A METAL CONNECTING SYSTEM

FOR

'BUSH-POLE' TRUSSES

A Thesis Presented To

The Faculty of Graduate Studies

The University of Manitoba

In Partial Fulfillment

of the Requirement for the Degree

Master of Science
in
Civil Engineering

by

Kris J. Dick, P.Eng.

December, 1988

ISBN 0-315-76659-X

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ABSTRACT

While living and working in Sierra Leone, West Africa from 1985 to 1987, the writer saw a need to investigate an alternate form of roof support mechanism that would utilize indigenous material, while at the same time improving the structural integrity of roofing systems, and creating an income-generating activity to assist in stimulating local economies. The concept of fabricating metal connectors for joining 'bush-poles' together to form standard Fink trusses was born.

Metal connectors, fabricated from 24-gauge galvanized sheet metal, were used in the manufacture of two, 24-foot span trusses. The full scale trusses were structurally tested and found to be capable of carrying a dead load approximately four times that which would be imposed on them in Sierra Leone.

Since this system was designed for use in Sierra Leone, or other developing regions, close attention was paid to the level of technology employed in its development. The testing procedures, material, and equipment used in connector fabrication were designed to be as transferable and appropriate as possible.

The metal connecting system for truss manufacture has shown itself to be a feasible method of improving the structural integrity of a roof system, reducing material requirements, and for potentially providing local income-generation.

FOREWORD

The need for the ability to fabricate the components of basic shelter through the use of indigenous construction material; the need to investigate the potential of creating a sustainable, income-generating, appropriate technology for developing regions; and personal experience living and working in Sierra Leone, West Africa for two years have combined to provide the impetus for this thesis.

This foreword provides the opportunity to thank those individuals and organizations who have supported me, both here in Canada and in Sierra Leone. To them I am most grateful.

Dr. A.M. Lansdown, Professor of Civil Engineering, University of Manitoba; for his verbal and physical assistance, along with his belief in the validity of appropriate technology.

Canadian International Development Agency(CIDA); for awarding me a scholarship to continue research in development and feed my family.

David Haslett, Field Staff Officer, CUSO Sierra Leone; for his kind ways, and his past and forthcoming support in the joint effort of trying to assist the Mende and Temne people of Sierra Leone.

Brenda Millar, West Africa Desk, CUSO Ottawa; for her support, pleasant manner and ability to ask the right questions.

Suzanne Dick, Wife, mother and proof-reader; for her love and caring.

K.J. Dick

Anola, Manitoba

Wordprocessing,
graphics, and
layout, by K.J. Dick.

TO

John H. Dick, P.Eng.

(1923 - 1987)

Father, friend, and engineer. A man with a wealth of general knowledge, common sense, and kindness. He is greatly missed.

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LIST OF ABBREVIATIONS

A	-	cross-sectional area
An	-	net cross-sectional area
ASTM	-	American Society for Testing and Materials
BWG	-	British Wire Gauge
c	-	distance from neutral axis to extreme fibre
Cc	-	slenderness ratio, compression
Ce	-	exposure factor
Cp	-	external pressure coefficient, wind
Cpi	-	internal pressure coefficient, wind
CSA	-	Canadian Standards Association
cu	-	cubic
CWC	-	Canadian Wood Council
d	-	penny, as in 2d nail
deg	-	degree
dia	-	diameter
Dwg	-	drawing
E	-	Modulus of Elasticity
F	-	force
fb	-	bending stress
fc	-	compressive stress, parallel to grain
fcp	-	compressive stress, perpendicular to grain
Fig	-	figure
Fn	-	nail resisting force

LIST OF ABBREVIATIONS

ft	-	foot
Ft	-	tensile stress
Fy	-	yield stress
ga	-	Gauge, metal thickness
hr	-	hour
I	-	Moment of Inertia
ID	-	inside diameter
in	-	inch
Ja	-	toe-nailing factor
Jb	-	nail clinching factor
Jc	-	configuration factor
Jd	-	diaphragm construction factor
Je	-	end penetration factor
Jp	-	nail penetration factor
Jy	-	side plate factor
k	-	effective length factor
kb	-	bearing factor
kc	-	slenderness modification factor
kd	-	load duration factor
kg	-	kilogram
kh	-	load sharing factor
ki	-	importance factor
kN	-	kilonewton

LIST OF ABBREVIATIONS

ki	-	importance factor
kN	-	kilonewton
kPa	-	kilopascals
kph	-	kilometers per hour
ks	-	service factor
ksi	-	thousand pounds per square inch
kt	-	treatment factor
kzt	-	size factor for tension
L	-	span
lbs	-	pounds, force or weight
Le	-	Leone, unit of Sierra Leone currency
LHO	-	Low Human Occupancy
LTI	-	Laminated Timber Institute
M	-	bending moment
m	-	metre
m.s.	-	mild steel
min	-	minute
mm	-	millimeter
MPa	-	megapascals
mph	-	miles per hour
N	-	newton (force)
Nl	-	lateral capacity per nail
n.a.	-	neutral axis

LIST OF ABBREVIATIONS

NBCC	-	National Building Code of Canada
nf	-	nails per group
No.	-	number
OD	-	outside diameter
P	-	point or concentrated load
Pcr	-	Critical compressive force, Euler
psf	-	pounds per square foot
psi	-	pounds per square inch
q	-	reference wind velocity pressure
Qe	-	external pressure, wind
Qi	-	internal pressure, wind
rdg	-	reading
S	-	section modulus
Sb	-	balanced snow load
Sg	-	ground snow load
sin	-	sine function
Su	-	unbalanced snow load
TDM	-	Timber Design Manual, LTI
TPIC	-	Truss Plate Institute of Canada
w	-	load per unit length
Xb	-	balanced snow load factor
Xe	-	exposure factor
Xu	-	unbalanced snow load factor

LIST OF ABBREVIATIONS

"	-	inches
'	-	feet
^	-	Indicates a number raised to a power, eg. in^3
<>	-	deflection
%	-	percent
#	-	pound, force or load
@	-	stress, ultimate or allowable
π	-	PI, constant = 3.141592654

ONE

1.0 INTRODUCTION:

Exploring new methods for the use of indigenous construction material in developing regions of the world is an important contribution that the engineer can make to promote strengthening of a sustainable local, regional, or national infrastructure.

This thesis examines the use of a connecting system whereby 'bush poles' can be used in the manufacture of pre-fabricated trusses in remote or developing regions. These trusses will be used for house and school construction in Sierra Leone, West Africa.

In Chapter Two, the concept, materials used, and manufacturing process of the connecting system is discussed. The type of timber available for applications in Sierra Leone, as well as what was used for testing purposes here in Manitoba is dealt with in Chapter Three. Chapter Four discusses the structural testing of the connecting system and provides the basis for discussion in subsequent chapters.

In order to present information succintly in the main body of the thesis, details of calculations, testing procedures, and drawings, have been assembled in various appendices. Given that the technical working language in Sierra Leone uses English units, and that this document will be used in the field in Sierra Leone, it was felt that all parties concerned would be best served if a convention of English units was observed.

Two

2.0 ROOF FRAMING AND THE CONNECTING SYSTEM:

2.1 GENERAL OVERVIEW:

While the writer was working as a construction supervisor and technical instructor in Sierra Leone, 1985-1987, there appeared a demonstrated need to improve the structural integrity of roofing systems in a variety of buildings. As one travelled from village to village one saw that the basic style of the roofs was similar but quality varied a great deal. This could be partially attributed to some local carpenters injecting their own anomalies into these structures, or to uncertainty of roof strength. Unnecessary columns in the centre of classrooms, extraneous roof bracing in questionable locations, excessive nailing causing poles to split, and inconsistency in construction were some of the problems that arose.

The notion of using pre-fabricated trusses as an alternative came quickly to mind as a means of establishing consistent structural integrity while making the roof construction process easier and quicker. The problem with this idea, however, was to devise a design that would incorporate the use of indigenous material into the manufacture of these trusses. If the design were to have a chance at being acceptable, maximizing the input of locally-available resources was essential, as this would reduce dependence on imported commodities and expertise, and ultimately, create a sustainable technology.

Currently, roof framing is accomplished through the use of round timber 'bush poles', that are cut from the jungle and either lashed or nailed together to form a system. Onto this roof framework is affixed galvanized corrugated steel sheeting(called C.I. sheet in Sierra Leone), or grass thatch.

The connection of the members in a truss is critical in the manufacture and design, which in this case, was complicated by the use of round timber. It was felt, that there were two main concerns:

1. to develop a method of connecting round timber members that would provide adequate resistance to the forces created in the truss; and, more importantly,
2. to develop a connecting system that would utilize locally available material.

2.2 SYSTEM DESCRIPTION:

The use of round timber presents constraints that are not present in timber trusses manufactured from dimensional lumber with a rectangular cross section. It is customary to join dimensional timber trusses with either wood gussets or steel plates. This system works well on material with a rectangular cross-section, since these plates are flat, but the amount of contact area between flat plates and round cross-sections is greatly reduced. This reduction in contact with the truss member results in a corresponding decrease in the force resisting capabilities of the connector due to the limited space available for nailing. A comparison of the application of a gusset to the two cross sections is illustrated in Fig. 2-1. This obvious disparity in contact area necessitated an alternative solution for connection.

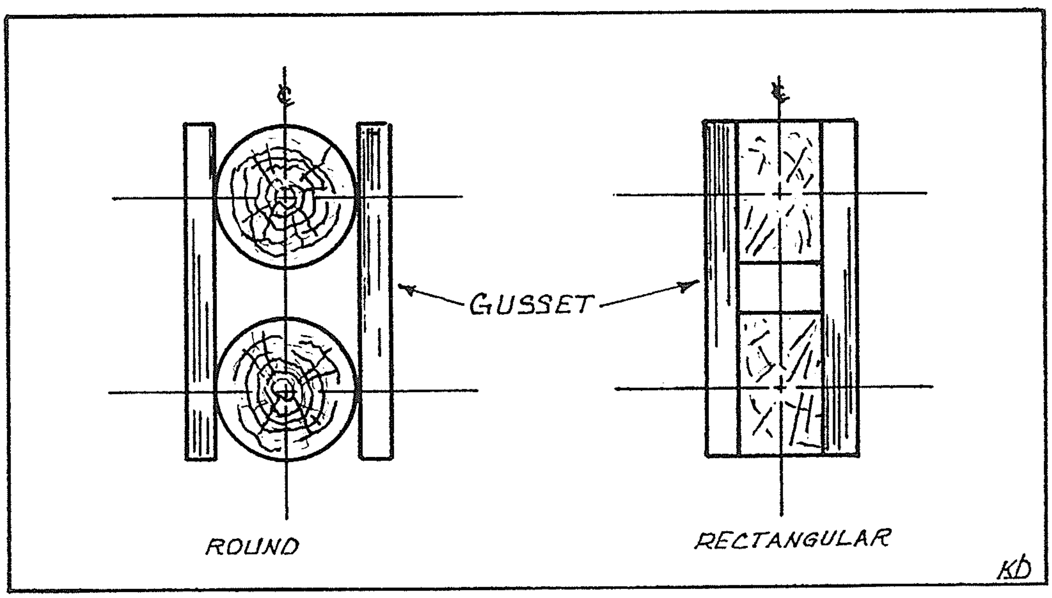


Fig. 2-1: Gusset Application - Round and Rectangular

To create enough surface area for adequate nailing capability, the connector had to conform to the circular shape of the truss members. After consideration of a variety of possible approaches, the direction the design process assumed was to create a connector that could be fabricated in two halves. If fabricated in this way application of the connector to one side of the truss would provide for proper alignment of the truss members. The truss could then be turned over and the other half of the connectors applied, in a manner similar to conventional gusset plates on dimensional timber.

Throughout this conceptual process one factor had to be kept in sight at all times: -- that whatever the design was to be it <u>must</u> be at a level of technology that would be applicable in Sierra Leone. This implied that the parameters of local technical understanding, the manufacturing process, and the fabricating equipment must be balanced appropriately.

The configuration that was decided upon for the connecting system is illustrated in figures 2-2 through 2-6. (Detailed design drawings are contained in Appendix 'E'.)

In terms of appropriateness, the type of connector material that was to be used tended to pose a more serious concern than the method. After considering such alternatives as bamboo, rope lashing, and sheet metal, the last option was chosen. It was felt that a standardised system could be designed more easily and could produce a method that would better ensure consistent structural integrity with the use of metal connectors. In addition, the potential to create localised income generation through the establishment of small local enterprises that would manufacture and market the system was appealing. Also, in many developing regions sheet metal is available as salvage from derelict motor vehicles and some containers, thus providing an appropriate source of material. In addition to a salvage source, new sheet metal material may be a viable option in some areas. It is foreseen that the use of new material may be the only option in certain situations.

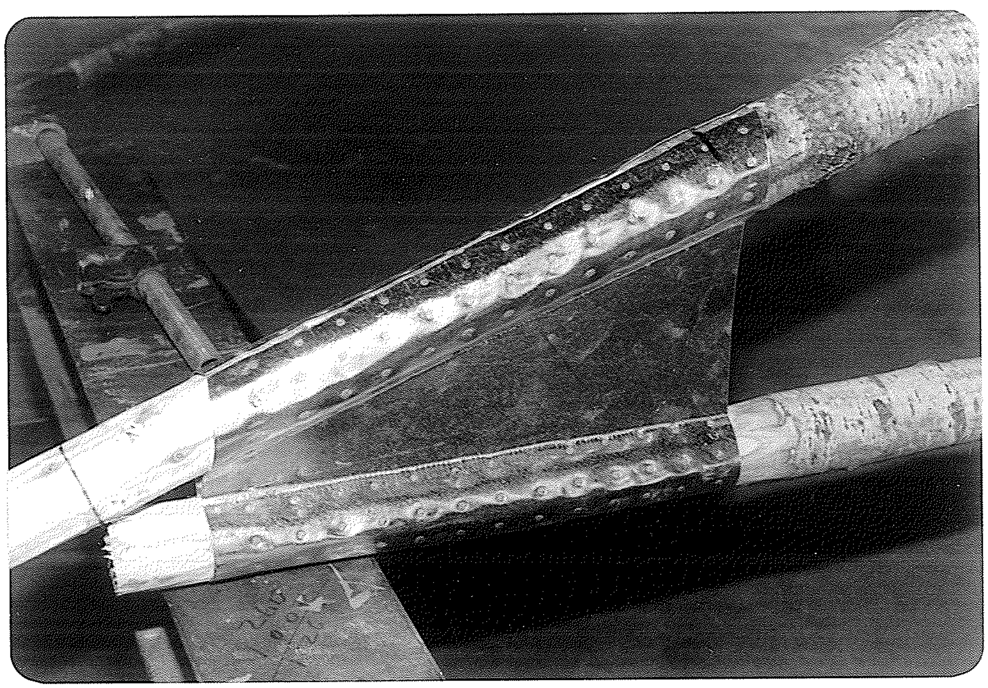


Fig. 2-2: Heel Connector

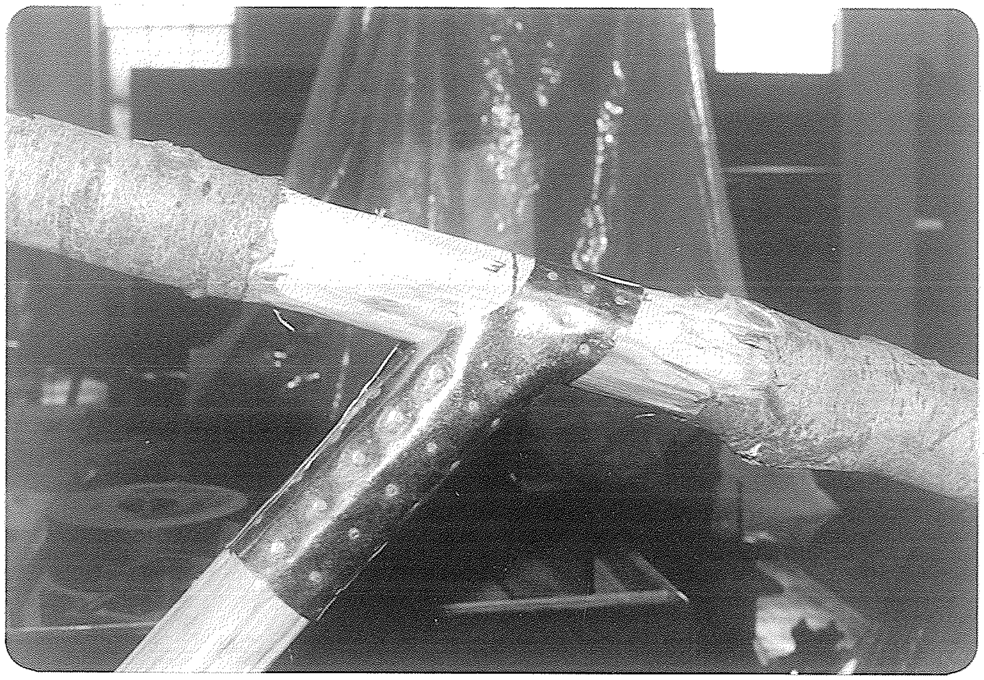


Fig. 2-3: Top Chord / Web Member Connector

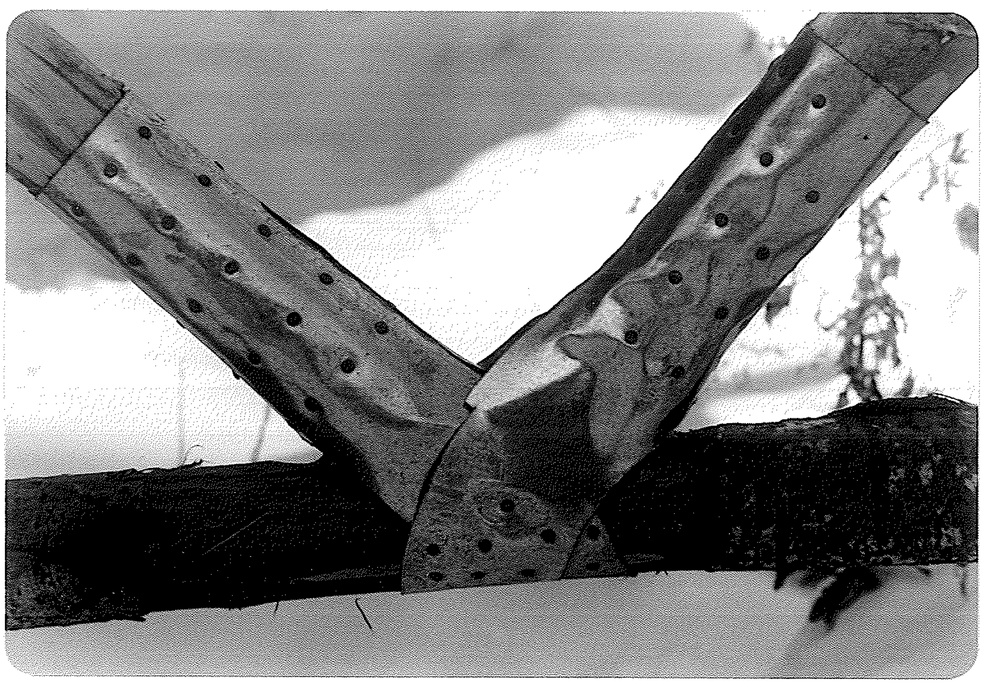


Fig. 2-4: Bottom Chord / Web Members Connectors

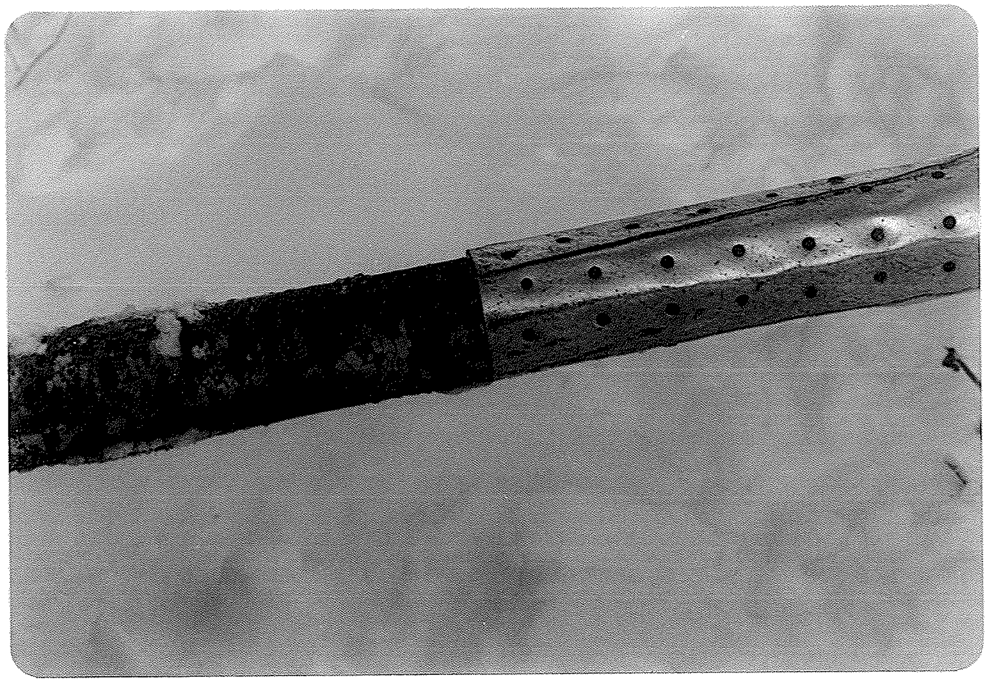


Fig. 2-5: Bottom Chord Centre Connector

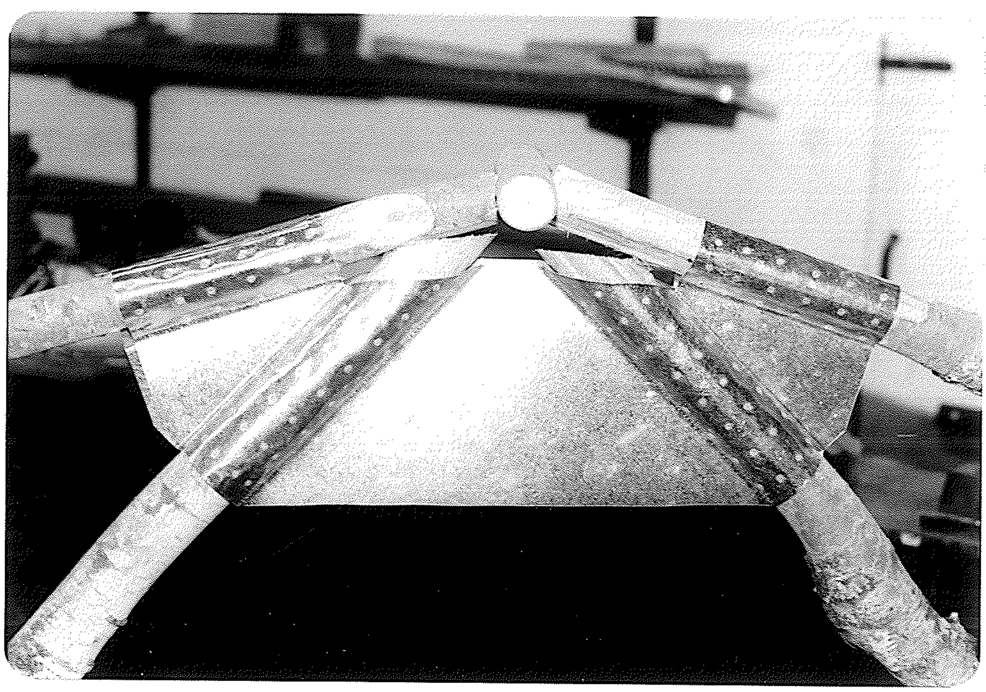


Fig. 2-6: Peak Connector

2.3 CONNECTOR MATERIAL:

The material selected for the manufacture of the connecting system may vary according to some of the following:

- availability of salvage sheet metal;
- availability of new sheet metal;
- cost of salvage and new material;
- workability of salvage metal, of various gauge sizes;
- dimensions of salvage metal;
- state of deterioration of salvage metal; and
- type of equipment available.

For the puposes of testing the viability of the connecting system, new sheet material was used. The primary objective of the testing was to determine if the connecting system could sustain the type of loading that would be encountered in Sierra Leone. If the connectors made from new sheet metal could withstand the imposed loading, the system would be considered a success. The rationale behind this statement is based on the fact that the test connectors were made from a material possessing strength characteristics inferior to that of automotive sheet metal. The engineering properties of new and reclaimed material are discussed below.

