased with concrete could be used in place of the conventional steel reinforced concrete columns. Even if the anticipated load, in practice, is higher than 400KN, bigger sizes of timber could be chosen. The overall cost will still be less than when steel reinforced concrete columns are used.

The concrete cover will protect the timber from fire hazards, thus, eliminating some of the greatest disadvantages of the use of timber as structural members.

5.2.0 Recommendation

Timber absorbs water from the wet concrete and swells but shrinks when the specimen dries up. This swelling and shrinkage phenomenon can cause cracking of the encasing concrete, therefore, it is recommended that steel fabric reinforcement be placed within the concrete to check any cracking that may arise. From this research, it could be noticed that all the specimens without steel fabric, regardless of the method of curing cracked before the test.

It is also recommended that member joints should be properly designed and in fact, more research should be carried out on member joint connections and splices to bring out empirical design concepts.

Bond strength between concrete and timber may be improved by making grooves on the timber's surfaces. The use of nails put at 45° angles at intervals on the timber could also be adopted. The surfaces of the timber could generally be roughened before the concrete is cast.

More research should be carried out on eccentrically loaded encased timber columns (both short and slender columns).

REFERENCE

1. ASTM. Special Technical Publication No. 169.
Tests and Properties of Concrete and Concrete
Aggregates. PP 14 - 40.

2. CEB/FIP (1978). Manual of Buckling and Instability.
Longman Inc. New Your. PP 11 - 29.

3. Cook J. P. (1977). Composite Construction Methods.
John Wiley and Sons. New York. PP 128 - 134.

4. Desh H. E. (1977). Timber, Its Structure and Properties.
5th Ed. London. The Macmillan Press Limited.
PP 199 - 289.

5. Mcintosh J. P. (1964). Concrete mix design. 2nd Ed.
Cement and Concrete Association, London, England.

6. Midwest Plan Service (1978). Professional Design
Supplement to the MWPS Structures and Environmental
Handbook. 5th Ed. PP 76 - 82.

7. Mironor S. A. and Buzhevick F. A. (1960). International
Symposium on Concrete and Reinforced Concrete in
hot Countries (Israel). R. I. L. E.

8. Purslow D. F. (1974). Methods of applying wood preservatives.
Building Research Establishment Report, Garston Watford.
PP 2 - 16.

10. Ryder G. H. (1969). Strength of Materials. 3rd Ed. E. L. B. S. Pitman Publishing Limited, London PP 246 - 248.

11. Sabnis G. M. (1979). Handbook of Composite Construction Engineering. Van Nostrand Reinhold Company, New York. PP 270 - 343.

12. Wangaard F. F. (1950). The Mechanical Properties of Wood. John Wiley and Sons Inc, New York. PP 216 - 218.