2.3.1 NEW SHEET METAL:

The material that was used in the manufacture of the test connecting system complied with the Standard A36 of American Society for Testing and Materials (ASTM). This sheet steel also meets or exceeds provisions of the Canadian Standards Association (CSA) Standard CAN3-086-M84 , Clause 10.7.1.3, which presents guidelines regarding metal used for gusset plates in roof truss manufacture. The properties of the sheet metal used are listed in Table 2-1.

Gauge	:	24	(0.0239 in.)
Yield point	:	36 ksi	(250 MPa)
Tensile strength	:	58 ksi	(400 MPa)
Elongation in 8 in.	:	20 %	
Elongation in 2 in.	:	23 %	

Table 2-1: New Sheet Metal Properties

2.3.2 RECLAIMED SHEET METAL:

The main source of recycled material to be used in the manufacture of the connecting system in developing regions would be autobody panels. Although there are many varieties of automobiles from which this sheet metal may be salvaged, minimum standards are set out by such organisations as American Society for Metals (ASM). These maximum expected standards (after ASM, 1976, pp.135, 145-179, 181-188) are listed in Table 2-2.

Yield point	: 38 ksi	(260 MPa)
Tensile strength	: 52 ksi	(358 MPa)
Elongation in 2 in.	: 30 - 41%	

Table 2-2: Reclaimed Sheet Metal Properties

Although the tensile strength is slightly less than the new sheet metal the elongation is much greater, which implies a material with increased workability making it less prone to cracking during forming. (See Chapter Seven for further comment on reclaimed sheet metal material.)

2.4 CONNECTOR MATERIAL SOURCE - DISCUSSION:

Each developing region or country presents a unique set of constraints which may impact upon connector material availability, source, cost, and type. There exists the very distinct possibility that, in a particular region, salvage material may not be in sufficient enough supply to furnish the amounts required for ongoing manufacture of the connecting system. This may be due all or in part to the following factors:

- competition for the scrap steel precluding its use for truss connectors;
- the amount of scrap reaching the market being limited or being controlled in certain directions;
- the original form of the potential scrap material being maintained longer. For example, vehicles kept running long past standards elsewhere; and
- vehicles or other sources being none-existent.

This list points to only a few possible factors and, again, may vary from place to place depending upon the level of development encountered. A realistic stance regarding the issue of connector material, sustainability and appropriateness is essential.

The ability of a village inhabitant to purchase a connecting system to fabricate trusses for his home may not exist in all cases. It could be argued further that this system is, in fact, totally impractical. If additional parameters are injected into this premise, however, the situation changes somewhat. The financial ability of many people in developing regions is below a level where they can even consider a metal connecting system for roof trusses. Their finances are directed to meeting the basic needs of daily living, and are a function of the level of development in their own region or country. In many instances, development projects or agencies subsidize the income-earning capacity and financial standing of members in a community. This is accomplished either directly, through salaries, or indirectly, by providing such items as vehicles and subsidized housing.

It must be recognised that the level of development required to enable this technology to be sustainable does not exist at the present time in many locations. In such cases the initial availability of the system could be aided through subsidization from various development projects already involved in financially assisted building construction programmes. It is also envisioned that the manufacturing process could be initiated under the auspices of these development programmes, and maintained until such time as local sustainability was reached.

It must be stressed, however, that the cost of the connecting system is low. The cost of the connectors is related to the cost of the corrugated roofing (C.I. sheet) for an average house in Sierra Leone in Table 2-3. It can be seen from the table that the connectors represents less than 7% of the roofing costs, and, in terms of the whole structure this percentage would be further reduced.

	CONNECTORS	CORRUGATED ROOF SHEETS
UNIT COST (Le)(1)	300 / set	8000 / bundle
SETS REQ'D (2)	10	--
BUNDLES REQ'D (3)	--	6
TOTAL COST(Le)	3,000	48,000
RATIO OF CONNECTOR TO SHEETING COST =	6.25%	

Note: 1. $1 CDN = 30 Leones, all costs estimated
2. Building size - 24'x 36', 10 trusses
3. 1 bundle = 20 leaves, 6 ft^2/leaf coverage

Table 2-3: Comparison of Connector and Sheeting Costs

2.5 MANUFACTURING PROCESS:

The fabrication of the metal connectors was accomplished with an hydraulic bench press and a male/female die which formed the various shapes. The main objective in developing the equipment for the manufacturing process was to keep the technology simple and easily tranferable. To that end, the press frame and forming jigs were constructed from materials commonly available. All of the components used in the manufacturing process were fabricated by the writer in his own shop. It was felt that if the press frame, forming jigs and other incidentals could be built in a small personal facility, then they could be replicated in Sierra Leone. Also, the design of these components was not made restrictive, as it is envisaged that the press setup and forming jigs will vary depending upon the material availability in each location. This variation is not a major concern as long as the basic principles of the manufacturing process are followed, the function of the fabricating components is maintained, and the material used to construct the press and jigs is of strength equal to or greater than that used in the prototype.

Fig. 2-7 shows the general arrangement of the hydraulic press with a forming jig in place.

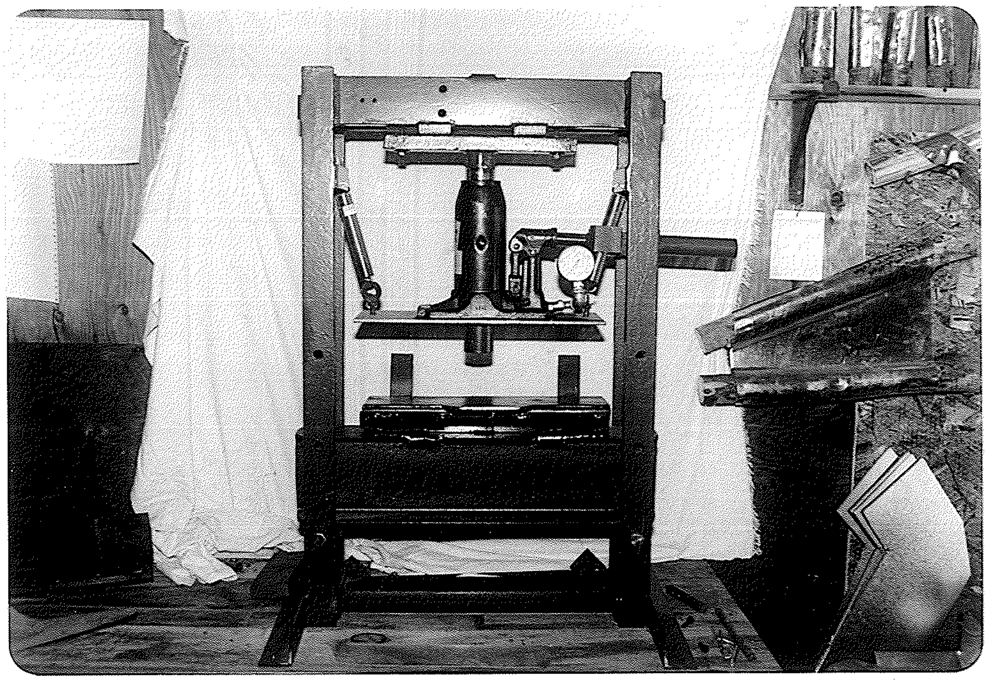


Fig. 2-7: General Arrangement of Press and Jig

2.5.1 HYDRAULIC PRESS:

The press frame was sized to sit on a bench for use and was of all-welded construction. An eight-ton, hand-operated hydraulic bottle jack was fixed to a platen which traveled up and down as the jack was activated. Fig. 2-8 illustrates the arrangement of the jack and frame, while detailed drawings of the press frame are contained in Appendix 'H'. One aspect of the frame that affects the forming of the connectors is the spacing of the side rails. The minimum distance between each set of side rails must exceed the width of the jig base, as shown in Fig. 2-9 below.

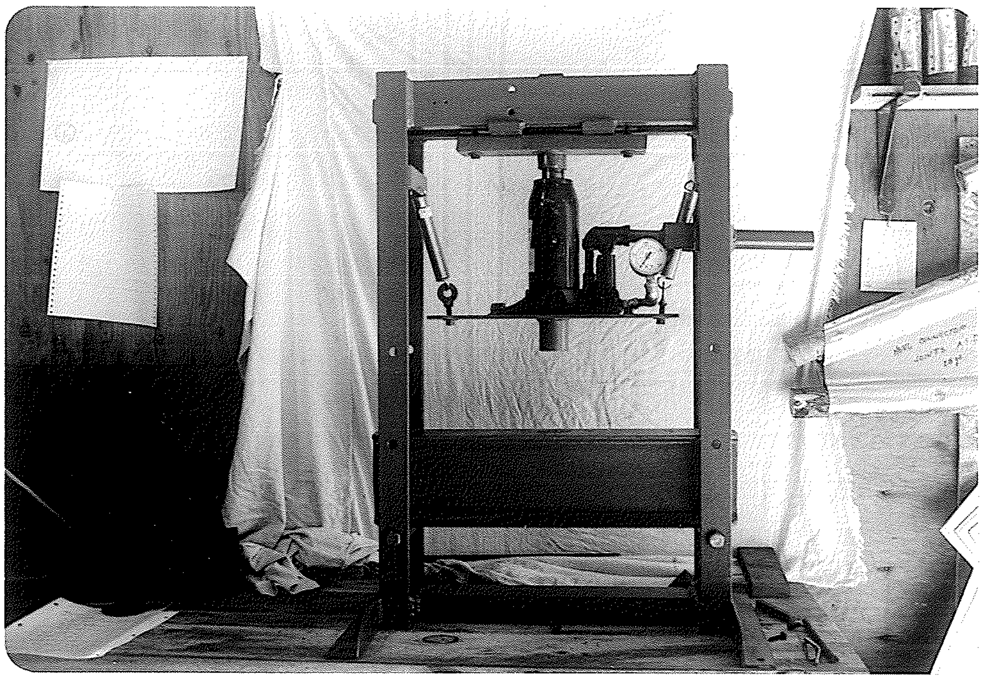


Fig. 2-8: Press Frame and Hydraulic Jack Arrangement

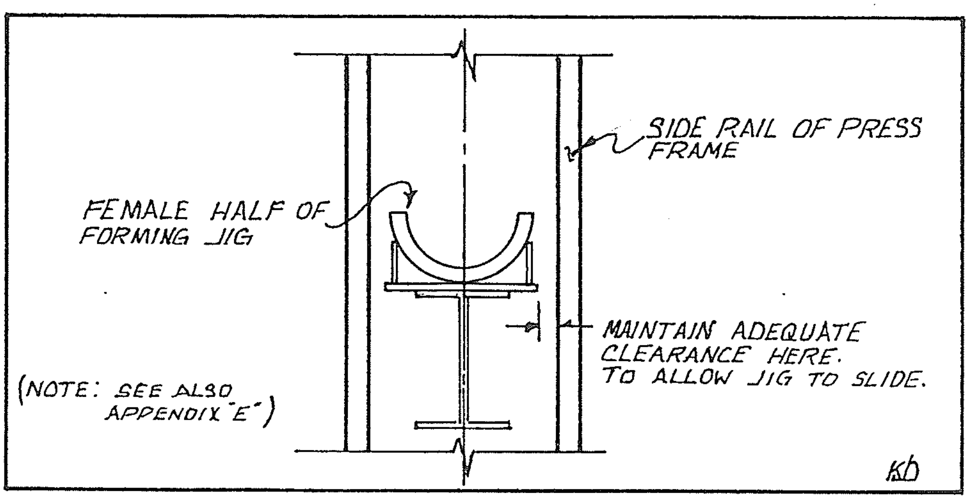


Fig. 2-9: Side Rail/Jig Base Relationship

As mentioned previously, the type of material and equipment available in developing regions is likely to vary from place to place. For this reason the fabrication of the press frame need not be restricted to a welded structure, as bolts or pins could be used equally effectively. It is envisaged that a good source of material for the press frame could come from the same derelict vehicles as the sheet metal for the connectors. The formed channel frame of a car or lorry would provide excellent material for the side rails, top beam and lower beam of the press frame.

2.5.2 FORMING JIGS:

The forming jigs were fabricated from standard-wall black iron pipe, cut lengthways in half and welded to a 1/4" (6 mm) mild steel plate. Cutting of the pipe was done with an oxy-acetylene cutting torch and attachment to the plate was done with electric arc welding. In the absence of a cutting torch the pipe could be cut with a hacksaw. It appears that welding to the base plate is the best means of attachment. Various connector sizes can be produced depending upon the pipe available. Table 2-4 relates pipe size to connector size, and detailed drawings of the forming jigs are contained in Appendix 'I'.

CONNECTOR SIZE (NOMINAL) (in)	PIPE SIZE		WALL THICKNESS	
	MALE DIE(1) (in)	FEMALE DIE(2) (in)	MALE DIE (in)	FEMALE DIE (in)
2	2.0 (2.375)	2.5 (2.469)	0.154	0.203
2 1/2	2.5 (2.875)	3.0 (3.068)	0.203	0.216
3	3.0 (3.500)	3.5 (3.548)	0.216	0.226
3 1/2	3.5 (4.000)	4.0 (4.026)	0.226	0.237

Notes: 1. Value in ( ) is actual outside diameter, OD, of the nominal pipe size.
2. Value in ( ) is actual inside diameter, ID, of the nominal pipe size.

Table 2-4: Connector Size Related to Pipe Size

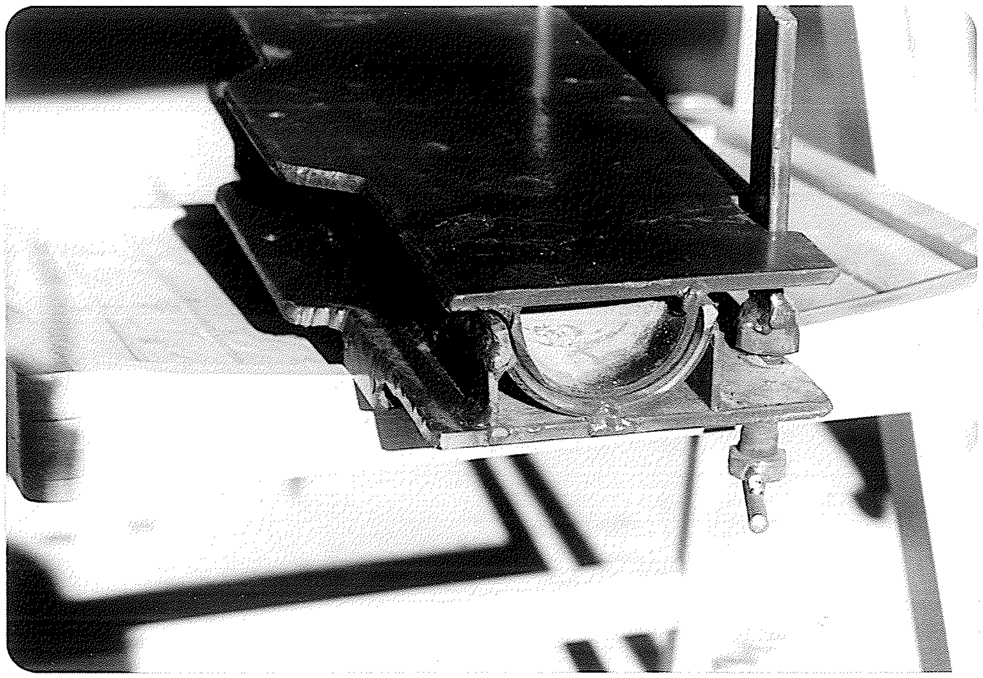


Fig. 2-10: Forming Jig

THREE

3.0 THE TIMBER TRUSS:

3.1 INTRODUCTION:

At the present time in Sierra Leone a variation of a King post truss is used to produce roof framing systems. Most often, rafters attached to a ridge pole with a vertical strut joining the peak to the bottom chord at the centre of span are combined with non-consistent bracing. In Fig. 3-1 this arrangement is illustrated. As can be seen from the

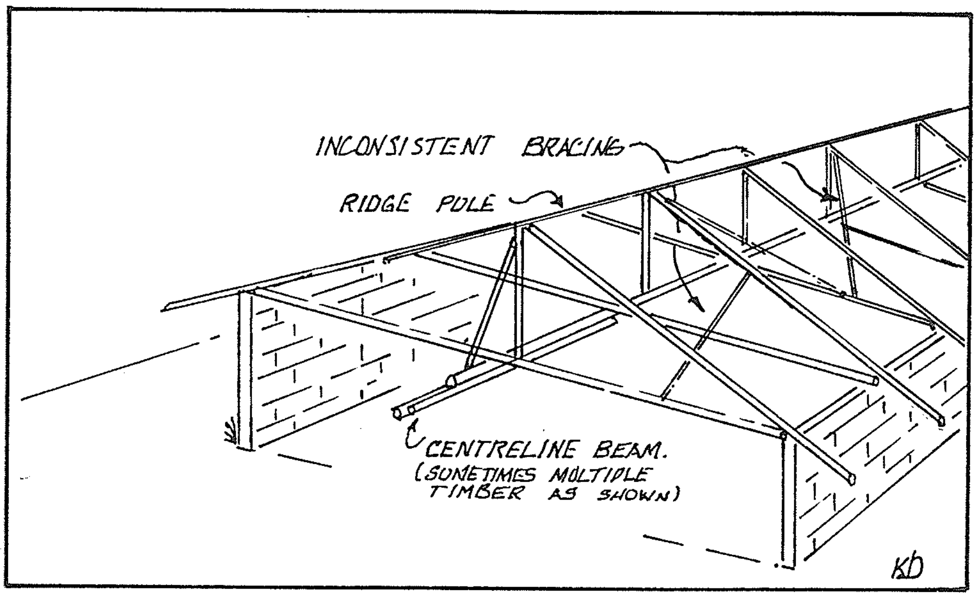


Fig. 3-1: Roof Frame Arrangement - Sierra Leone

drawing, the bottom chord is not a clear span. Additional timber is used to provide a beam which runs along the centreline of the building and sits on the dividing walls of the house or school. With ever-increasing concern for resource depletion the use of centreline beams can be eliminated through the employment of trusses, thereby reducing extraneous material. While some bracing would still be required on the bottom chord the support pole diameter would be much smaller than that of the centreline beams.

3.2 TRUSS CONFIGURATION:

The type of truss constructed for structural testing and proposed for use in the field was that of a Fink or 'W' truss. A standard slope of 4 units rise in 12 units run was used. Fig. 3-2 shows the basic dimensions of the chosen

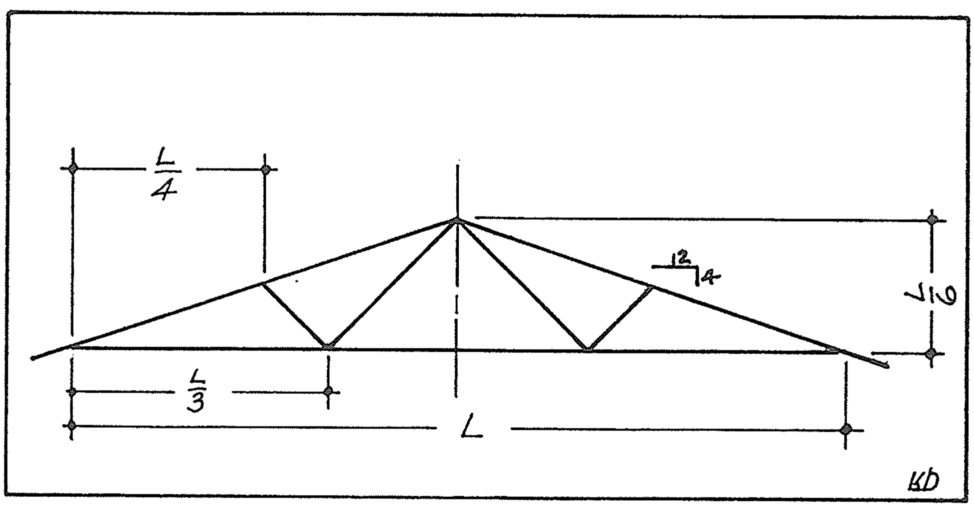


Fig. 3-2: Truss Member Length Relationships

truss, while Appendix `E' presents details of the trusses that were tested.

The span used for the basis of calculation and testing was 24'-0" (7310 mm). In the field, however, use of the connecting system for spans ranging from 20'-0" (6100 mm) to 30'-0" (9140 mm) is anticipated. The diameter of the poles could be increased to provide the larger cross-section required. There will, however, be no need to alter the geometric configuration of the metal connectors as the slope of the trusses will remain the same. The potential changes in the connectors for spans greater than 24 feet would be:

- accomodate the larger pole diameter; and
- provide a heavier gauge connector material.

3.3 MATERIAL:

The timber that would be used for the manufacture of the trusses in Sierra Leone is known as `Kande'. Appendix `A' discusses some the properties of the Kande tree as determined by the writer from tests done while still in Sierra Leone. The overall quality of the timber is superior to the Northern Aspen that was used in the structural testing described in this thesis. A few observed characteristics of the Kande tree are:

- it can withstand extreme flexure before cracking appears;

- it has a density comparable to that of spruce;
- it appears to take a nail without splitting; and
- it does not appear to be affected signifigantly by termites.

The timber used in the construction of the test trusses was of the Northern Aspen (poplar) species, [letter designation -`F', grade stamp -`N. Aspen'. (after LSD of Wood Structures, 1986, pp. 47-48)]. In general, N. Aspen is a light wood with moderate strength. Poplar poles were used because they represented a timber that was not inherently as strong as the Kande tree. This is not to say, however, that poplar is an inferior material in general. The material for the trusses brought with it some inherent weaknesses which put the structure at an initial disadvantage. The reason for this was twofold: first, poplar trees tend to grow the way they want which is not always the best for truss manufacture: - often this provides kinks or bends in the raw poles; but secondly, and more importantly, the wood in the poplar poles tends to be of decidedly lower quality than that of a Kande tree. Thus, it was felt that, if the poplar pole truss would support the imposed loading required then it could be assured that trusses manufactured from the Kande tree would also handle the same loading.

The characteristics of the poplar poles used in the truss fabrication which had a negative impact on the performance of the structure are summarized as follows:

- Poplar has a less-consistent timber quality than one would normally expect, and tends to go rotten or 'punky' in the centre (heartwood);
- It is hard to determine or predict the integrity of the timber from the outside, (although this could be said of many species.);
- Poplar does not grow straight, therefore 'built-in' eccentricities were unavoidable. The test truss was, therefore, potentially predisposed to bending or strength reduction in these locations;
- The timber was relatively green, implying a moisture content greater than 19%. A time span of only two weeks elapsed between harvesting of poles from the bush and structural testing. The effect that moisture has on wood can be seen in Fig. 3-3.

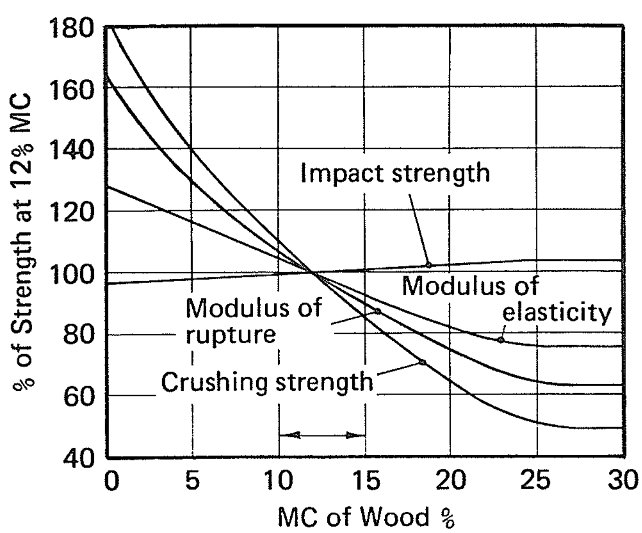


Fig. 3-3: Effect of Moisture on Strength Properties
(source: CWC Datafile SP-1, 1987, p.5)

FOUR

4.0 TRUSS ANALYSIS AND TESTING:

4.1 INTRODUCTION:

Although the load carrying capacity of the members of a truss can be predicted theoretically, when considering the structure as a whole it is imperative to actually test that truss and observe its performance under loading. In this chapter the structural testing carried out on the poplar pole trusses is discussed. Due to the nature of the information required many explanatory calculations need to be presented. In order to keep the discussion relatively uncluttered these calculations appear in seperate appendices with reference made to the specific appendix within this chapter appropriate. Other calculations are included within the discussion.

4.2 BASIS OF ANALYSIS AND DESIGN:

The analysis and design of the pole truss was based on standardized criteria set out by the following organisations and publications:

- Canadian Standards Association, (CSA);
- National Building Code of Canada, (NBCC);
- Truss Plate Institute of Canada, (TPIC);
- Canada Wood Council, (CWC); and
- Laminated Timber Institute of Canada, (LTIC).

In addition to the above references, conventional engineering structural mechanics was used in analysing the various theoretical components and requirements of the truss.

Assumptions or conventions are invariably adopted at the beginning of a theoretical analysis to assist the engineer in the design process. Throughout the design of the pole trusses any assumptions made for this reason were always done in such a way as to error on the conservative side, that is, on the side of safety.

4.2.1 ANALOGUE:

Two conventional analysis techniques were employed in the determination of the forces acting within the truss members: first, that the truss was pin-connected at the joints, and secondly, that simple analogue lines were used to represent the pole truss members.

To eliminate the need to analyse an indeterminate structure, all of the joints in the truss were considered pin connected and simply supported. This allowed the calculation of member forces to be done using the Method of Joints, which assumes that two conditions exist; (Grinter, 1955):

1. there is a frictionless pin at every joint; and
2. there is no bending in any member of the truss.

The fact that the metal connectors provide semi-rigid joints in the pole truss is ignored when using this convention. It is known that the use of a pin connected truss approach yields a conservative result, as this joint rigidity provides some load-sharing to occur within the pole/connector combination which, can increase the capacity of the truss.

The axial loads within the truss members act over the entire cross section of the pole. The use of analogue lines provides a way to satisfy another fundamental assumption used in the analysis of trusses; (Grinter, 1955; and, TPIC, 1988):

3. the gravity lines of all truss members meeting at joint must intersect at a single point, to prevent harmful eccentricities.

The analogue lines and corresponding lengths used in the pole truss design are illustrated in Fig. 4-1.

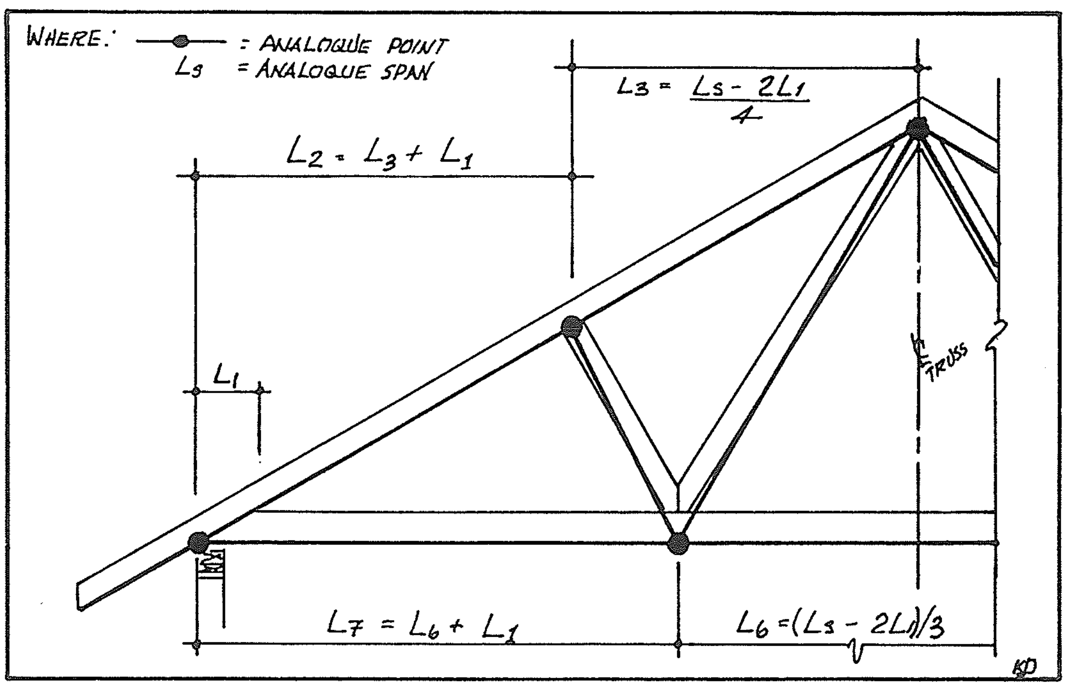


Fig. 4-1: Truss Analogue Points and Lengths

4.3 LOADING REQUIREMENTS FOR SIERRA LEONE:

The metal connecting system will be used within Sierra Leone for the fabrication of bush-pole trusses which will be used in the construction of schools. The roof structure will be subjected to the following imposed loading in Sierra Leone:

- roofing material, corrugated sheeting;
- timber truss members;
- timber roof purlins; and
- wind loading.

From calculations and details contained in Appendix 'L', Table 4-1 presents a summary of loading requirements for Sierra Leone.

LOAD TYPE	LOAD / TRUSS (lbs) (1)
Roofing Material	141
Timber Truss Members	39
Timber Roof Purlins	16
Wind	(-) 1010
Total Load w/o Wind	195
Total Load with Wind	(-) 815

Note: 1. Based on 4'-0" centre to centre spacing.
2. Wind loading results in a net suction effect.

Table 4-1: Loading Requirements - Sierra Leone

4.4 TEST TRUSS DETAILS:

Two poplar pole trusses, each with a span of 24'-0", were fabricated by the writer for purposes of testing of the metal connecting system. Details of the testing procedure, component design and test results are presented in this chapter.

4.4.1 TRUSS LOADING:

Within the constructed roof system the truss will be subjected to various types of loading. The effect of the imposed load can be examined theoretically, but, it is quite difficult to replicate precisely the practical situations in a laboratory environment without costly or elaborate equipment. For this reason, a simple means of loading the truss which simulated the effect caused by a uniformly distributed load had to be chosen. It was decided to use a series of point loads to achieve this objective. (after CSA CAN3-S307-M1980, Section 6.) The rational for this choice was based on two requirements:

1. a procedure that could be used in Sierra Leone; and
2. a technique that provided a top chord bending moment that was the same or greater than caused by a uniformly distributed load.

The first point required that a simple loading arrangement be used, while the second required an examination of the bending moment created by uniform and point loads across the span. In Fig. 4-2 the location of the point loads is shown for the top chord.

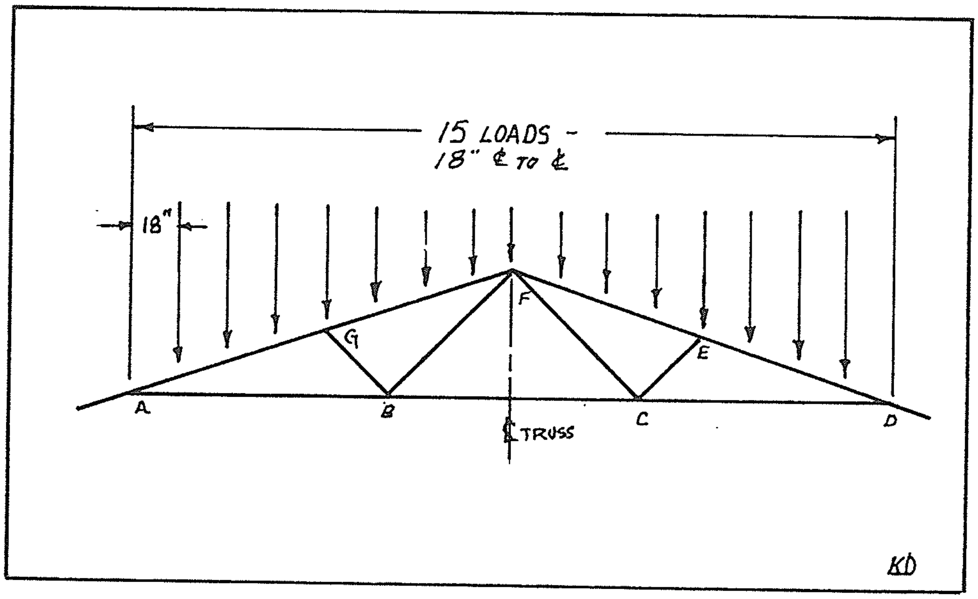


Fig. 4-2: Arrangement of Point Loads on Test Truss

To facilitate calculation of the bending moment the top chord was considered to be a two-span continuous beam with a rigid central support. The end conditions of the beam were assumed to be pinned. It was felt that using pinned connections at the ends and an unsinking central support would yield a result for the bending moment on the side of safety.

The bending moment at the central support created by the point loads was determined using the Three-Moment Theorem for concentrated loads, (see Appendix 'C'). In Table 4-2 this moment is compared to the moment at the same point due to the load spread uniformly on the top chord.

M2C (concentrated)	-	5/32 wL^2
M2U (uniform)	-	4/32 wL^2
M2C - M2U	-	1/32 wL^2
% M2C > M2U	-	20 %

Table 4-2: Comparison of Bending Moments

Based on the information contained in Table 4-2, the bending moment created by the approximately equivalent point loading is in excess of that produced from a uniform loading condition, in keeping with comments made in section 4.2 regarding conservative conventions and assumptions. Appendix 'C' contains detailed calculations of loading and member forces relating to the test truss loading conditions.

4.4.2 TIMBER SIZING:

The diameter of the timber used for the test trusses will be related to the size of the pipe used in the manufacture of the forming jigs. Sizes of 2.5" (64 mm) and 3.25" (83 mm) diameter connectors were prepared for the fabrication of the trusses. Once the member forces were calculated, the timber sizes which satisfied the allowable stress requirements was selected for use in the truss.

Appendices 'B' and 'C' discuss and illustrate the procedures used in determining the allowable stresses and the corresponding truss member cross sections.

4.4.3 CONNECTOR DESIGN AND SIZING:

The main factor that influenced the final design of the metal connectors was the provision of enough surface area to ensure adequate nailing at each joint to resist the member forces. In order to provide this the connector material had to conform to the cross-section of the timber to provide enough resistance. Thus, the idea for a jig to form the half round shape was born. In addition to providing the required surface area for nailing, it was felt a connector that would fit the shape of the timber would provide greater joint stiffness due to its curved nature as compared to flat gussets. (see Fig. 2-1, p.2.3.)

Having decided upon the basic form the connector should take, it was originally hoped that one standardized shape could be produced to handle the connection for all of the joints. To provide adequate strength and stability it became increasingly apparent a single style of connector could not meet this requirement. The present connecting system, therefore, has four basic shapes for the eight joints. It should be pointed out, however, that the tension connection at the centre of the bottom chord is simply a sleeve and constitutes only minimal forming.

Appendix 'D' contains a detailed discussion of the procedure used in determining the connector size for each joint, while Appendix 'E' contains the design drawings for the various connector shapes.

4.4.4 NAILING SCHEDULE:

The number of nails required for each joint/connector combination is dependent upon two factors:

- the unit lateral resistance per nail; and
- the force in the truss member.

The value used for the unit lateral resistance per 1" nail was taken to be - 30 lbs per nail, (132 N per nail). (after CAN3-086-M84 , Engineering Design in Wood). Once the number of nails per joint was calculated, the spacing of the nails on the connector was done in accordance with standards laid out in CAN3-086-M84. Appendix 'D' discusses in detail the procedure used in determining the nailing schedule for the test trusses.

4.5 TEST PROCEDURE AND SET-UP:

The testing of the poplar pole trusses was based on the CSA Standard - 'S307-M1980 , Load Test Procedure for Wood Roof Trusses for Houses and Small Buildings, April 1980'.

The two 24' span trusses were tested individually to determine their load carrying capacity. Fig. 4-3 illustrates the general arrangement used for structural testing. The ends of the truss were supported on a steel trestle with wooden blocks clamped on either side to prevent lateral slippage (see Fig. 4-4). The amount of deflection in the trestle was assumed to be negligible, but was checked by calculation. The support deflection for a 2000-pound truss load (1000-pounds at the centre of each steel trestle) was computed to be 0.0011 inch. The deflection of the trestles was, therefore, considered negligible. (see Appendix 'C' for calculations.)

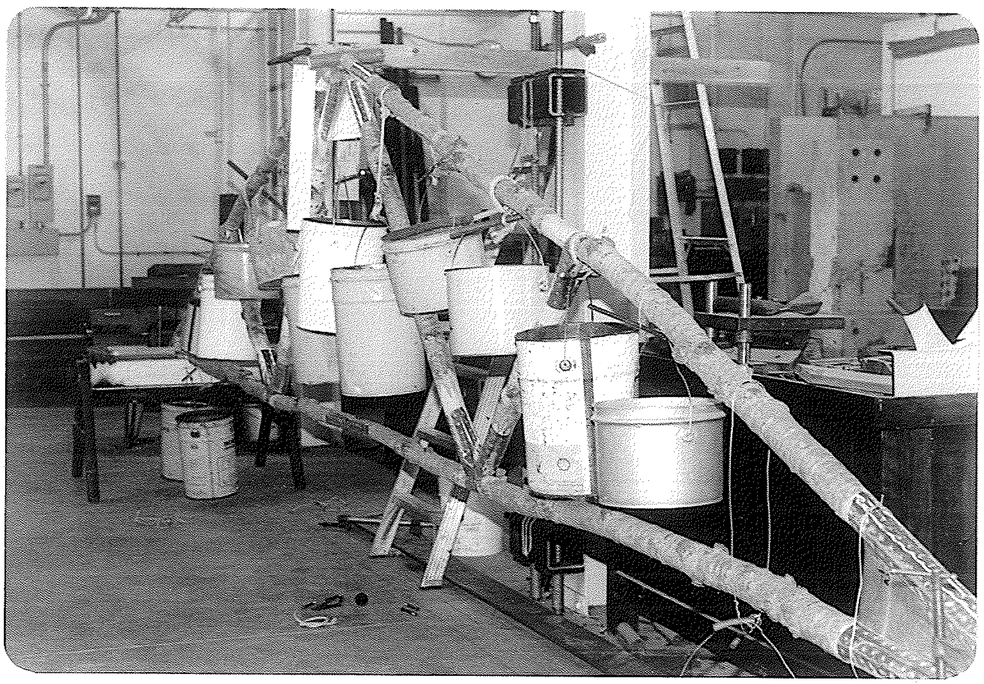


Fig.4-3: General Arrangement for Structural Testing

The deflection was measured in six locations using a taut wire below the bottom chord, attached to the steel trestles approximtely 10 inches away from the truss, reducing the effect of the relative deflection due to the trestle deformation below the already negligible 0.0011 inch. These locations were:

- midspan of bottom chord (Joint H);
- each panel point on bottom chord (Joints B and C);
- midspan from end support to panel points; and
- at the peak of the truss.

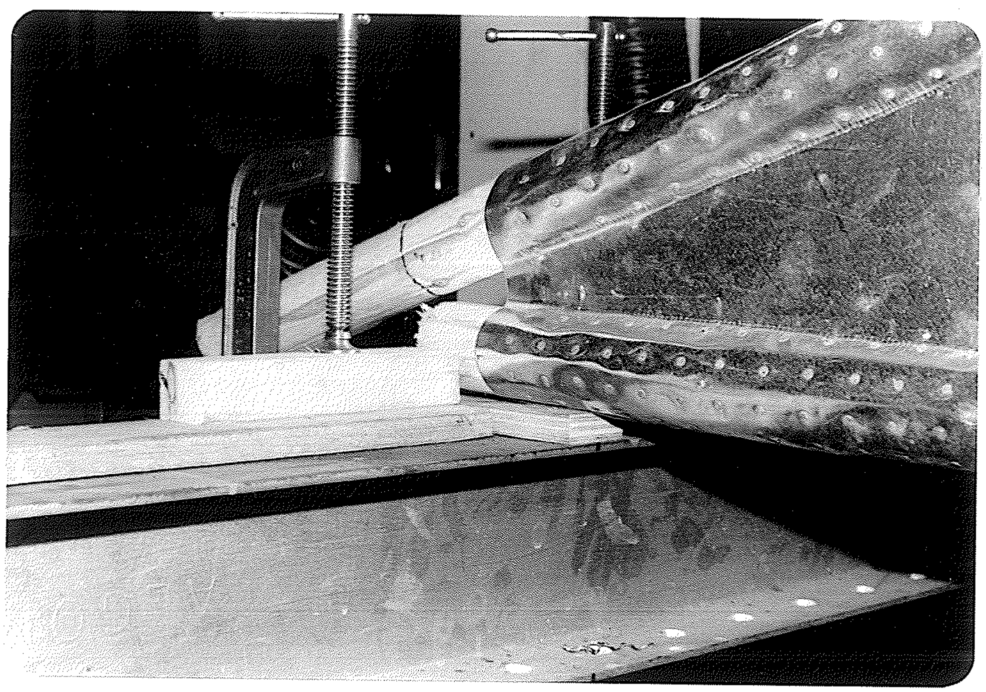


Fig. 4-4: End Support Condition

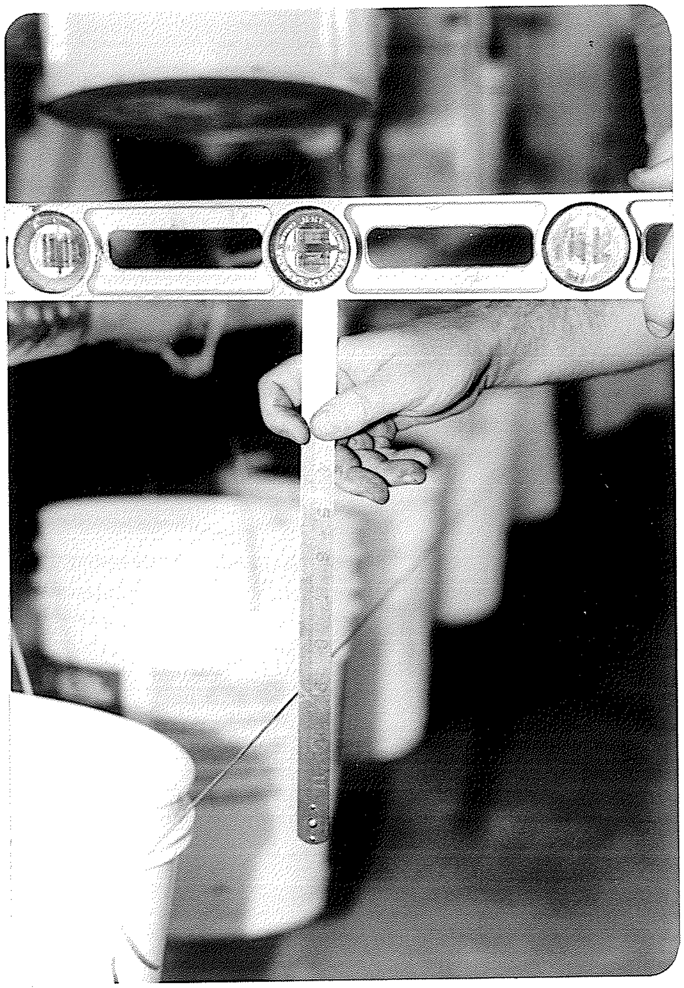


Fig. 4-5: Deflection Measurement

Deflection readings were taken by measuring from the taut wire to the bottom chord of the truss with a steel rule and spirit level (see Fig. 4-5). The accuracy of the readings was approximately 1/16 of an inch.

In section 4.4.1 the location of point loads was discussed. At each load point a five-gallon steel pail was attached to the truss with a rope or wire in such a way as to allow the pail to swing freely, avoiding contact with the truss members. The procedure at this stage then became a repeated set of steps:

1. deflection measurements were taken at each point, twice;
2. a 5-lb increment of load was added to each pail symmetrically, starting at the centre and

working outwards. A maximum of 20 pounds per pail was applied to the bottom chord to simulate load due to ceiling material;

3. deflection readings were taken at each point, and recorded;
4. an increment of load was applied to top chord, and was recorded;
5. deflection readings were taken again and recorded;
6. Steps 4 and 5 were repeated until failure.

The truss had minimal lateral support at three locations: at the peak; and at each end on the trestle. It is recognised that this represented a situation that would not likely occur within a completed structure. There was, however, concern and interest as to what load the truss could take in this very unfavourable condition which could occur during the erection phase on the roof. It was suggested that if the truss could withstand the load of two average to heavy village carpenters, in an almost unsupported condition, then it could withstand the erection process in Sierra Leone. It is a concern that during the erection stage of the roof, the truss must be able to withstand some abuse, as the quality of workmanship cannot always be guaranteed when the vigilance of a supervisor is absent.

4.6 TEST RESULTS:

Within this section the observations made, and measurements taken during each of the two truss tests is presented. Comments on the implications of the test results are left to Chapter Five.

4.6.1 TRUSS TEST T1:

A total load of 870 pounds was supported by truss T1 prior to failure. Of this total, 750 pounds was attached to the top chord, which represents an equivalent distributed load of 15.6 psf (based on a truss spacing of 2 feet on centre -- based on common construction practice in Canada). Fifteen pails were connected to the top chord (for a total of 750 pounds) and a total of 120 pounds (three pails) had been attached to the bottom chord when failure occurred. In hindsight, it was apparent that the initial load that was placed on the top chord of the truss was somewhat excessive and did not allow for the entire eleven loads to be attached to the bottom chord before failure occurred. The loads on the bottom chord were located at midspan, however, which produced the greatest stress on the structure with this load. Fig.4-6 illustrates the load configuration at failure.

The failure of the truss occurred at joint G (see Dwg.No. E24-1), midspan between the heel and the peak on the upper chord. Collapse of the truss was as a result of the timber member breaking at this point and not a consequence of connector malfunction. Failure of the truss had not really been expected to occur with this amount of loading, however, upon inspection of the failed member the heartwood was found to be 'punky' or soft.

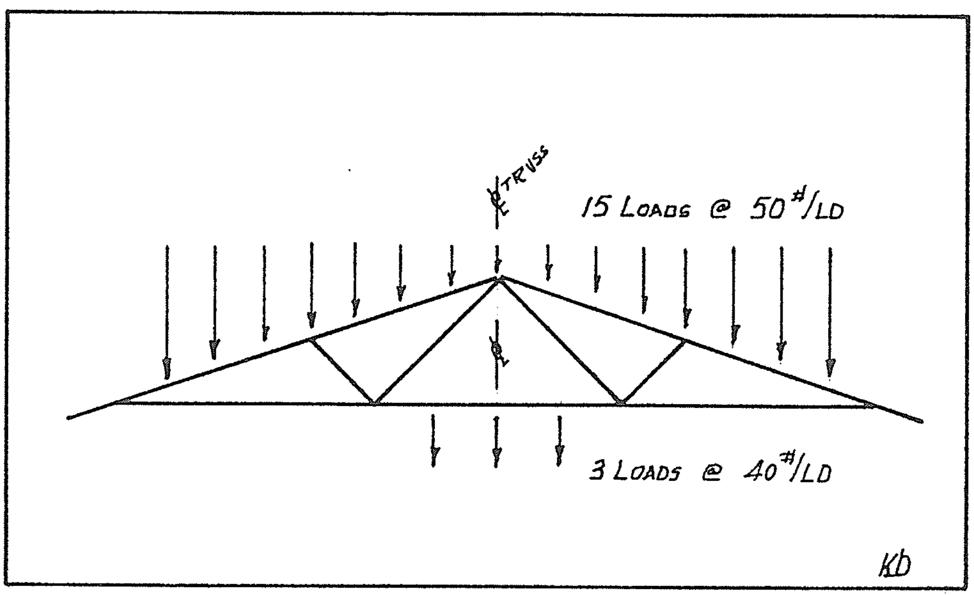


Fig. 4-6: Truss T1 Loading

The connector at G was removed to facilitate closer examination of the broken member. Of particular interest here was the amount of effort that was required to pull and pry the connector loose from the timber, which seemed to indicate that there was still plenty of holding power left at this joint. The mode of failure and the condition of the heartwood are illustrated in Fig. 4-7 and Fig. 4-8.

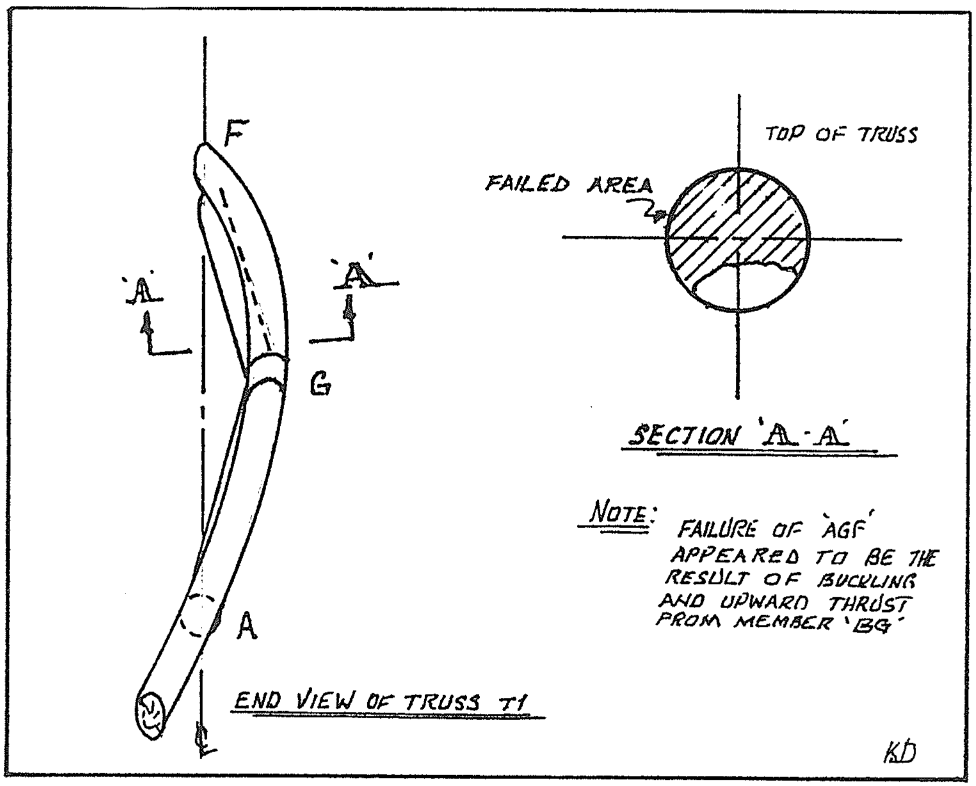


Fig. 4-7: Failure in Member AGF - Truss T1

After the collapse of the truss, photographs were taken of the failed timber for future reference. The section of the truss around panel point G was cut out and kept for further examination and photographs.

Fig. 4-8: Heartwood at Joint G - Truss T1

Due to the untimely collapse of the truss mentioned above there were no deflection readings taken, except for initial reference measurements. The test on truss T1 provided valuable guidelines for the second test, however, in addition to useful information concerning the connectors, that is discussed in Chapter Five.

4.6.2 TEST TRUSS T2:

The structural test of truss T2 provided information that augmented that obtained form the first test.

Having the results of test T1 to work with, it was decided that the initial loading of truss T2 should be lower, and that a slower pace would be used in adding incremental loads.

After reference readings were taken for deflection measurements a weight of 20 pounds per pail was placed on the top chord at the 15 load points. This corresponds to a total load of 300 pounds, or, an equivalent uniform load of 6.25 pounds per square foot (psf). Deflection readings were taken and recorded, after which the basic load of 20 pounds per point was applied to the bottom chord to simulate a ceiling load of 5 psf.

At this stage in the testing severe out-of-plane buckling occurred in member AGF (see Fig.4-9). It was decided that lateral support would be provided near the panel points G and E on the top chord to reduce this buckling and to allow for continuance of the testing, particularly since the connector capacity was a major matter to be explored. The lateral supports did not offer any additional load resisting capability to the truss. After the lateral supports were in place deflection readings were taken to check if any

Fig. 4-9: Buckling of Member AGF

movement had occurred as a result of the support application. The peak deflection measurement was retaken. It was found that the bottom chord of the truss had not moved, and so testing was continued. The reason for resetting the peak was that the lateral support at this location had been pushed slightly out of alignment, which would have rendered further deflection measurements suspect.

When the buckling occurred in member AGF the loads were removed to facilitate application of the lateral bracing. After the bracing was in place, the load of 520 pounds was replaced and left for a period of 1 hour and 20 minutes. Deflection readings were then taken, recorded and the load left at 520 pounds for a further 2 hours. This total of 3 hours and 20 minutes allowed the truss to seat itself and provided an opportunity to examine the connectors for any signs of buckling.

It had been decided earlier that if no additional deflection had occurred over this time span that an additional 5 pounds per point, or 75 pounds in total, would be added to the top chord of the truss. Deflection readings taken after the 3 hours and 20 minutes, supporting a load of 520 pounds indicated that no movement had taken place on the bottom chord. At the new total of 595 pounds deflection readings were taken, recorded and the truss left to stand until the next morning.

When the next set of deflection readings was taken, 18.5 hours had elapsed since the application of the last load increment. The measurements indicated that no additional deflection had occurred on the bottom chord, which implied that no inelastic deformation or creep had occurred in the truss. Also at this time, all of the connections were examined for signs of buckling or any other abnormalities, but none were found.

The truss was now subjected to an additional 75 pound increment, bringing the total load to 670 pounds. Deflection readings indicated that no further movement had taken place, so it was decided to increase loading another increment, raising the supported load to 745 pounds. When the measurements had been taken and recorded, the truss was left for 1 hour.

After the truss had supported 745 pounds for approximately one hour the standard load increment of 75 pound was again added to the top chord of the truss. With a total load of 820 pounds there was some interest in whether the top chord felt 'rubbery', an indicator of impending buckling failure. While the top chord was being 'tested', it was noticed that the peak of the truss was slowly moving down, and before any further observations could be made the structure collapsed. Again, the top chord member -AGF- had failed as a result of buckling near the lateral support and transferred its load to panel point B which also failed under the increased forces. Upon inspection of the connectors there was no indication that any of them had contributed to the truss failure. Any buckling of the connector material over and above what had been previously recorded while the truss was still upright was attributed to twisting during collapse.

Details of the deflection readings that were taken during the various loading phases are contained in Appendix 'J'. It was found that the total bottom chord midspan deflection was 1 inch. If a criterion for maximum deflection of the span/180 is used, the limiting value would be 1.6 inches. The measurements obtained from truss T2 indicate that it performed satisfactorily, in spite of the near total lack of lateral support, which could be expected in a well-braced structure in the field.

The condition of the connectors was examined closely during the testing procedure. The following observations, presented in point form, were recorded after the truss had initially seated itself with a total load of 520 pounds applied. (Note: These observations were recorded after the initial instability in member AGF. Thus, some buckling of material may have resulted from this.)

JOINT	CONNECTOR	LOAD	COMMENTS
A	Heel	520#	- no buckling
G	BG	520#	- slight buckling under top chord. - could be due to combination of member BG seating and AGF buckling.

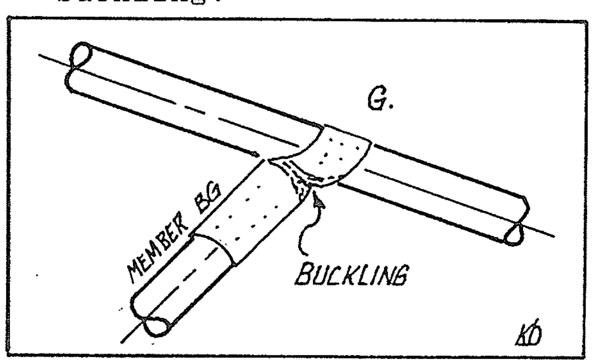


Fig. 4-10: Connector BG

JOINT	CONNECTOR	LOAD	COMMENTS
B	BG/BF	520#	- some buckling on one side, appeared to be result of members seating.

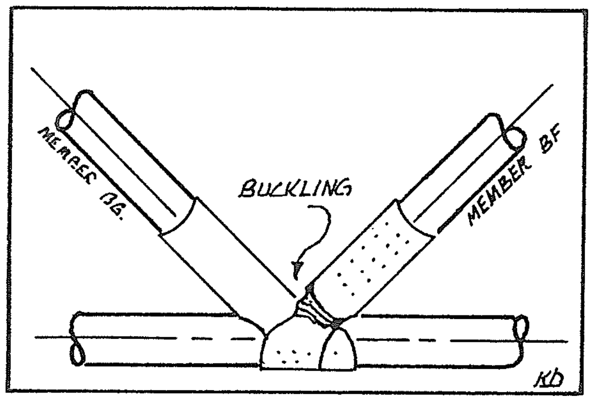


Fig. 4-11: Panel Point B

JOINT	CONNECTOR	LOAD	COMMENTS
F	Peak	520#	- no buckling, some seating taken place but seemed to create no problems.
H	Centre	520#	- no buckling or slippage
C	CF/CE	520#	- small amount of buckling due to member seating.
E	CE	520#	- no buckling or movement.
D	Heel	520#	- no buckling or movement.

During the remainder of the testing there was no change in the observed connector configuration except after collapse. The two halves of the peak connector did press together more tightly directly under the ridge pole during load, but there was no indication of buckling. Fig.4-12 illustrates the failure mode of truss T2.

Fig. 4-12: Failure Mode of Truss T2

FIVE

5.0 DISCUSSION OF RESULTS:

It must be admitted at the onset, that there was a certain amount of disappointment when the first test truss T1 failed at a load of 870 pounds, even though its function was to provide preliminary information so that test T2 could be observed in a more informed manner. As the examination of the timber and connectors progressed, however, combined with some reflection and calculation, the initial disappointment faded. It was replaced with an optimism based on few a facts that were immediately recognizable:

- the connecting system had produced a truss capable of supporting a load greater than 4 times what would be required of it in Sierra Leone;
- it was the timber that had failed and not the connectors;
- the poplar wood used in the truss was clearly of inferior quality as compared to that of the Kande tree; and
- the test was a bit unfair in that lateral bracing during testing was minimal in comparison to what would be encountered in the field, even in the worst case of inadequate supervision.

The information gleaned in the first test led to a second truss test -T2- which provided much more data. Although the ultimate total load the truss carried was slightly less than that carried by T1, 820 pounds, a good set of deflection data was gathered, along with important information regarding connector behaviour.

Three important factors had an impact on the performance of the two trusses:

1. joint construction;
2. timber quality; and
3. lateral support.

The discussion of the test results revolves about these three factors.

1. Joint Construction:

When the connecting system comes to be used in the field, careful fabrication of the joints cannot be guaranteed. It was for this reason that during the process of manufacturing the truss, the panel points were assembled with gaps between the timber members to simulate a less than

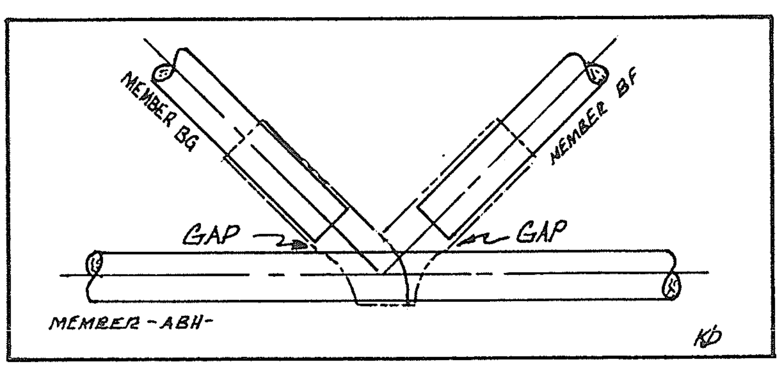


Fig. 5-1: Gaps Between Members at Joint B

ideal assembly. In Fig. 5-1, the connection of the members at joint B illustrates this point.

The only buckling of the connectors appeared to result from the seating of the members, that is, as the gaps between the timbers were closed up when the loading was applied. In section 4.6.2 the observations made on T2 connectors seemed to indicate that this was indeed the case.

Although the T1 truss did not sustain a load long enough to make detailed observations, its collapse, as well the failure of T2, did provide important information as to the performance and strength of the connectng system. In neither case did the connectors 'let go' of the timber. The two trusses each collapsed as a result of exceeding the timber

Fig. 5-2: Joint B - Truss T2, Post Collapse

strength and collapsed with the connecting system intact. In particular, after the top chord failure in truss T2, the load was transfered through member BG and the panel point B which resulted in member ABH failing (see Fig. 4-12,p.4.26) Except for a small amount of bending there was no apparent damage to the connector at B. What is particularly interesting is that none of the nails in the connector had been pulled, the connector remained tight to the timber. This implies that there was still quite a bit of strength remaining in the joint.

Based on these observations, and the overall performance of the system, it appears that the connectors functioned well. Although the criteria for use in Sierra Leone was exceeded, it is believed that the connecting system has the capability to withstand greater loads when combined with a superior grade of pole. This matter is discussed further in Appendix 'A'.

2. <u>Timber Quality:</u>

It is likely easier to find a needle in a haystack than it is to find a straight poplar pole on the Canadian prairies. The inconsistency in quality, taper and straightness of the poles placed the test trusses at a disadvantage from the onset. The rational for using this type of timber was based on two factors:

1. poplar poles are common and easily obtained;

2. if poplar could sustain the required loading in spite of its poor construction quality, then the timber used in Sierra Leone should perform at least equally as well.(see Appendix 'A')

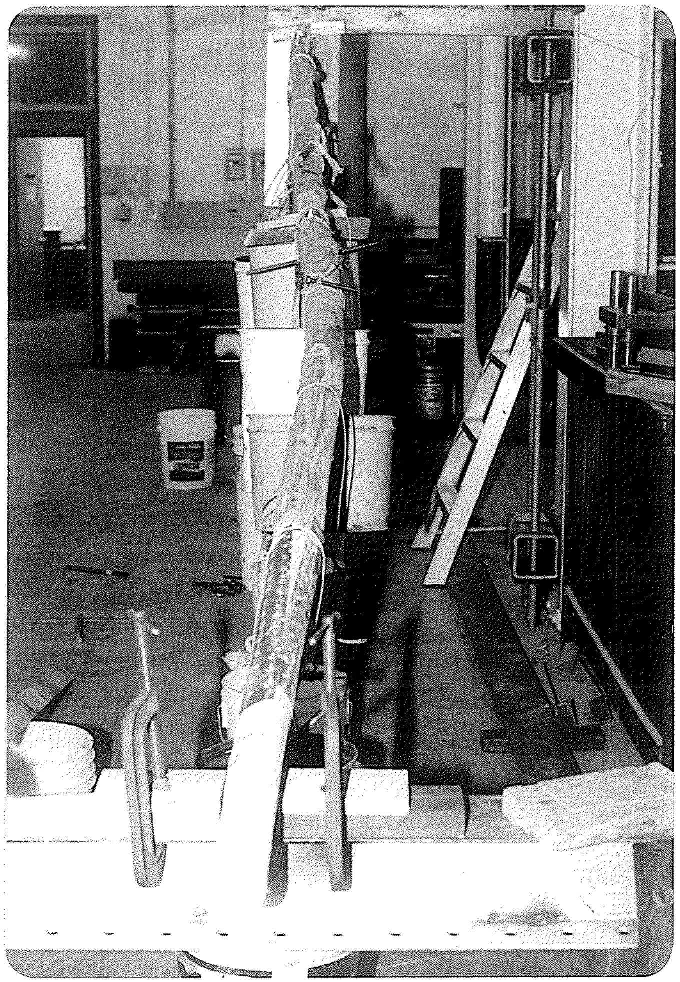


Fig. 5-3: Eccentricity in T1

An effort was made to procure local timber that was as sound as possible. Eccentricities, bends, and knots, however, were still present in all of the poles. Fig. 5-3 shows the eccentricity in the top chord member (DEF) of truss T1.

In both instances the timber in the truss failed before any signs of connector malfunction In truss T2 the forces in the top chord exceeded the calculated strength of the material, while in truss T1, it was seen that an inconsistent heartwood in the top chord likely promoted the failure. It could be seen upon inspection that the top chord of each truss caused the structures to collapse as a result of tensile failure brought on by the extreme out-of-plane bending that occurred. Panel point G in truss T1 is shown in Fig. 5-4 illustrating the tensile - buckling failure. Note that the connector is still in good condition.

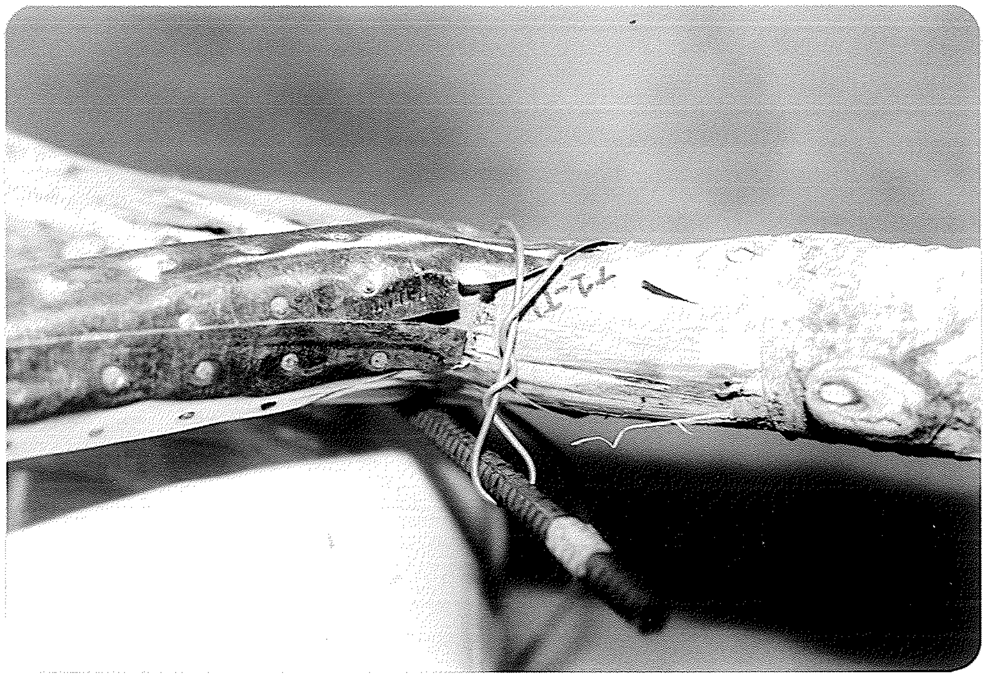


Fig. 5-4: Joint G - Truss T1, Post Collapse

Appendix 'C' contains information and calculations regarding the sizing of the timber for the trusses, recommending that a minimum diameter of 2.5" should be used for the top and bottom members. Although poles were selected in an attempt to comply with this requirement, the amount of taper found in the poplar poles precluded this standard from being met consistently over the entire length of the member.

Since all strength properties of timber depend upon its cross-sectional area, it was inevitable that the load carrying capacity of the actual truss would be reduced as compared to that calculated in Appendix 'C'. Appendix 'K' contains data on the taper in truss members.

3. Lateral Support:

The collapse of T1 and T2 was a direct consequence of the top chord of each truss being subjected to an axial force creating axial and bending stresses that exceeded the strength of the timber. In trusses generally, a compressive axial load acts on the top chord causing it to behave like a slender column, and subject to such design criteria as effective length and the Euler critical load. The force at failure in member AGF for both trusses has been calculated and is shown in Fig. 5-5 .

The general arrangement for structural testing required the truss to perform in a situation that would not likely be encountered in practice, that is with lateral bracing missing. Within a roof framing system the truss is one of many elements in the total diaphragm, acting in conjunction with other trusses and lateral bracing. It was felt that although the test might have been somewhat hard on the truss, erection procedures in Sierra Leone could place short term load on the structure that would rarely occur here during

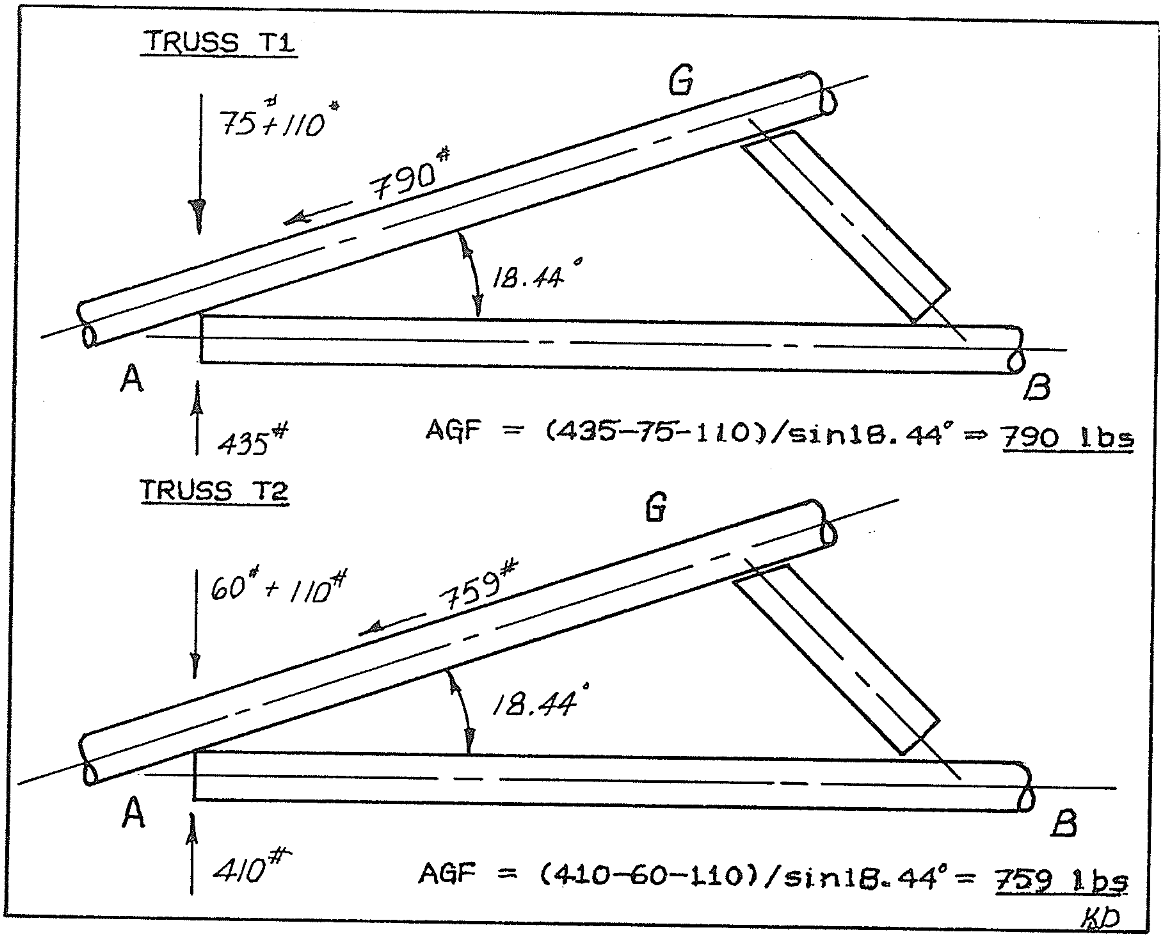


Fig. 5-5: Force in Member AGF at Failure

the same process, due to better field supervision. The main concern was for worker safety, so the single truss had to be able to support the weight of at least two village artisans, approximately 300 pounds. (Assumed symetrical in this case.)

The load carrying capacity of the truss in the completed structure was greatly reduced due to the lack of lateral bracing during the testing procedure. In normal practice, there would be roof purlins spaced a minimum of 4 feet on centres, running at right angles to the top chord of the truss. The impact that shortening the effective length of the top chord has is discussed using Euler's formula, a well known criterion for elastic column buckling.

From the information contained in Fig. 5-5 the axial load at failure in the top chord of T1 was 790 pounds. The shape of the buckling in Fig. 4-9, approximates that of a pinnned end column shown in Fig. 5-6. Based on these conditions and constraints the Euler formula was used to investigate the behaviour of the top chord.

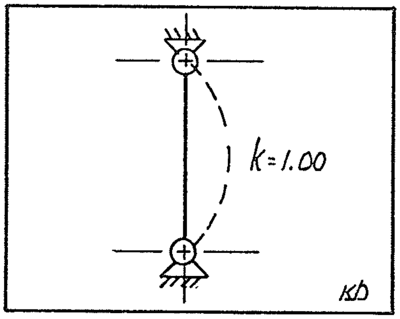


Fig. 5-6: Pin End Condition

The load at which column buckling occurs is referred to as the critical compressive force and is given by the following relationship; (after Timoshenko, Vol.2, 1951,pp. 184-195, and; LSD of Wood Structures, 1986, pp. 135-140.)

EQN 5-1: $$P_{cr} = \pi^2 EI / kL^2 \quad ,$$

where:

- Pcr = Critical compressive force
- E = Modulus of elasticity of timber member
- I = Moment of inertia of timber member
- k = Effective length factor
- π = PI, constant

Inputting the corresponding values for the failed truss T1 in equation 5-1 yields the following result. (Note: An average diameter of 2 inches was used in the calculation.)

Values input;

E = 971,500 psi (CAN3-086-M84, Table 50)

I = ¦¦d^4/64 in^4 = 0.7854 in^4

L = 150 inches (Length of top chord)

k = 1.0 (pinned end condition assumed)

Sustituting in equation 5-1;

Pcr = ((3.14^2)(971500)(0.7854)) / (150^2)

<u>Pcr = 334 lbs</u>

This value represents approximately 40% of the force that was reached in member AGF at failure. The fact that Pcr is considerably less than the obtained force in either T1 or T2 may be an implication of the factors listed:

- end conditions were not acting as pinned ends, (affects k);
- the force in web member BG was providing some assistance to member AGF in resisting buckling, (affects k);
- the taper of the cross-section does not average out to be 2 inches, (affects I);
- elastic modulus E of the timber was higher than calculated;
- the length of the column could be considered to be the distance between the heel and peak connectors, making the

length L = 120 inches. Pcr at this length becomes = 522 lbs, 66% of the T2 failure load.

Solving for `k' in equation 5-1 with Pcr equal to the failure load in T2, 790 pounds, yields an effective length factor of 0.42. This value is unlikely since Pcr for a fixed end column uses a theoretical `k' value of 0.5. It is surmized that the higher force value found, for member AGF in trusses T1 and T2, was caused by a combination of these five factors. The Euler elastic buckling equation does, nevertheless, point to the fact that lateral support is an extremely important parameter within the bush pole truss design process.

When the effective length of the top chord is reduced through the use of purlins, and bracing or sheeting, the critical compressive force, Pcr, increases dramatically. Pcr for braced purlin spacings of 2- and 4-foot centres are presented as examples of how the load carrying capacity of the truss is improved. (Note: A pole diameter of 2 inches was used in each of the calculations.)

PURLIN SPACING	-	2'-0"	4'-0"
Pcr (lbs)	-	13,074	3268

The resistance to buckling has been upgraded to such an extent that failure, instead of out-of- plane buckling collapse, would probably occur due to compressive or tensile stresses being exceeded within the truss. Table 5-1 relates the total load on the bush pole truss to the recommended minimum diameter of the top chord, based on the Euler critical compressive force, and previous assumptions.

TOTAL LOAD(1) (lbs)	DIAMETER REQ'D FOR 6'-0" (2) (in)	DIAMETER REQ'D FOR 12'-0" (3) (in)
200	1.37	1.93
500	1.71	2.43
1000	2.04	2.89
1500	2.26	3.20
2000	2.43	3.44
2500	2.57	3.63

Note: 1. Based on a net load acting downwards on the truss.
2. Represents the distance from panel point to panel point, unsupported.
3. Represents the distance from heel to peak,unsupported.
4. Based on Euler Pcr=π^2EI/L^2 , solved for d , where; d = (64PL^2/π^3E)^1/4 , and, P = P/2sin18.44 degrees

Table 5-1: Top Chord Diameter Based on Pcr (4)

If out-of-plane buckling is not considered a problem once purlins and bracing are in place, then the resistance to in-plane flexure must be examined to see if bending will potentially become a problem prior to a compressive or tensile failure. (Appendix 'B' contains information regarding appropriate allowable stresses.)

The bending moment suggested by TPIC, 1988, (p.16) to be used in top chord design is, M = wL^2/10, for panel point to panel point distance. If this equation is used in conjunction with the formula for flexural stress, M/S = @, then a relationship between the pole diameter and the loading can be established.

If; M = Bending moment = wL^2/10 in-lbs

S = Section modulus = ¦¦ d^3/32 in3

@ = Ultimate stress = 3000 psi , (based on Table G-1, Appendix 'G')

L = Panel length = 60 inches

Solving for 'd' yields;

d = ((32w(60^2) / 10¦¦(3000))^1/3

EQN 5-2 <u>d = (1.22 w)^1/3</u>

Table 5-2 was developed based on equation 5-2. In the table, a calculated pole diameter is related to various truss loading conditions. When the values obtained from EQN 5-2 were compared in the light of the observations made during the testing of the trusses, the calculated diameters appeared slightly conservative. Loads in excess of 800 pounds were

LOAD (w) (lbs/ft) (1)	UNIFORM LOAD (psf) (1)	TOTAL TRUSS (lbs) (1)	MEMBER DIAMETER (in) (2)	FACTORED DIAMETER (in) (3)
8	2.0	192	2.14	1.50
10	2.5	240	2.30	1.61
20	5.0	480	2.90	2.03
40	10.0	960	3.60	2.52
60	15.0	1440	4.18	2.93
80	20.0	1920	4.60	3.22

Note: 1. Based on truss spacing = 4'-0" on centres.
2. Calculated value using, d = (1.22 w)^1/3 .
3. 70% of value calculated in 2. Based on observed buckling of T1 at a load of 870 pounds and a diameter of 2.25" ; 2.25/3.6 = 62.5% .

<u>Table 5-2: Top Chord Diameter Based on Flexure and Modified on the Basis of Observation</u>

carried by the trusses before failure without any signs of deflection between the panel points. A calculated diameter for a loading of 960 pounds is 3.6 inches, compared to an average value of approximately 2.25 inches in the test truss. It is, therefore, recommended that the calculated values in Table 5-2 be reduced by a factor of 30% to yield a 'suggested' pole diameter. The factored diameter becomes; suggested / calculated = 70% .

Concern may exist that the factored diameters may not satisfy the allowable compressive stress in the top chord member -AGF- has been investigated for a total truss load correspsonding to 15 psf from Table 5-2. A diameter of 2.26 inches, from Table 5-1, has been used as the minimum allowable diameter for the panel length based on the previous discussion regarding buckling and flexure.

Axial Load in AGF = 1440 / 2sin18.44 degrees

= 2276 lbs

Allowable stress = 734 psi , (from Appendix 'B')

Stress in AGF = 2276 / (3.14x 2.26^2 / 4)

= 350 psi < 734 psi

It can be seen that the compressive stress created is within acceptable limits.

At the beginning of this chapter it was said the three main factors that affect the quality of the bush pole truss were; joint construction, timber quality and lateral support. It must be emphasized that no one parameter has a greater impact on the overall truss performance than do the remaining two. The performance of the bush pole structure is dependent upon the sum total of material assessment, careful design and proper construction.

Six

6.0 SUMMARY / CONCLUSION:

Structural testing of two 'bush pole' trusses has indicated that both the metal connecting system and the use of indigenous timber material provide an effective means of fabricating this roof framing component.

It was found that the truss displayed the ability to sustain an imposed load approximately four times greater than will be required of it in Sierra Leone, even when loaded in the most pessimistic manner. This result is made all the more promising when combined with the knowledge that the timber to be used for truss fabrication in West Africa is of superior quality in comparison to the poplar poles used for the test trusses.

Only slight buckling of the metal connectors was observed during structural testing. This buckling was seen to coincide with seating of the timber truss members as load was applied. Once this seating process was complete, further distortion of the connecting system was not apparent.

In each of the tests, collapse of the structure was induced by failure in the top chord of the truss. Severe out-of-plane buckling in the timber member created stresses in the material that were in excess of those that could be resisted. This condition led to a buckling failure in the timber member.

The failure of both trusses was seen to be a consequence of exceeding the strength characteristics of the timber material. Post-collapse inspection indicated that malfunction of the connecting system was not a factor in the inability of the truss to sustain further loading. Increased lateral support to the truss during the structural testing is seen as an important factor in future investigations of this connecting system.

In addition to considering the structural integrity of a truss constructed in this manner, the potential for creating an income-generating activity through the manufacture and marketing of a metal connecting system was also cited as an objective. To investigate the feasibility of this process in a developing region, the connecting system will be manufactured and tested in Sierra Leone starting in January of 1989. From the work done to date with the system it appears that it has every chance of proving to be a success. It would, however, be less than prudent to make any statements beyond this point, except to say, that it is envisioned that this in-field testing period will provide an opportunity to lay the groundwork for advancing the connecting system from the development stage to an implementation phase. The information obtained while in Sierra Leone will be most important in directing further development of the metal connecting system, and in defining appropriate conditions for future structural tests.

The metal connecting system for bush pole trusses has proven to be feasible. The potential of this connecting system, however, is not seen as being restricted for use in West Africa only. Developing regions in both the north and south of Canada could benefit from this level of technology. The ability to produce structural components that have the capability to make the provision of shelter more affordable by an increasing population and to enhance the options for self-sufficiency, appears as an important contribution.

SEVEN

7.0 RECOMMENDATIONS FOR FURTHER RESEARCH:

The main reason for including this chapter in the thesis was to address the times when the phrase, "...I wish I had the time to investigate ... more", was said. During the design, development, and testing of the bush pole truss some ideas for further investigation emerged. Three of them have been discussed below.

1. CONNECTOR TESTING APPARATUS:

In both test situations, T1 and T2, the collapse of the truss was brought on by the failure of the timber material. Aside from a small amount of buckling due to seating of the truss members, the metal connectors did not malfunction to any great extent. It is recommended, therefore, that a testing apparatus be developed to specifically examine connector behaviour up to failure.

A weldment using hydraulic jacks to simulate the loading conditions at specific panel points within the truss, is proposed. (see Fig. 7-1.) With this arrangement the connectors could be conveniently stressed to a higher degree than within a complete structure. The timber members would be sufficiently short enough to reduce the risk of timber failure prior to connector malfunction.

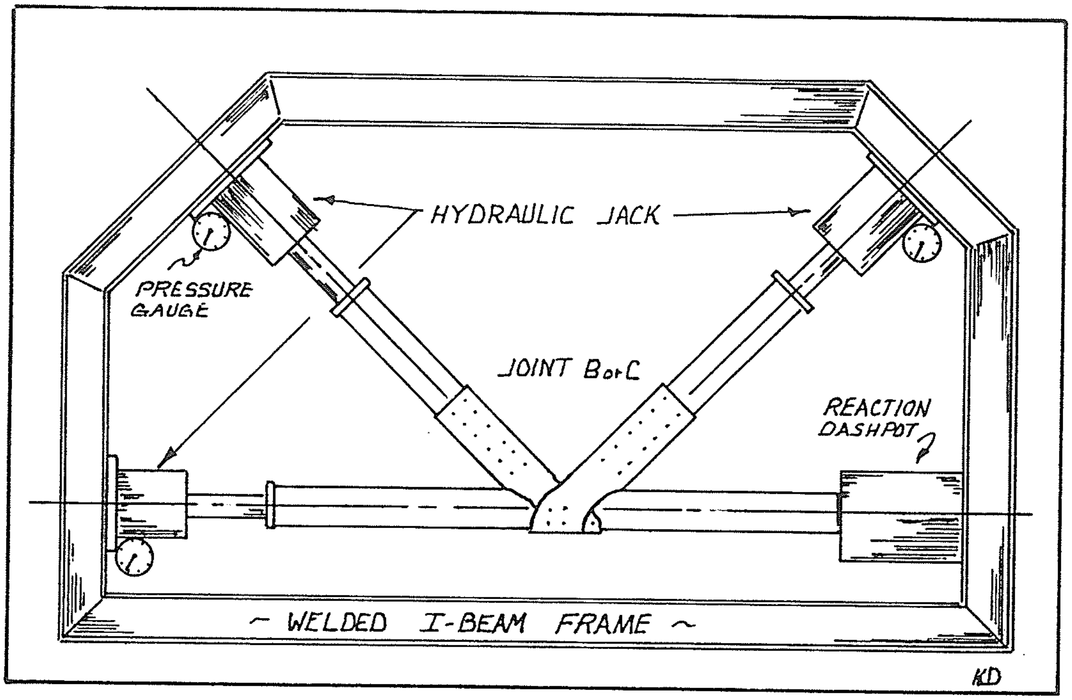


Fig. 7-1: Connector Testing Apparatus

2. SPRUCE POLE TRUSSES:

From a practical standpoint, the use of Spruce trees for pole trusses would be a better material choice than poplar. It is further envisaged that this connecting system has a potential for use in developing regions of Canada. It is recommended that the same style and configuration of pole trusses be fabricated using spruce poles and tested fully in accordance with CSA-S307-M1980, which was used as the basis of tests T1 and T2. It is further recommended that the spruce trusses be tested in a pair, with all the pertinent bracing and sheeting that would be used within the structure based on its end use location.

3. TIMBER QUALITY GAUGE:

An appropriate testing gauge, that would give a carpenter or engineer the ability to get some idea as to the quality of the timber poles, prior to truss construction, would be an asset. It is seen as imperative that the testing device be a transferable technology and not dependent upon elaborate gauges, electricity or the like. Fig. 7-2 is a sketch of what such a device might look like.

A known weight, with an attached pin or probe (1/8" diameter or less), would be dropped a set distance through a guide. Since the probe-weight combination hits the timber with an equal force each time, a relationship between penetration into the wood and location within the cross-

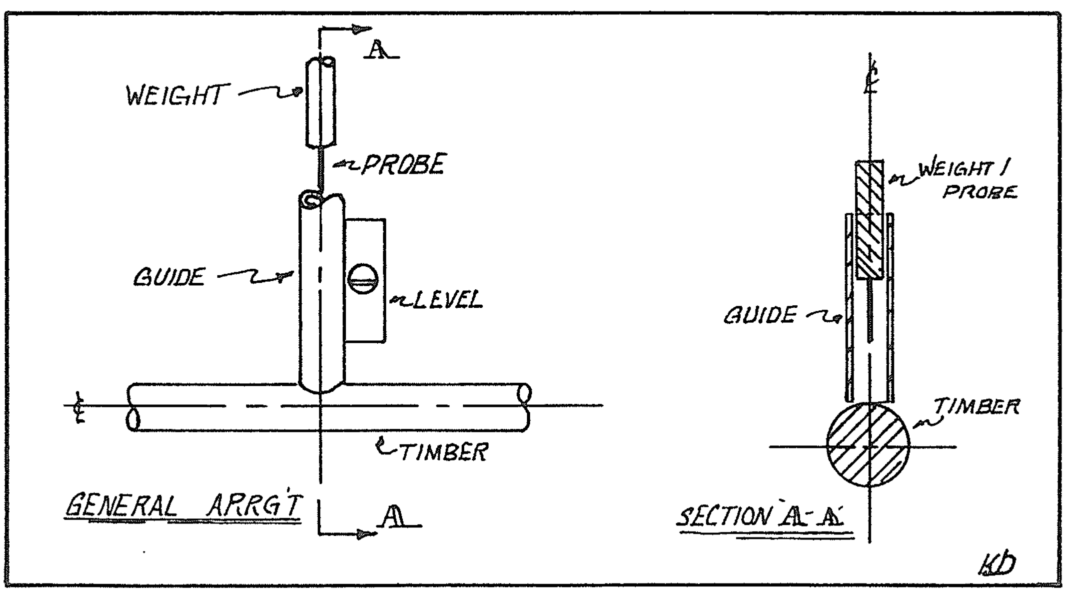


Fig. 7-2: Timber Quality Gauge

section can be developed. Every time the probe has been dropped the wood is drilled out to the depth of penetration, until the centre of the pole is reached. If the test pole were to be used in the structure it has been suggested that the probe hole could be along the neutral axis in bending members. The suitablity of a pole for use in a truss could be related as follows;

MATERIAL TYPE - PENETRATION DEPTH - LOCATION IN CROSS SECTION - MAXIMUM ALLOWABLE STRESS

Development of the `Connector Testing Apparatus', and the `Timber Quality Gauge' is recommended for future projects related to `bush-pole trusses'.

4. CONSTRUCTION LOADS:

Workmen crawling around on the truss may create non-uniform loads on the structure. Future truss testing should include unbalanced concentrated load configurations.

5. RECLAIMED METAL CORROSION:

The metal connectors formed from reclaimed material should be investigated to observe their behaviour. Given that autobody sheet steel is more ductile than 24 gauge galvanized sheet metal, combined with potentially signifigant corrosion, this may result in potentially less rigid joints.

APPENDICES

METRIC - IMPERIAL EQUIVALENTS

METRIC		IMPERIAL
LENGTH		
25.4 millimeters(mm)	=	1 inch (in)
1 metre (m)	=	3.28 feet (ft)
AREA		
645.16 square millimeters(mm^2)	=	1 square inch (in^2)
1 square metre (m^2)	=	10.76 square feet(ft^2)
FORCE		
1 newton (N)	=	0.224 pound force (lb)
1 kilonewton (kN)	=	224 pound force (lb)
STRESS		
1 megapascal (MPa)	=	145 pounds per square inch (psi)
LOADING		
1 kilonewton per square metre (kN/m^2)	=	20.88 pounds per square foot (psf)
1 kilonewton per metre (kN/m) (= N/mm)	=	68.52 pounds per foot (lbs/ft)

APPENDIX 'A'

DATA ON 'KANDE' or 'MONKEY APPLE' TREE:
SIERRA LEONE, WEST AFRICA (1986)

Date: October, 1986

Location: Yonni, Sierra Leone

Ultimately, the connecting system developed within this thesis will be used in house, school and meeting hall construction in Sierra Leone, West Africa. Prior to leaving Sierra Leone in 1987, after a two year stay, the writer conducted tests on the local variety of tree that is used in roof construction - the Kande or Monkey Apple tree - in order to gain some information as to its engineering properties. The tests that were performed are outlined in this appendix.

The objective of the testing was to try and establish some of the material properties of this variety of tree that are required for design purposes. The properties determined were; resistance to nail withdrawal, moment capacity, and the modulus of elasticity.

In the absence of elaborate testing equipment, simple alternatives were employed utilizing material that was on hand. Figures A1-1 and A2-1 illustrate these alternatives.

The 'bush poles' (a colloquial term for any tree used in building) were cut and allowed to air dry for approximately three weeks. This time span represents the average delay between cutting in the local forest and the pole's subsequent use in structures in the area. This does not represent the properties of the wood over the useful life of the structure but this provides information about the material on an initial basis. Care was taken to select specimens that were straight and had a cross section with little variance in diameter or taper from end to end. A total of 15 poles were cut, with the final 10 test pieces selected from among this group of 15.

SUBSECTION A1 - NAIL WITHDRAWAL TESTING:

In Figure A1-1 can be seen the apparatus and arrangement that was used in determining the nail withdrawal capacity of the tree.

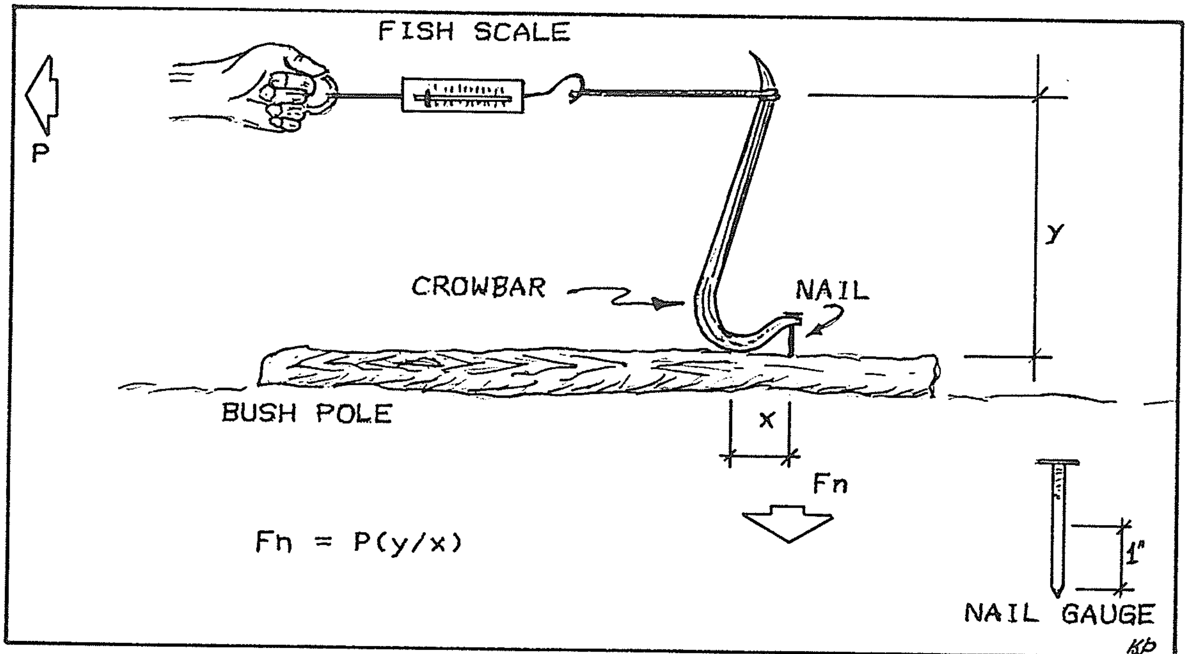


Fig. A1-1: Nail Withdrawal Testing Arrangement

The items used in the test were;

1 - crowbar

1 - fish scale, 0 - 100 lbs

10 - each, common wire nails:
6d - 2.0" - 11 BWG ,
8d - 2.5" - 10 BWG ,
10d - 3.0" - 8 BWG ,and,
20d - 4.0" - 6 BWG .

NAIL WITHDRAWAL TEST - PROCEDURE:

Ten common wire nails of 2.0", 2.5", 3.0" and 4.0" were each marked 1 inch up from the point of the nail where the nail first becomes a straight shank. (see Fig. A1-1) Each group of nails was driven into a seperate pole. Adequate spacing was ensured so that cracking of the pole would not be initiated. All nails were driven in normal to the pole, and, in a section where the diameter of the pole was at least 2 inches in diameter. The crowbar was positioned, then the nail was pulled out by means of a rope and fishscale attached to the crowbar as in Fig. A1-1. Care was taken to keep the rope and scale parallel with the pole to ensure that the proper force system was maintained. The force registered on the fish scale just as the nail started to move was recorded.

NAIL WITHDRAWAL TEST - CALCULATIONS:

The resisting force of the nail was determined from the following relationship;

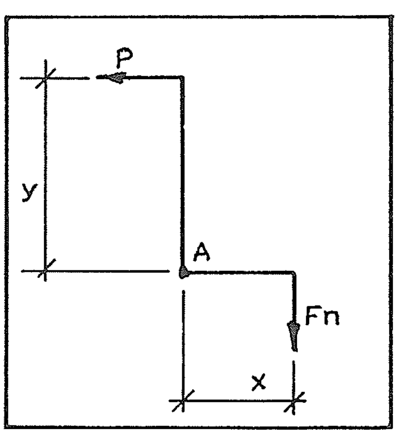


Fig. A1-2: Force System

where;

x = 2.75 in.

y = 21.50 in.

Taking moments about A yields;

MA = Py - Fnx , reduces to

Fn = P (y/x) = P(21.5/2.75)

Fn = 7.82 P

All of the readings obtained from the fish scale were, therefore, multiplied by a factor of 7.82 to give the actual resisting force Fn of the nail. The values recorded for each set of 10 nails were then averaged and this figure used for the unit withdrawal capacity of the nail.

SUBSECTION A2 - BENDING TEST:

Figure A2-1 shows the arrangement of equipment for determining the bending stress and bending moment resistance, from which the modulus of elasticity was also computed.

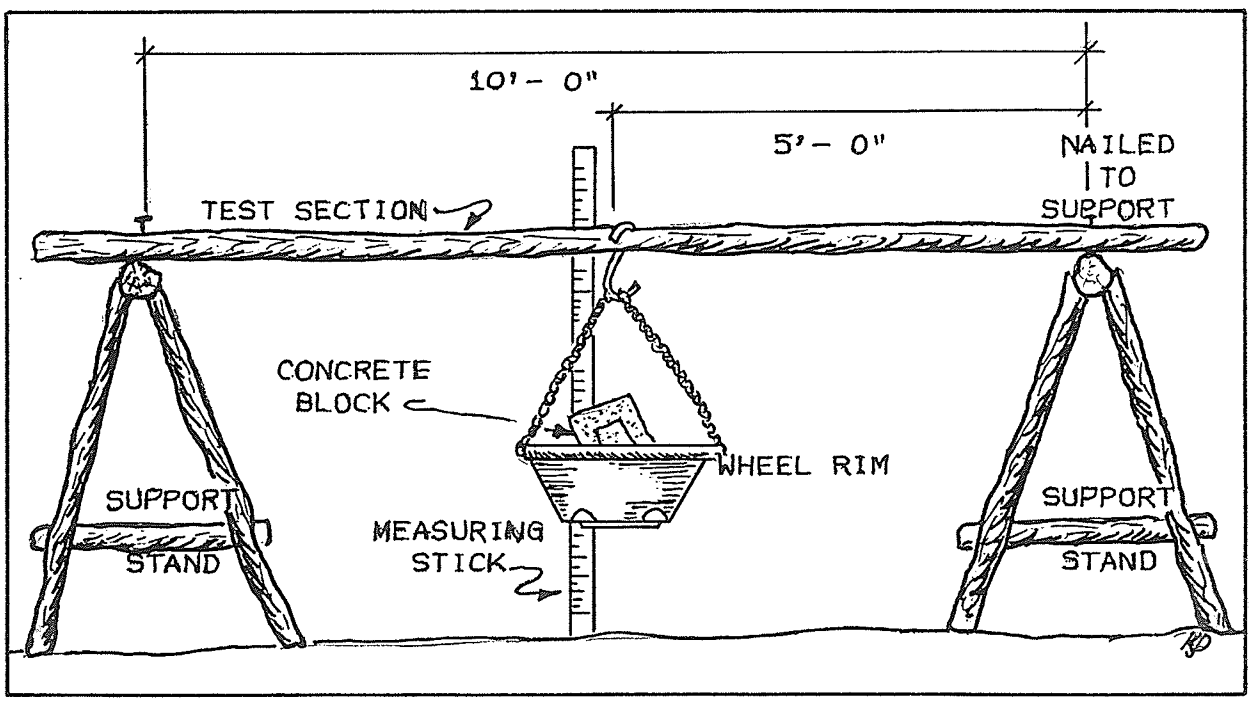


Fig. A2-1: Bending Test Equipment Arrangement

The material used to conduct the bending test was;

1 - 16" wheel rim	- rope and hook
1 - 24" spirit level	1 - weight scale
2 - support stands	1 - felt tip marker
20 - 20d (penny) wire nails	- various blocks of known weight
10 - bush poles, 12 - 14 feet long	

BENDING TEST - PROCEDURE:

The section to be tested was placed on the two stands as in Fig. A2-1. The specimen was then marked with a felt pen to indicate the support point spacing of 10'-0", and the midspan point of 5'-0". The stands were also marked to ensure that the specimen was always placed in the same location for each test. The specimen was then nailed to the stand at each end to prevent slippage under load, however, by doing this, the pole was not restrained from rotating on the support and any resisting moment created at the ends was felt to be negligible.

Prior to the test the wheel rim and various concrete blocks were weighed. These blocks were placed in the wheel rim in the same order during each test as follows;

1. Wheel Rim - 40 lbs
2. Concrete Block 1 - 42 lbs
3. Concrete Block 2 - 38 lbs
4. Concrete Block 3 - 32 lbs, in addition

to these were 3 boxes of roofing nails each weighing 7 lbs.

Before the wheel rim was hung on the pole an initial measurement was taken using the measuring stick and spirit level. This was done to determine the specimen's height above the ground and be a reference for subsequent deflections of the beam as load was applied. Also, the circumference of each end and the mid point of the test specimen was recorded for calculation of diameters. Next, the wheel rim was hung on the pole and the resulting displacement measured using the measuring stick and level. This process was repeated as each block was loaded into the wheel rim and continued until either a maximum load of 173 pounds was reached, or failure of the test specimen occurred. This procedure was carried out on all ten specimens and the information recorded.

BENDING TEST - CALCULATIONS:

The material properties to be determined from the bending test, stress and modulus of elasticity, were computed using the principles of engineering mechanics.

Calculation of E:

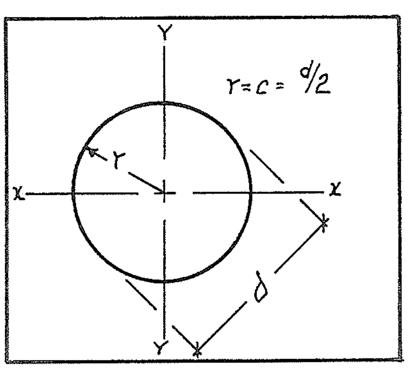


Fig.A2-2: Section Properties

The modulus of elasticity(E) was calculated using the following relationship between load and deflection;

<> = PL^3 / 48 EI, solving for E;

E = PL^3 / 48<>I

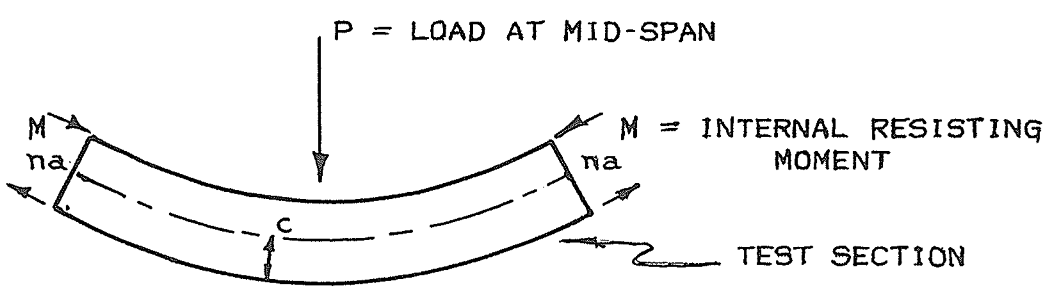
where:

P	= Load at midspan	-	lbs
L	= Span of beam	-	in
<>	= Deflection	-	in
I	= Moment of inertia	-	in^4

The value for E (psi) was calculated for each loading condition on all specimens. The average E value was determined from a maximum of 7 loading conditions and the corresponding deflections. This average value was then used in the calculation of the strain.

Calculation of @:

The stress at the extreme fibre of the pole was computed from the following relationship between the bending moment and the section modulus.



$$@ = \frac{M}{S} = \frac{M}{I/c} = \frac{8PL}{\pi\ d\wedge 3} \quad \text{(psi)}$$

Fig. A2-3: Section Properties in Bending where;

@	= Stress at extreme fibre	-	psi
P	= Load at midspan of beam	-	lbs
L	= Span of beam	-	in
c	= Distance from neutral axis to extreme fibre (same as r)	-	in
na	= Neutral axis		
d	= Diameter of section	-	in
M	= Bending moment, PL/4	-	lb-in
S	= Section modulus, (||d^3/32)	-	in^3
π	= PI, constant, 3.1415927		

The stress was calculated for each loading condition using the above relationship. The value obtained for the stress was then used in the calculation of the strain using the following well known relationship;

E = @ / e , where;

E	= Modulus of elasticity	-	psi
@	= Stress at extreme fibre	-	psi
e	= Strain	-	in/in

Solving for e;

e = @ / E , the values obtained can be used to plot a stress - strain graph.

SUBSECTION A3 - TEST RESULTS:

The results of the nail withdrawal and bending tests appear tabulated at the end of this appendix.

SUBSECTION A4 - CONCLUSION :

Although the equipment used for testing was not elaborate, the results obtained give a good idea of the Kande tree's material characteristics and strength. The E values that were calculated appear fairly consistent, thus, at this stage it is assumed that reasonably accurate information has been determined in this testing. When the values for E and @ are compared with those for Northern Aspen, it appears that the use of Northern Aspen in the laboratory testing of the truss was conservative. These parameters are compared in Table A4-1.

SPECIES GROUP	MODULUS OF ELASTICITY (psi)	BENDING STRESS (psi)
KANDE	1,120,000	8,000
N. ASPEN	971,500 (1)	2,152 (2)

Notes: 1. Based on CAN3 - 086 - M84, Table 50.
2. Based on CWC Datafile WD-2 (see Appendix 'B')

Table A4-1: Comparison of Kande and N. Aspen

SUBSECTION A5 - TABULATED RESULTS:

NAIL WITHDRAWAL:

NAIL SIZE (in)	WITHDRAWAL FORCE lbs	kN
2	229	1.02
2.5	261	1.16
3	361	1.60
4	415	1.85

Table A5-1: Nail Withdrawal Capacity - Kande Tree

BENDING TEST:

There were seven possible increments of load during the bending test. For each load increment the bending stress was calculated using the relationship; M / S = @. The E-value was determined from the deflection reading at each load increment and the equation; E = PL^3 / 48<>I (see Fig. A2-2). An average E-value was determined from these figues and appears in Table A5-2.

TEST SECTION No.	NOMINAL DIAMETER (in)	SECTION MODULUS (in^3)	MOMENT OF INERTIA (in^4)	AVERAGE 'E' VALUE (psi)
1	2.29	1.179	1.350	905,000
2	2.25	1.118	1.258	1,069,000
3	2.33	1.242	1.447	1,011,000
4	2.10	0.909	0.955	1,432,000
5	1.73	0.508	0.439	1,134,000 (1)
6	1.81	0.582	0.527	1,218,000
7	2.15	0.976	1.049	678,000 (2)
8	1.81	0.582	0.527	1,303,000
9	1.91	0.684	0.653	970,000
10	1.69	0.474	0.400	1,455,000 (3)

Notes:
1. During the sixth load increment the nail gave way on one end of the specimen, thus, any results were suspect after that point.
2. The specimen tested was drier than the rest and the midspan load was next to a knot in the pole. The failure was brittle at the knot.
3. This specimen failed slowly, not brittle like No.7 which allowed for good observation of the failure process.

<u>Table A5-2: Modulus of Elasticity</u>

APPENDIX 'B'

CALCULATION OF ALLOWABLE DESIGN STRESSES:

BASED ON: CSA CAN3 - 086 - M84,
TPIC - January, 1988,
TDM - 1980, and,
CWC - Datafiles, 1986.

The following subsections of Appendix 'B' illustrate the process in determining the allowable stress, 'f', that has been used in the design of the timber truss members. The initial value of each particular stress was obtained from the Canadian code - CSA CAN3-086-M84 , and, the Timber Design Manual(TDM), corresponding to the classification of Northern Aspen, Grade No. 2. These values are subjected to 'stress modification factors' employed to better reflect the type of working environment in which the truss will be ultimately used.

The stresses of concern within Appendix B are:

B1 - Tensile Stress, Parallel to grain,

B2 - Compressive Stress, Parallel to grain,

B3 - Compressive Stress, Perpendicular to grain,

B4 - Bending Stress, at extreme fibre,

B5 - Combined Stress - Tension and Bending, and

B6 - Combined Stress - Compression and Bending.

SUBSECTION B1 - ALLOWABLE TENSILE STRESS:

The basic relationship of load, cross sectional area and tensile stress is given by the following:

$$P/A \le Ft\,(Kd * Kt * Ks * Kh * Kzt * Ki) \quad ,$$

where;

P = Tensile force

A = Cross sectional area of member

Ft = Initial allowable stress

Kd = Load duration factor

Kt = Treatment factor

Ks = Service condition factor

Kh = Load-sharing factor

Kzt = Size factor for tension

Ki = Importance factor

Table B1-1 presents the value assigned to each stress modification factor.

STRESS FACTOR	ASSIGNED VALUE	CODE/ SOURCE	CLAUSE No.	REMARKS
Kd	1.15	CAN3	4.4.1.2	Snow loading
Kt	1.00	CAN3	4.4.3.2.2	No treatment
Ks	1.00	CAN3	4.4.2.3	Dry service
Kh	1.00	CAN3	4.4.4	Tension member
Kzt	1.2	TPIC	4.3.4.4	Tension member
Ki	1.00	TPIC	4.3.4.7	Not LHO building

Table B1-1: Stress Modification Factors - Tension

The allowable stress can be calculated once the stress modification factors have been determined. The initial value of 'ft' is obtained from Table 50, p.175, CAN3-086-M84, (3.1 MPa for N.Aspen No. 2 grade) which yields the following:

Fa = Fallowable = 3.1(1.15 * 1.0 * 1.0 * 1.0 * 1.2 * 1.0) MPa

The allowable tensile stress becomes therefore;

Tensile Stress-Ft = 4.28 MPa (620psi)

SUBSECTION B2 - ALLOWABLE COMPRESSIVE STRESS, PARALLEL:

The basic value of 'fc' , compression parallel to the wood grain, is subjected to the following stress modification factors;

P/A < or = fc (Kd * Ksc * Kt * Kh * Kc * Ki)

where;

P = Compressive force

A = Cross sectional area of member

fc = Initial compressive stress allowed

Kd = Load duration factor

Ksc = Service condition factor

Kt = Treatment factor

Kh = Load sharing factor

Kc = Slenderness modification factor

Ki = Importance factor

Table B2-1 lists the values assigned for the various stress modification factors.

STRESS FACTOR	ASSIGNED VALUE	CODE/ SOURCE	CLAUSE No.	REMARKS
Kd	1.15	CAN3	4.4.1.2	Snow loading
Ksc	1.00	CAN3	4.4.2.3	Dry service
Kt	1.00	CAN3	4.4.3.2.2	No treatment
Kh	1.10	TPIC	4.3.4.2	Compression member
Kc	1.00	CAN3	4.5.3.3.6	Cc < 10
Ki	1.00	TPIC	4.3.4.7	Not LHO building

Table B2-1: Stress Modification Factors - Compression

An initial value of 'fc' = 4.0 MPa is used in determining the allowable compressive stress, Fc, parallel to the grain for N.Aspen, Grade No. 2. (CAN3 ,p.175, Table 50.)

$$Fc = 4.0\ (1.15 * 1.0 * 1.0 * 1.10 * 1.0 * 1.0)\ \text{MPa}$$

Therefore;

$$\boxed{\text{Allowable Compressive Stress-Fc} = 5.06\ \text{MPa}\ (734\ \text{psi})}$$

SUBSECTION B3 - ALLOWABLE COMPRESSIVE STRESS, PERPENDICULAR:

The compressive stress, 'fcp' , perpendicular to the grain is modified as follows;

P/A < or = fcp (Kd * Ksc * Kt * Kh * Ki * Kc * Kb)

where; P = Compressive force

A = Cross sectional area of member

fcp = Initial compressive stress

Kd = Load duration factor

Ksc = Service condition factor

Kt = Treatment factor

Kh = Load sharing factor

Ki = Importance factor

Kc = Slenderness modification factor

Kb = Bearing factor

Table B3-1 lists the stress modification factors and the values assigned to them.

STRESS FACTOR	ASSIGNED VALUE	CODE/ SOURCE	CLAUSE No.	REMARKS
Kd	1.15	CAN3	4.4.1.2	Snow loading
Ksc	1.00	CAN3	4.4.2.3	Dry service
Kh	1.10	TPIC	4.3.4.2	Compression member
Kt	1.00	CAN3	4.4.3.2.2	No treatment
Ki	1.00	TPIC	4.3.4.7	Not LHO building
Kc	1.00	CAN3	4.5.3.3.6	Cc < 10
Kb	1.10	CAN3	4.5.4.2	Bearing = 100 mm

Table B3-1: Stress Modification Factors - Compression(P)

The allowable compressive stress perpendicular to the grain can now be calculated. The initial stress value is 1.23 MPa for N.Aspen, Grade No.2.(CAN3-086-M84, Table 50, p.175.)

Fcp = 1.23 (1.15 * 1.0 * 1.0 * 1.1 * 1.0 * 1.0 * 1.1) MPa

Thus,

Allowable compressive stress-Fcp = 1.71 MPa (248 psi)

SUBSECTION B4 - ALLOWABLE BENDING STRESS:

The allowable stress at the extreme fibre of the member due to bending action, 'fb' , is subject to the following stress modification.

M/S < or = fb (Kd * Ksb * Kt * Kzb * Kh * Ki * Kg)

where;

M = Bending moment

S = Section modulus of member (3.14*D^3/32)

fb = Initial bending stress

Kd = Load duration factor

Ksb = Service condition factor

Kt = Treatment factor

Kzb = Size factor

Kh = Load sharing factor

Ki = Importance factor

Kg = Specific grade factor

Table B4-1 presents the stress modification factor values for bending stress.

STRESS FACTOR	ASSIGNED VALUE	CODE/ SOURCE	CLAUSE No.	REMARKS
Kd	1.15	CAN3	4.4.1.2	Snow loading
Ksb	1.00	CAN3	4.4.2.3	Dry service
Kt	1.00	CAN3	4.4.3.2.2	No treatment
Kzb	1.50	TPIC	4.3.4.4	Bending member
Kh	1.15	TPIC	4.3.4.2	Bending member
Ki	1.00	TPIC	4.3.4.7	Not LHO building
Kg	0.55	CAN3	5.4.2.1	Construction grade

Table B4-1: Stress Modification Factors - Bending

In accordance with Canadian Wood Council (CWC) - Datafile WD-2, p.27, which states, "strengths for round poles are taken as 80% of the specified strength for Select structural grade", the value of fb becomes; fb = 13.6 MPa for N.Aspen (from Tables 9 and 13, pp.22, 27, CWC - Datafile WD-2.)

The modified value of the allowable bending stress at the extreme fibre is calculated using;

Fb = 13.6 (1.15 * 1.0 * 1.0 * 1.5 * 1.15 * 1.0 * 0.55) MPa,

which yields;

Allowable bending stress-Fb = 14.84 MPa (2151 psi)

<u>SUBSECTION B5 - COMBINED STRESS, TENSION AND BENDING:</u>

Interaction of the members in the truss can potentially create a situation of combined loading involving tension and bending stresses. Therefore, the effect of these stresses is checked against a standard inequality. There is no set value in this relationship. As long as the stress values meet the criterion all is assumed to have met the standard. The inequality is as follows;

$$\underset{\text{TENSION}}{\frac{P/A}{Ft}} + \underset{\text{BENDING}}{\frac{M/S}{Fb}} \leq 1.0$$

SUBSECTION B6 - COMBINED STRESS, COMPRESSION AND BENDING:

As in subsection B5, the combined stresses of bending and compression are examined using a similar format to establish whether standards are met. The basic inequality is as follows.

$$\underset{\text{COMPRESSION}}{\frac{P/A}{Fc}} + \underset{\text{BENDING}}{\frac{M/S}{Fb}} \leq 1.0$$

If the summation of these two values is found to be less than unity, then the standard has been satisfied.

SUBSECTION B7 - SUMMARY OF ALLOWABLE STRESSES:

Table B7-1 summarizes the factored allowable stresses. The table presents data based on two sources; CSA CAN3-086-M84, Table 50,p.175, and, CWC Datafile WD-2, Tables 9 and 13,p.27, specified strengths for poles. (1986).

STRESS TYPE	CAN3 Gr.#2	80% OF CAN3 SELECT	CWC WD-2 (1)
Ft	621 (4.28)	721 (4.97)	1620 (11.17)
Fc	734 (5.06)	777 (5.36)	1482 (10.82)
Fcp	248 (1.71)	248 (1.71)	363 (2.50)
Fb	602 (4.15)	734 (5.06)	2152 (14.84)

Note: 1. Values are factored allowable stresses.

Table B7-1: Allowable Stresses - psi (MPa)

APPENDIX 'C'

CALCULATION OF TRUSS LOADING and MEMBER FORCES:

BASED ON: CSA CAN3 - 086 - M84 ,

TPIC - January, 1988 , and,

CSA - S307 - M1980,

CWC - Datafile WC-3 , 1987,

TDM - 1980.

SUBSECTION C1 - TEST TRUSS:

SPAN : 7310 mm (24 ft.)

STYLE : Fink (see Dwg # 24-1)

SLOPE : 4" rise in 12" run, 1:3

PITCH : 1/6

LOADING : Complies with;

1. NBCC - subsection 9.4.2
2. TPIC - 88 - section 3
3. CSA-S307-M1980 - section 6

CONSTRAINTS : As per TPIC - 88, pp. 21-23

1. Ground snow load not less than 1.7 kPa, or, 4.2 kPa
2. Top or bottom chord size not to exceed 38 x 140 mm. (2" x 6")
3. Top slope between 1:2.3 and 1:4.8
4. Spacing 610 mm (24") centre to centre
5. Total dead load not greater than 0.5 kPa

SUBSECTION C2 - TEST LOADING:

In conjunction with the test procedures outlined in CSA S307-M1980, the truss is designed to withstand appropriate snow loading, a dead load of 0.25 kPa on the top chord and 0.5 kPa on the bottom chord. Although the truss is to be used in Sierra Leone the writer was interested in snow loading, as future investigations will consider the feasibility of the metal connecting system for truss manufacture and use in developing regions of Canada.

In order to produce the worst case for truss loading the procedures outlined in the Timber Design Manual, (pages 40-44), were followed for balanced (Sb) and unbalanced (Su) snow loading conditions, in conjunction with the appropriate coefficients Xu, Xb and Xe .

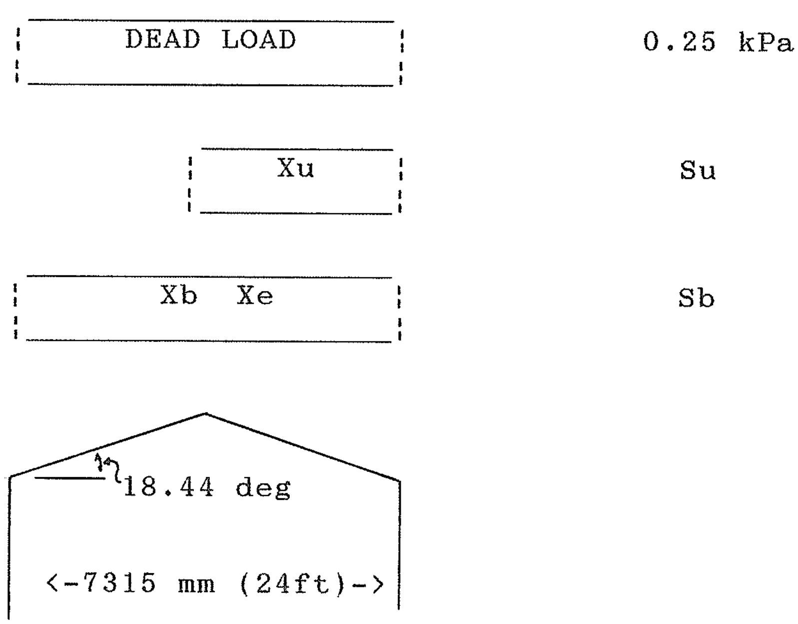


Fig. C2-1: Snow and Dead Loading on Truss

(NOTE: Throughout Appendix 'C' and 'D' metric and imperial units have been used in calculations, with major tables containing both. The reason for this was to provide examples for readers in Sierra Leone, of how one shifts between the two conventions.)

BALANCED SNOW LOAD:

From Fig. C2-1 the balanced snow load becomes;

EQN C1 - $Sb = Sg * Xb * Xe$,

where:

- Sb = Balanced snow load in Kpa, (kN/m^2)
- Sg = Ground snow load in kPa, (kN/m^2)
- Xb = Snow loading coefficient
- Xe = Snow loading coefficient

Using a ground snow loading of 1.7 kPa equation C1 yields;

$Sb = 1.7 * 0.8 * 0.75$

$Sb = 1.02$ kPa , thus,

Balanced snow loading-Sb = 1.02 kPa (21.3 psf)

UNBALANCED SNOW LOADING:

Due to the effects of wind, in conjunction with the orientation of a structure, snow may accumulate in a pattern other than evenly distributed over the roof surface. To account for this condition the following equation is used.

EQN C2 - $Su = Sg * Xu$,

where:

Su = Unbalanced snow load in kPa , (kN/m^2)

Sg = Ground snow load in kPa, (kN/m^2)

Xu = Snow loading coeffcient

Inputting the appropriate values equation C2 becomes;

$$Su = 1.7 * (5 + 18.44 / 25)$$

$$Su = 1.7 * 0.94$$

$$Su = 1.6 \text{ kPa, therefore,}$$

Unbalanced snow load-Su = 1.6 kPa (33.42 psf)

SUBSECTION C3 - TOTAL TRUSS LOAD:

The total truss loading is made up of the snow load and the dead load of the truss material and ceiling/roofing covering. For purposes of truss testing CSA-S307-M1980, section 6 presents guidelines for dead loads;

MEMBER	MINIMUM LOADING - kPa (psf)
Top chord	0.25 (5.22)
Bottom chord	0.50 (10.44)

The total load on the top chord of the truss will then be a summation of the dead load and the unbalanced snow load. The unbalanced snow load is used as it produces the higher loading value. Equation C3 relates these parameters as follows.

EQN C3 - $TL = Su + DL$,

where:

TL	=	Total load on truss top chord, kPa (kN/m^2)
Su	=	Unbalanced snow load, kPa (kN/m^2)
DL	=	Dead load, kPa (kN/m^2)

Inputting values into equation C3 yields;

$$TL = 1.6 + 0.25$$

$$TL = 1.85 \text{ kPa} \quad , \quad (38.64 \text{ psf})$$

NOTE: The value of 0.25 kPa for the dead load represents the weight of the truss, asphalt shingles and 12 mm plywood based on the following material weights:

Truss weight: (after Urquhart & O'Rourke, p.51)

w = [0.5(1 + 0.1 * span) / 20.8854] = 0.082 kPa

Asphalt shingles: (based on 210# per square)

w = (210/100) * 0.04788 = 0.100 kPa

Sheathing:(based on 12mm plywood)

w = 12 * 0.006 kPa/mm = 0.072 kPa

Total load = 0.254 kPa

A dead load of 0.25 kPa was, therefore, used in the calculations. It should also be noted here that this dead load would exceed that expected in Sierra Leone. The roof covering used is normally 28 - 30 ga. corrugated metal sheeting with a weight of approximately 0.030 kPa. Including the truss this would result in a load of 0.112 kPa, 44% of the design dead load.

The loading on the bottom chord was taken as 0.5 kPa as the only loading anticipated would be plasterboard (Gyproc) @ 0.100 kPa; 10 mm plywood @ 0.060 kPa; or, in the case of Sierra Leone, woven mats of neglible weight.

PANEL LOAD:

In determining the total weight that the truss will have to support the loading is broken down into the 'panel load'. A Fink truss has four panels which each have a horizontal projection length of the span/4, or, 1819 mm (6 ft) measured along the span or bottom chord. This concept can be seen in Dwg.# E24-1, where the horizontal projection is shown on panel DEK. The panel load is calculated based on the following.

PANEL LOAD - TOP CHORD:

Horizontal Projection of Panel	:	1819 mm
Truss Spacing	:	610 mm
Horizontal Area / Panel	:	(1819 * 610)/10^6 = 1.12 m^2
Loading / Panel	:	1.85 kN/m^2 * 1.12 m^2 = 2.07 kN (465 lbs)
Total Top Chord Load	:	4 panels * 2.07 kN = 8.28 kN (1861 lbs)

PANEL LOAD - BOTTOM CHORD

Horizontal Projection of Panel	:	2438 mm	
Truss Spacing	:	610 mm	
Horizontal Area / Panel	:	(2438 * 610)/10^6	= 1.49 m^2
Loading / Panel	:	0.5 kN/m^2 * 1.49 m^2	= <u>0.74 kN</u> (166 lbs)
Total Bottom Chord Load	:	3 panels * 0.74 kN	= <u>2.22 kN</u> (499 lbs)

<u>TOTAL TRUSS LOAD:</u>

TOTAL TRUSS LOAD = TOP CHORD LOAD + BOTTOM CHORD LOAD

TTL = 8.28 kN + 2.22 kN

Therefore,

> Total Load/Truss- TTL = 10.5 kN (2360 lbs)

Fig. C3-1 illustrates the loading on a 24' truss with the loads concentrated at the joints.

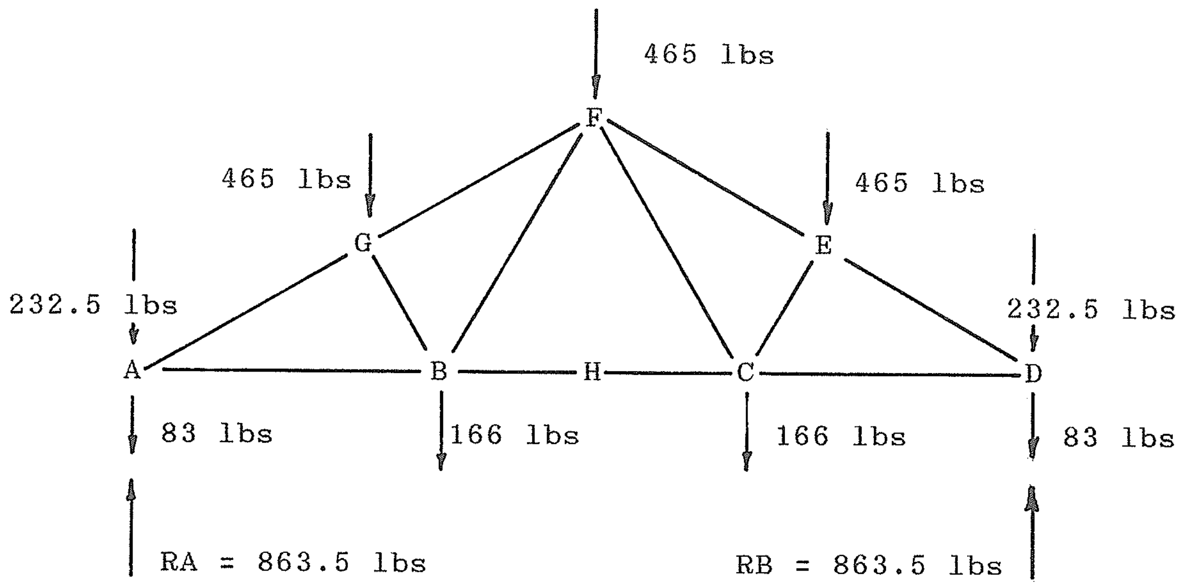


Fig. C3-1: Truss Loading - 24'- 0"

SUBSECTION C4 - ANALYTICAL DETERMINATION OF MEMBER FORCES:

Using standard analytical analysis the compression and tension forces in the truss members were calculated at each joint. The sign convention used was as follows.

(-) = compressive force

(+) = tensile force

Due to the symmetry of the truss, joints A, B and G provide all of the member forces with joint F acting as a calculation check.

JOINT A:

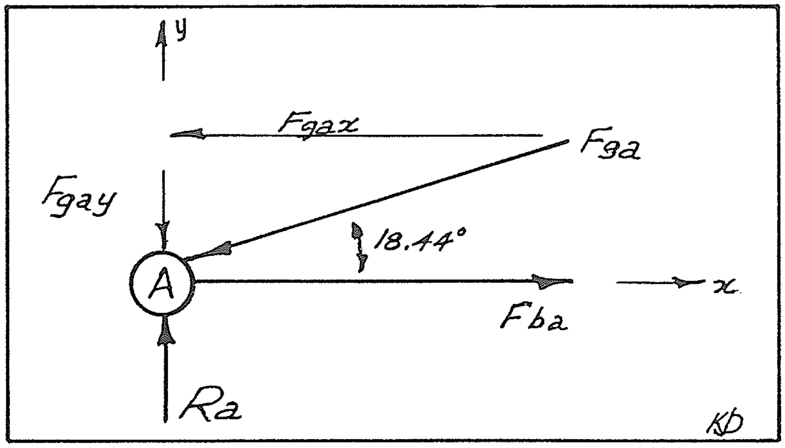


Fig. C4-1: Joint A Member Forces

Fy = 0 => Ra - Fgay = 0

Ra - Fga sin 18.44 = 0

Ra/sin18.44 = 3.85/0.3163 = Fga = 12.17

Fga = (-) 12.17 kN

Fx = 0 => Fba - Fgax = 0

Fba - Fga cos 18.44 = 0

12.17 * (0.9486) = Fba = 11.54

Fba = (+) 11.54 kN

JOINT G:

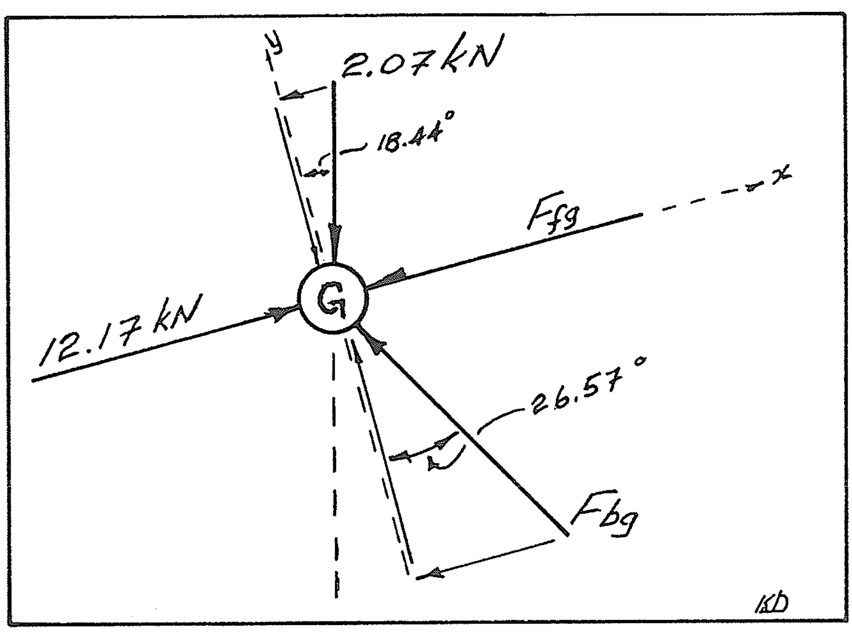


Fig. C4-2: Joint G Member Forces

Fy = 0 => Fbgx - 2.07 cos 18.44 = 0

Fbg cos 26.57 - 2.07 cos 18.44 = 0

2.07cos 18.44/cos 26.57 = Fbg = 2.20

Fbg = (-) 2.20 kN

Fx = 0 => 12.17 - Ffg - 2.07sin 18.44 - 2.2sin 216.57

12.17 - 0.6548 - 0.9840 = Ffg = 10.53

Ffg = (-) 10.53 kN

JOINT B:

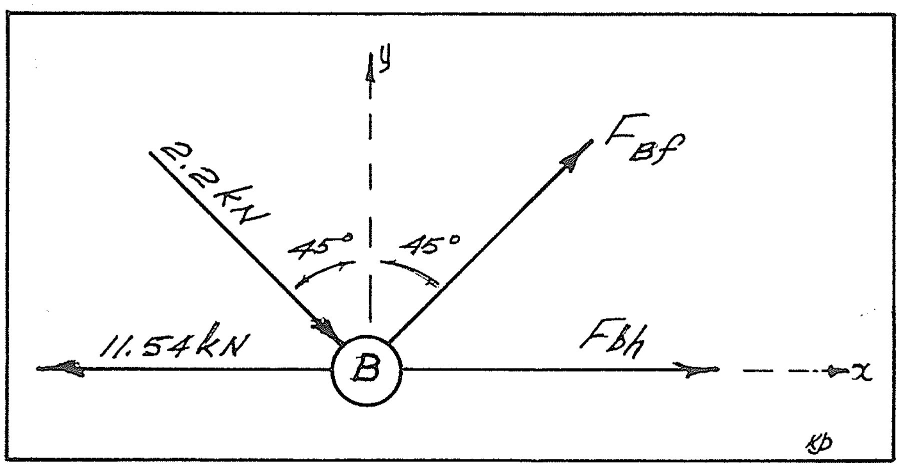


Fig. C4-3: Joint B Member Forces

Fy = 0 => Fbfy - 0.74 - 2.2 cos 45 = 0

Fbfcos 45 - 0.74 - 2.2cos 45 = 0

[0.74 - 2.2(0.7071)]/(0.7071) = Fbf = 3.25

Fbf = (+) 3.25 kN

Fx = 0 => Fbh + 3.25sin 45 + 2.2sin 45 - 11.54 = 0

11.54 - (3.25(0.7071) + 2.2(0.7071) = Fbh = 0

Fbh = (+) 7.68 kN

JOINT F:

Joint F is checked to see if the calculated member forces balance, thus indicating computations are correct.

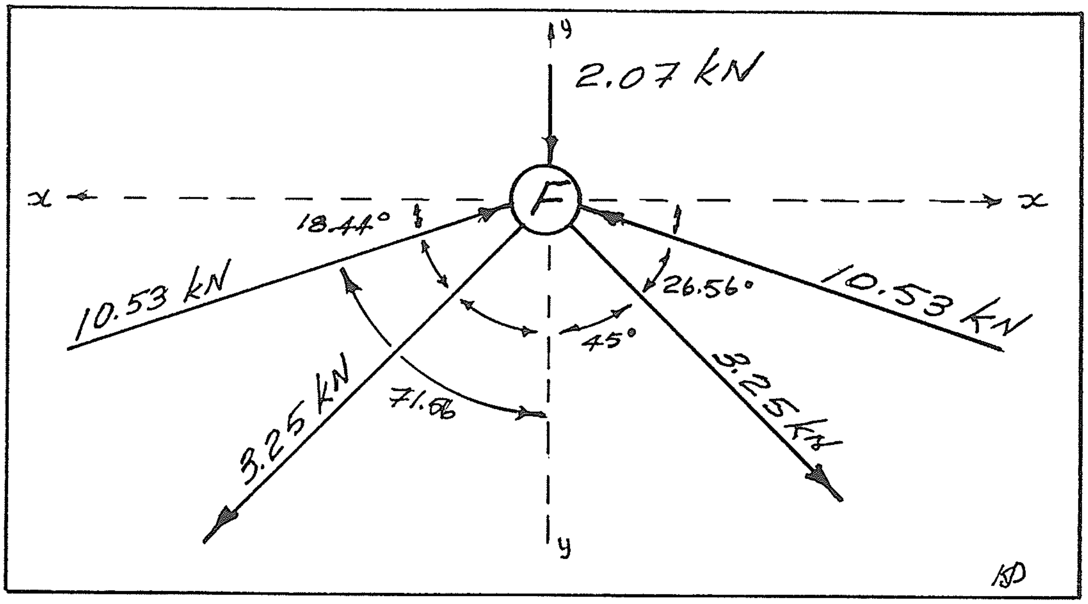


Fig. C4-4: Joint F Member Forces

Fy = 0 => [(10.53 cos 71.56)*2] -

[2.07 + (2*(3.25 cos 45))] = ?

(3.33 * 2) - [(2.298 * 2) + 2.07] = ?

6.66 - 6.66 = 0 ,

therefore, the calculated member forces check out to be correct. The following table summarizes the forces and the required cross sectional area of each member.

MEMBER	FORCE-F (lbs) (N)	TENSION or COMPRESSION (T/C)	ALLOWABLE STRESS-f (psi) (MPa)	REQUIRED AREA-A (in^2) (mm^2)
AG	2,736 (12,170)	C	1482 (10.22)	1.85 (1191)
AB	2,594 (11,540)	T	1620 (11.17)	1.60 (1033)
GF	2,367 (10,530)	C	1482 (10.22)	1.60 (1030)
GB	495 (2,200)	C	1482 (10.22)	0.33 (215)
BF	731 (3,250)	T	1620 (11.17)	0.45 (291)
BH	1,727 (7,680)	T	1620 (11.17)	1.07 (688)

Notes:
1. Using working stress design the x-sectional area is determined by, A = F/f
2. The allowable stresses are based on Table B7-1, CWC - WD-2.
3. Jig #1 = 2.50" dia. = 3160 mm^2.
4. Jig #2 = 3.25" dia. = 5354 mm^2.

Table C4-1: Summary of Member Force and Area

SUBSECTION C5 - CHECK OF ALLOWABLE STRESSES:

Now that the member forces have been determined for the previous loading conditions on a 24'- 0" span truss the size of the truss members are chosen, section moduli computed and the resulting stresses compared to the allowable stresses.

Based on the information in Table C4-1 and given the fact that only two jig sizes are available for connector forming, Jig #1 has been chosen for the truss construction. The following truss member properties are associated with Jig #1 ;

Diameter	:	63.5	mm	(2.50 in)
Section Modulus	:	25,137	mm^3	(1.54 in^3)
X-Sectional Area	:	3160	mm^2	(4.90 in^2)

TENSILE STRESS CHECK:

Largest tensile load	:	11,540 N	
Tensile Stress (P/A) , ft	:	3.65 MPa	
Allowable Tensile Stress - Ft	:	11.17 MPa	
Ratio - ft/Ft	:	0.33	< 1.0 (ok)

COMPRESSIVE STRESS CHECK:

Largest compressive force	:	12,170 N	
Compressive stress (P/A), fc	:	3.85 MPa	
Allowable Compressive Stress-Fc	:	10.22 MPa	
Ratio - fc/Fc	:	0.38	< 1.0 (ok)

STRESS DUE TO BENDING MOMENT:

In addition to the stresses created by axial loads of compression and tension on the truss members the bending stress, fb, must also be examined. This stress is associated with the uniform loading on the top and bottom chord and is related to the bending moment and section modulus in the following manner. (working stress design)

$$fb = M / S ,$$

where:

fb = Stress at extreme fibre, MPa ,(N/mm^2)

M = Bending moment, N-mm

S = Section modulus, mm^3

BENDING MOMENT - TOP CHORD:

In keeping with TPIC, section 4.1.2.2, p.12, the moment created by the design loading is as follows;

TOP CHORD MOMENT

WL^2/10

where:

W = Design load,kN/m

L = (L2 + L3)/2

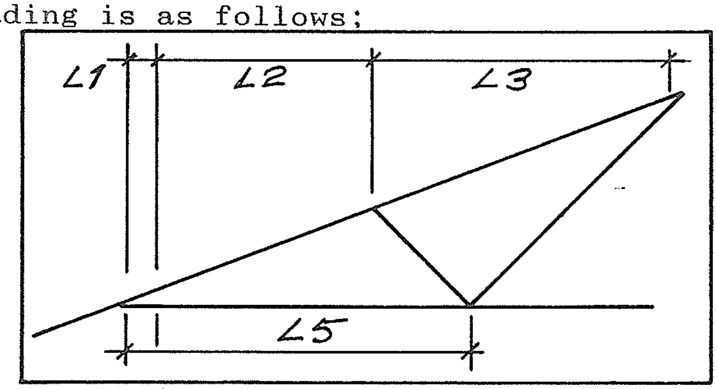


Fig.C5-1 - Design Lengths

From Dwq. E24-1 the scarf length, L1, is taken to be 406 mm, (16 in). The assumption used here is that this represents the length of the metal connector and that bending is not considered to take place inside. The length, L, used in the bending moment calculation is, therefore;

Span/2 = 7310/2 = 3655 mm

Span/2 - L1 = 3655 - 405 = 3250 mm

L3 = 7310/4 = 1828 mm

L2 = 1828 - 406 = 1422 mm

L = (1828 + 1422)/2 = 1625 mm , (5'-4")

Having determined L, the bending moment is calculated;

$M = WL^2 / 10$

$M = [(1.85*0.610)*(1.625^2)] / 10$

$M = 0.298$ kN-m , (219.7 ft-lb)

Using a member size corresponding to jig #1, the resulting stress is calculated.

$fb = M / S$

$fb = (0.298 * 10^6) / 25{,}137$ {N-mm / mm^3}

$fb = 11.85$ MPa < Fb

BOTTOM CHORD MOMENT:

The bending moment created on the bottom chord is due to a 0.5 kN/m^2 uniform loading. The resulting moment is therefore;

$M = WL5^2 / 8$, (see Fig.C5-1 for L5)

$M = [(0.5*0.610)*(((7.310-0.200)/3)^2)]/8$

$M = 0.21$ kN-m , (154.88 ft-lb) .

Bending stress becomes;

$fb = M / S$

$fb = (0.21 * 10^6) / 25{,}137$

$fb = 8.35$ MPa < Fb .

COMBINED AXIAL LOAD AND BENDING STRESSES:

The combined effect of the bending moments and the axial loads are investigated as per the standards outlined in subsection B5 and B6 , Appendix B.

LOAD T/C	CHORD	RATIO ft/Ft	RATIO fb/Fb	RATIO fc/Fc	SUMMATION
T	Bottom	0.33	0.56	--	0.89
C	Top	--	0.79	0.38	1.17

Table C5-1: Bending and Axial Stress

POST STRUCTURAL TEST COMMENT:

It has been seen that the member of concern was the top chord member AGF, which, in both trusses was responsible for the collapse of the structure. The failure of the top chord was, however, a result of out-of-plane buckling which caused the flexural stresses to be exceeded. Observation of loads in excess of 800 pounds did not indicate that in-plane bending between panel points was a concern for the loads that will be encountered in Sierra Leone. Tables 5-1 and 5-2 compile the data regarding buckling and flexure as related to pole diameters.

STRESS PERPENDICULAR TO THE GRAIN:

The stress created by the loading at joint A and D is investigated to compare it with the allowable stress Fcp.

Loading	:	3.85 kN	= 3,850 N
Bearing Area	:	100 mm x 20 mm	= 2000 mm^2
Allowable Stress	:	2.50 MPa	= 2.50 N/mm^2
Stress due to Load	:	3,850/2000	= 1.925 N/mm^2
Ratio fcp/Fcp	:	1.925/2.50	= 0.77

As the ratio of the stress due to loading to the allowable stress is less than unity (0.77 < 1.0), the standard has been met for this parameter.

BENDING MOMENT ON A TWO-SPAN CONTINUOUS BEAM:

In section 4.4.1 the test loading of the truss was discussed. The following calculations and information form the basis for Table 4-2.

The Three-moment-equation is stated as follows:

$$M1L1 + 2M2(L1 + L2) + M3L2 = - P1L1^2(k1 - k1^3) - P2L2^2(2k2 - 3k2^2 + k1^3).$$

Since the ends of the beam are pinned the moments M1 and M3 become equal to zero; M1 = M3 = 0.

Solving for M2 yields;

EQN C4 : $\underline{M2 = 15/32\ PL}$,

where L is the distance from the pinned end to the centre support.

In order to compare the concentrated load moment to the uniform load moment the equivalent loading is calculated for the six point loads;

$w = 6P/2L$, which yields;

EQN C5 : $\underline{P = wL/3}$

Substituting this value of P into EQN C4 ;

$M2 = (wL/3)\ (15/32L)$, which becomes;

EQN C6 : $\underline{M2 = 5/32\ wL^2}$

The value for M2 in equation C6 is now compared to the same bending moment M2 created by the uniform loading.

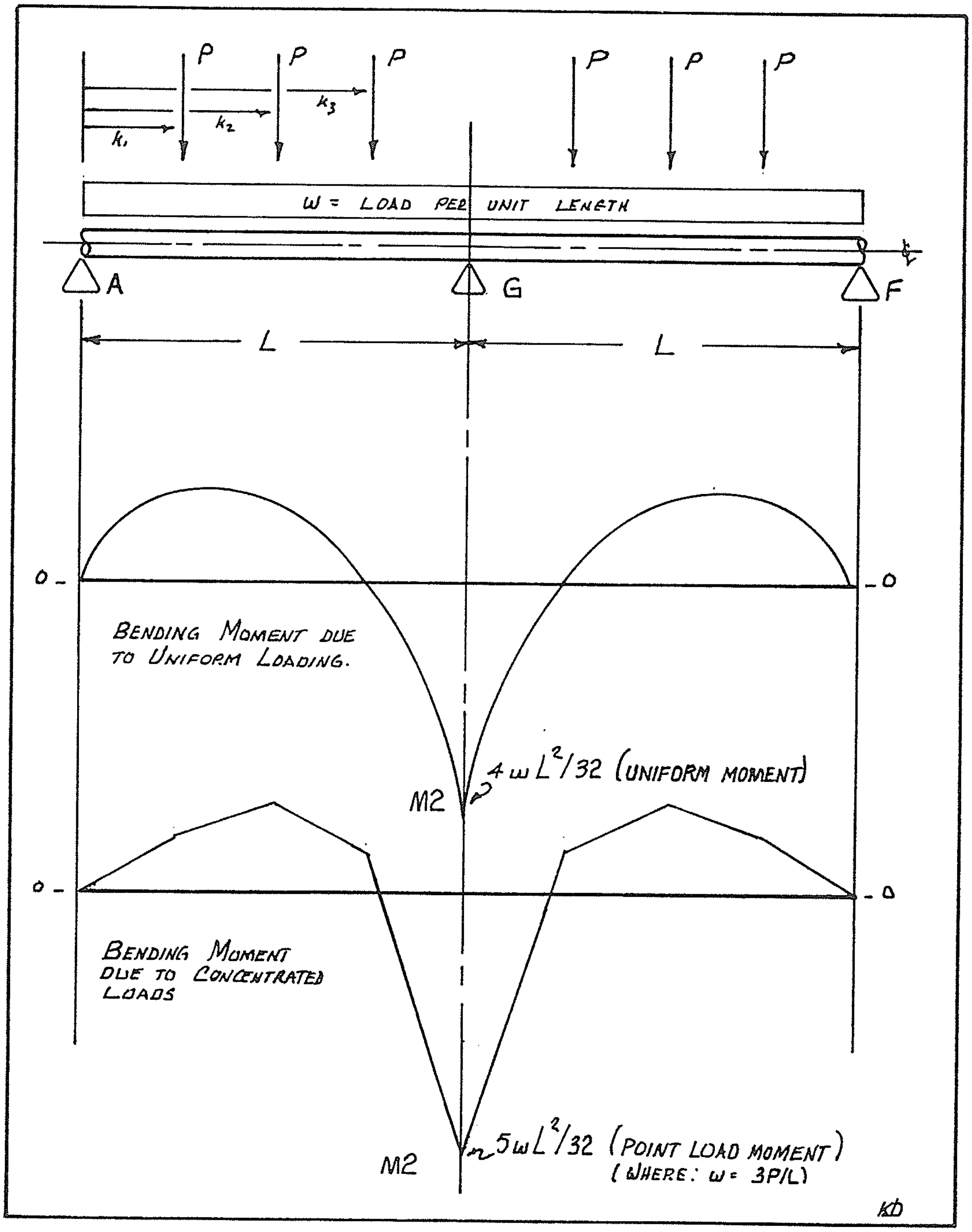


Fig. C5-2: Continuous Beam Bending Moments

The case of a two span continuous beam subjected to a uniform load is a common situation documented in many texts and appearing in standard tables. For that reason the calculations will not be shown since the result is of the most concern in this instance (see Fig. C5-2).

The moment M2 due to uniform loading is given by;

EQN C7 : $\underline{M2 = wL^2/8}$

A comparision between the point-load and the uniform-load bending moment indicates that the concentrated loads produced a 20% greater moment at the central support, M2. Table 4-2 summarizes the results of calculations.

TRESTLE DEFLECTION:

If the truss is assumed to support a load of 2000 pounds this results in a point load of 1000 pounds on each trestle at midspan. Based on the following details the deflection was calculated;

Trestle material	-	6" x 6" Light column
Trestle span	-	40"
Load at Midspan	-	1000 pounds

A simply supported beam is assumed and the deflection calculated using the well known relationship;

$$<> = PL^3 / 48EI \quad ,$$

where;

<>	=	Deflection in inches
P	=	Concentrated load midspan = 1000 pounds
L	=	Length of span = 40 inches
E	=	Modulus of elasticity = 29,000,000 lbs/in^2
I	=	Moment of Inertia = 41.7 in^4

Substituting these values yields;

<> = 0.0011 inch.

APPENDIX 'D'

CONNECTOR DESIGN:

BASED ON: CSA - S347 - M1980,
TPIC - January, 1988,
CSA - CAN3 - 086 - M1984,
TDM - 1980,
CWC - Datafiles, 1986
CISC - Limit States Design, Steel

SUBSECTION D1 - CONNECTOR MATERIAL:

The material used for the connecting system of the truss can potentially come from a variety of sources, however, for the purposes of testing galvinized sheet metal was used. The sheet steel conforms to ASTM A36, having the following properties; (Meets or exceeds CAN3-086-M84, clause 10.7.1.3)

Yield point	:	250 MPa ,	36 ksi
Tensile strength	:	400 MPa ,	58 ksi
Elongation in 200 mm	:	20 %	
Elongation in 50 mm	:	23 %	

Although the material that will be used in field practice would likely be automotive sheet steel, the properties of autobody panels would meet or exceed the minimum strength parameters for ASTM A36. It is felt, therefore, that testing the connecting system using galvinized sheet metal for the manufacture of the connectors is practical.

Using the yield strength of the steel the capactiy of the connectors can be determined. Three gauges of steel have been selected, 24 ga, 26 ga and 28 ga. Their yield resistance is listed below based on the following relationship;

$$L = @ \; AnFy \quad ,$$

where:

L = Load capacity , N/mm , (lb/in)

@ = Performance factor, 0.9. (Adams et al,p.44)

An = Net area of material, mm * unit width

Fy = Yield stress, MPa , (psi)

The yield strength of the three gauges can now be determined as follows. (United States Standard Gauge Size)

24 GAUGE : Thickness - 0.607 mm , (0.0239 in)

L = 0.9 * (0.607) * 250 {mm*N/mm^2}

L = 136 N/mm , (776 lb/in)

26 GAUGE : Thickness - 0.455 mm , (0.0179 in)

L = 0.9 * (0.455) * 250

L = 102 N/mm , (585 lb/in)

28 GAUGE : Thickness - 0.378 mm , (0.0149 in)

L = 0.9 * (0.378) * 250

L = 85 N/mm , (486 lb/in)

Based on the previous calculations Table D1-1 presents the load resistance values for the two jig sizes and the three gauge materials.

JIG No.	MATERIAL 24 ga	26 ga	28 ga
1	27 200 (6092)	20 400 (4592)	17 000 (3815)
2	35 300 (7923)	26 520 (5973)	22 100 (4962)

Note: 1. Jig #1 - 64 mm dia., perimeter = 200 mm
2. Jig #2 - 83 mm dia., perimeter = 260 mm

Table D1-1: Connector Load Resistance Values,N (lbs)

From the information in Table D1-1 it can be seen that, theoretically, connectors made from any of the three gauges should be able to resist the axial forces that are generated in the truss.

SUBSECTION D2 - CONNECTOR NAILING:

The allowable load per nail in the connector is given by the following relationship. (CAN3-086-M84, clause 10.8.4)

$$Nl = Nu*nf(Kd*Ksf*Kt)(Jc*Jy*Jp*Je)*Ja*Jb*Jd$$

where:

Nl = Lateral capacity per nail, N

nf = Number of nails in a group

Kd = Duration factor

Ksf = Service factor

Kt = Treatment factor

Jc = Configuration factor

Jy = Side plate factor

Jp = Nail penetration factor

Je = End grain nailing factor

Ja = Toe-nailing factor

Jb = Nail clinching factor

Jd = Diaphragm construction factor

Table D2-1 presents the nail load modification factor values.

LOAD FACTOR	ASSIGNED VALUE	CODE/ SOURCE	CLAUSE No.	REMARKS
Kd	1.00	CAN3	4.4.1.2	Snow loading
Ksf	0.80	CAN3	Table 21	>15% moisture likely
Kt	1.00	CAN3	4.4.3.2.2	No treatment
Jc	1.00	CAN3	Table 47	Normal spacing
Jy	1.25	CAN3	10.8.4	Steel side plates
Ja	na	CAN3	10.8.4.2	Applies to wood plates
Je	1.00	CAN3	10.8.4.3	No end grain nailing
Ja	1.00	CAN3	10.8.4.4	No toe-nailing
Jb	1.00	CAN3	10.8.4.5	No nail clinching
Jd	1.30	CAN3	10.8.4.6	Diaphragm construction

Table D2-1: Nail Load Modification Factors

The load resistance per nail can now be calculated using an initial value for N = 115 N, corresponding to Table 45, CAN3-086-M84, p.163, for N.Aspen using a 1" common wire nail.

Nl = 115*1*(1.15*0.80*1.00)*(1.00*1.25*1.00)*1.0*1.0*1.3

Nl = 115 * 1.495

Nl = 171.9 N/nail , (38 lb/nail)

The use of diaphragm construction techniques cannot be consistently guaranteed in countries such as Sierra Leone for example, thus, if the factor Jd is omitted from the above calculation the value for N becomes;

Nl = 115 * 1.15,

Nl = 132 N/nail , (30 lb/nail). This value will be used in the calculation of connector size, or nails per joint.

Based on the design loads shown in Dwg. 24-1 , the nails required for each connection is shown in Table D2-2.

JOINT	MEMBER	LOAD(kN) (1)	NAILS REQUIRED(nf) (2)
A	AB	+11.54	88
	AG	-12.17	93
B	BG	- 2.20	17
	BF	+ 3.25	25
H	Centre	+ 7.68	58
F	FG	-10.58	80

Notes: 1. (+) = tensile force,(-) = compressive force
2. Nail calculation; Design load/(Load allowed/nail)
eg. AB = (11.54kN*1000N/kN)/132 N/nail
nf = 87.42 ==> 88 nails.

Table D2-2: Nails Required Per Connection

SUBSECTION D3 - CONNECTOR SIZING:

The length of connector required at each joint is calculated based on information contained in Table D2-2 and CAN3-086-M84, Table 47, p.165. As there are two jig sizes the number of rows of nails that can be accomodated in either connector is determined first.

JIG #1:

Diameter	: 63.5 mm , (2.5in.)
Perimeter	: 200 mm , (7.85in.)
Nail Spacing, Parallel to grain	: 20*nail dia = 36mm
Perpendicular to grain	: 10*nail dia = 18 mm

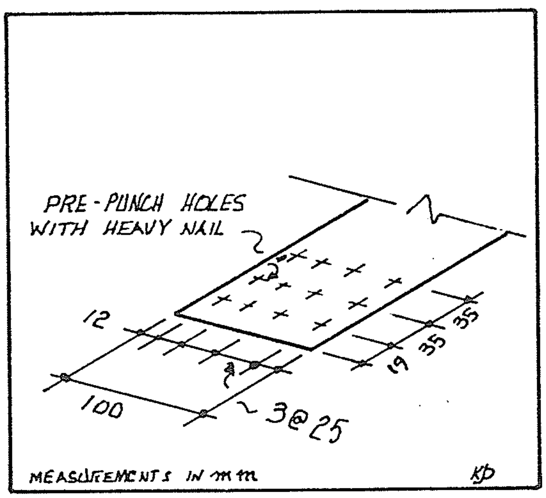


Fig. D3-1: Jig#1 Nail Spacing

Using a standard width of 200 mm, the number of rows available based on a spacing of 20 mm would be; 200/20=10 spaces, allowing for 9 rows of nails with a spacing of 35 mm along the grain. The keep the number of rows even in each connector half, however, 8 rows will be used in the connector sizing calculations. (4 rows per half)

JIG # 2

Diameter	:	83 mm , (3.25 in.)
Perimeter	:	260 mm , (10.21 in.)
Nail spacing, Parallel to grain	:	20*nail dia = 36 mm
Perpendicular to grain	:	10*nail dia = 18 mm

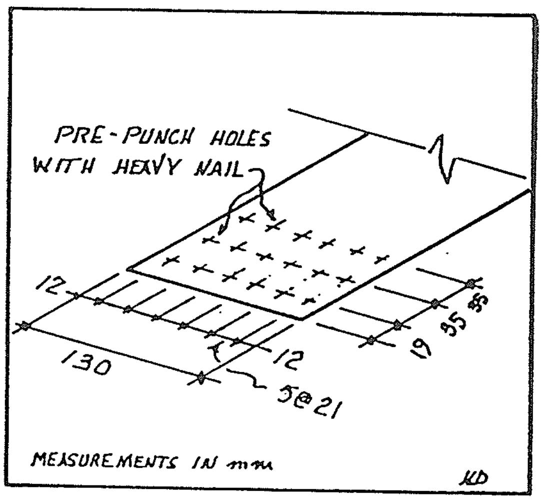


Fig. D3-2: Jig#2 Nail Spacing

Using a standard width of 260 mm, the number of rows available based on a spacing of 20 mm would be; 260/20 = 13 spaces, allowing for 12 rows of nails with a spacing of 35 mm along the grain.

CALCULATION OF CONNECTOR LENGTH:

A sample calculation of the determination of connector length is provided, in conjunction with Table D3-1.

Sample Calculation: Joint A - Member AB or DC:

Connector Size	:	Jig #1 - 64 mm dia.	
Member Force	:	11.54 kN	
Required Nailing	:	11,540N/132N/nail	= 88 - 1" nails
Nails per Row	:	88 nails/8rows	= 11 nails/row
Connector Length	:	11 nails*35 mm spacing	= 385 mm
Final Length	:	406 mm , (16 in.)	

Table D3-1 presents a summation of connector size for the various truss joints based on the approach shown in the sample calculation.

JOINT	MEMBER(1)	CONNECTOR LENGTH(2) JIG #1	JIG #2	REMARKS
A	ABH	400 mm (16in)	300 mm (12in)	Bottom Chord
A	AGF	430 mm (17in)	300 mm (12in)	Top Chord
G	BG	150 mm (6in)	na	Web member (3)
B	BF	150 mm (6in)	na	Web member (3)
H	ABH	500 mm (20in)	360 mm (14in)	Bottom chord centre connector

Notes: 1. Joints indicated represent half truss, thus, ABH = DCH, AGF = DEF, BF = CF, and BG = CE.
2. A minimum connector length restriction of 150 mm, or 6 inches was imposed by the author.
3. Web members are limited to 64 mm dia only.

Table D3-1: Connector Lengths - Based on 1" Nail Size

LATERAL FORCE ON JOINT B and G CONNECTORS:

Due to the angle at which members BG and BF meet the top and bottom chords, a small lateral force component acts on the connector. The minimum number of nails required has been calculated as follows.

JOINT G - Member BG :

Force	:	2.2 sin 26.57 deg.	=	0.984 kN
Nails Required (1")	:	984 N/132 N/nail	=	8 nails

JOINT B - Member BG :

Force	:	2.2 sin 45 deg.	=	1.56 kN
Nails Required (1")	:	1560 N/132 N/nail	=	12 nails

JOINT B - Member BF :

Force	:	3.25 sin 45 deg.	=	2.3 kN
Nails Required (1")	:	2300 N/132 N/nail	=	18 nails

The following table contains information on connector lengths if 1.5" common wire nails are used.

JOINT	MEMBER(1)	CONNECTOR LENGTH(2) JIG #1	JIG #2	REMARKS
A	ABH	300 mm (12 in)	200 mm (8 in)	Bottom Chord
A	AGF	330 mm (13 in)	230 mm (9 in)	Top Chord
G	BG	150 mm (6 in)	na	Web Member(3)
B	BF	150 mm (6 in)	na	Web Member(3)
H	Centre	355 mm (14 in)	300 mm (12 in)	Bottom Chord

Notes:
1. Joints indicated represent half truss, thus ABH = DCH, AGF = DEF, BG = CE and BF = CF.
2. A minimum connector length restriction of 150 mm, or 6 inches was imposed by the author.
3. Web members are limited to 64 mm dia only.

Table D3-2: Connector Lengths - Based on 1.5" Nail Size

APPENDIX 'E'

DESIGN DRAWINGS

Contained in this appendix are drawings related to the truss connecting system, as listed below:

Dwg. No. E24-1 - Test Truss - 24'-0" Span;

Dwg. No. E24-2 - 45 Degree Connector;

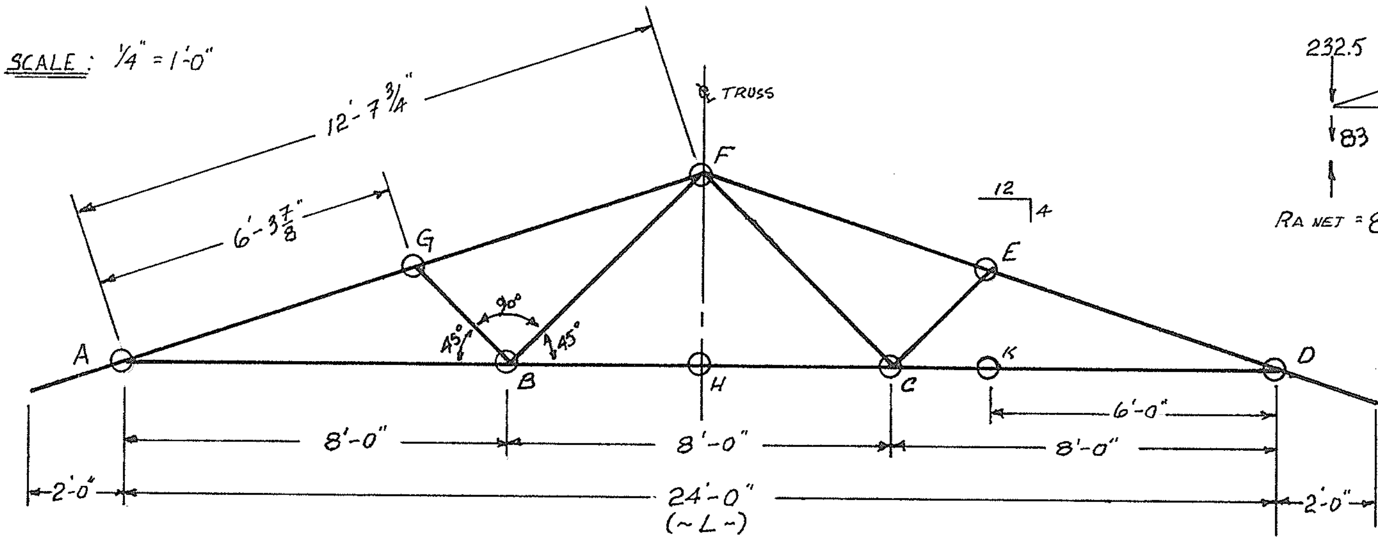
Dwg. No. E24-3 - 60 Degree Connector;

Dwg. No. E24-4 - Peak Connector;

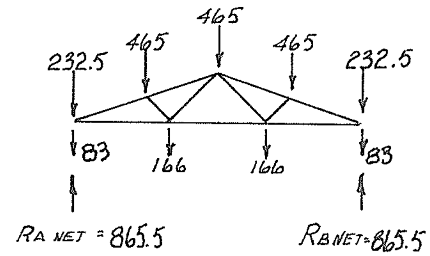
Dwg. No. E24-5 - Heel Connector; and

Dwg. No. E24-6 - Centre Connector.

SCALE: 1/4" = 1'-0"



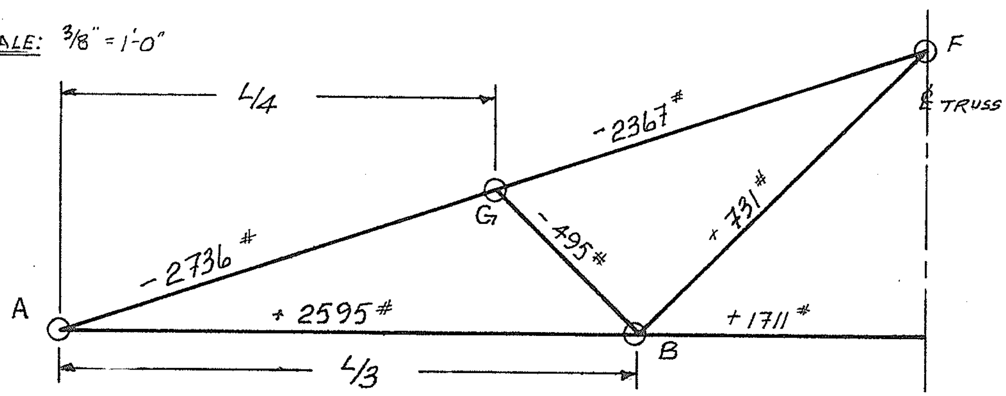
FULL TRUSS



LOADING (lbs)

(NOTE:- FIGURES ARE ROUNDED.
- BASED ON LOADING OF 1.85 kN/m², or 38.6 psf)

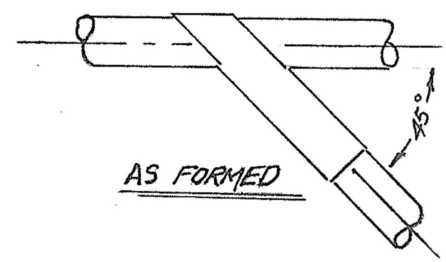
SCALE: 3/8" = 1'-0"

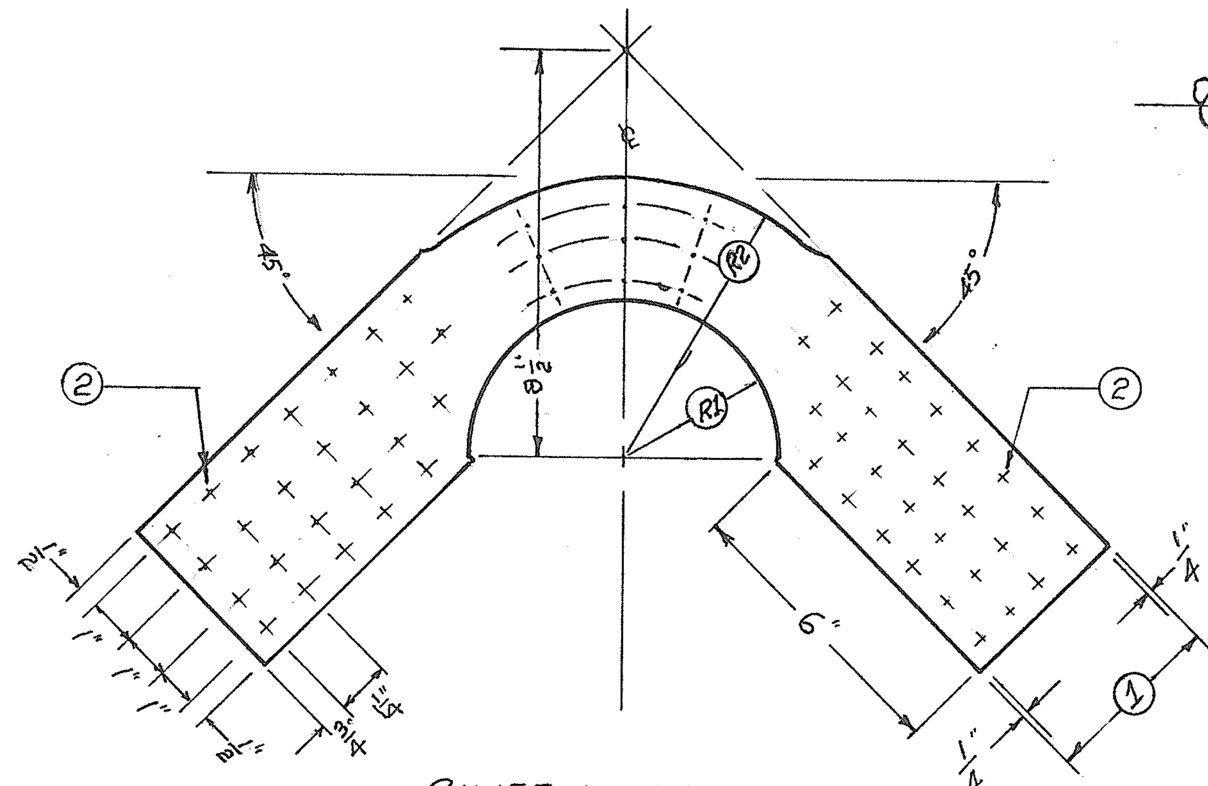


(-) COMPRESSION
(+) TENSION

HALF TRUSS (SHOWING MEMBER FORCES)

TEST TRUSS - 24'-0" SPAN	
DRAWN BY:	K. J. DICK
DATE:	NOV 1, 1988
SCALE:	AS SHOWN
DWG. No. - E24-1	





GENERAL LAYOUT

RADII	JIG 1	JIG 2
R1	3 1/4"	4"
R2	5 3/4"	6 1/2"

USE JIG #1 WITH TIMBER SIZE 2" - 3" ϕ

USE JIG #2 WITH TIMBER SIZE > 3" ϕ

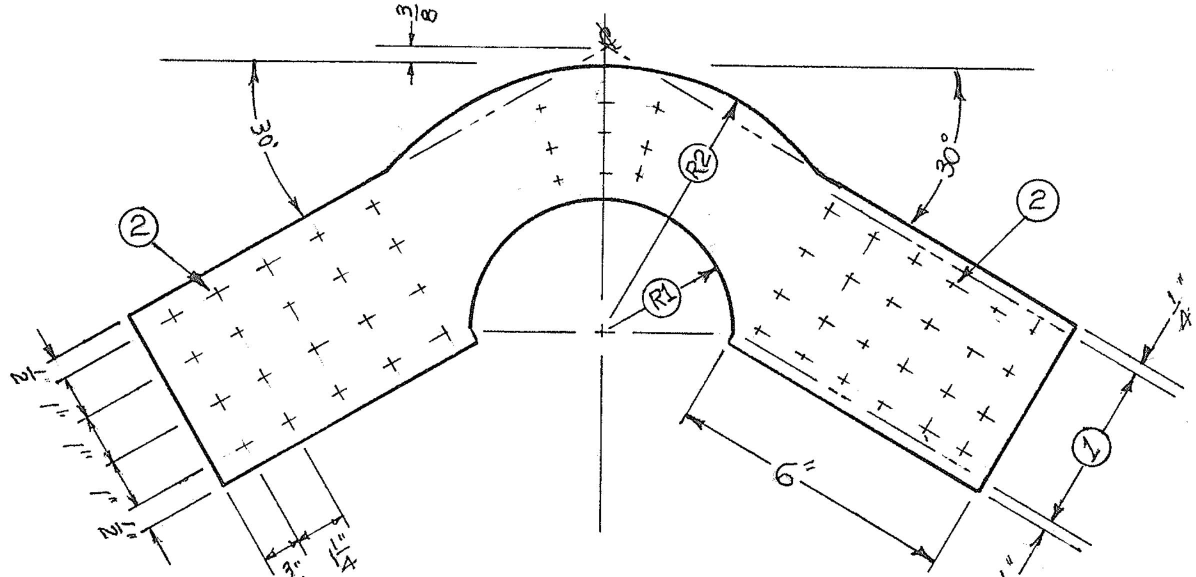
NOTE:

1. DIMENSION VARIES WITH CONNECTOR SIZE:
   JIG#1 = 3 1/2"
   JIG#2 = 5 1/8"
2. HOLES PRE-PUNCHED WITH HEAVY NAIL

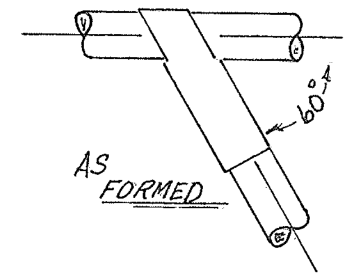
45° CONNECTOR

DRAWN BY: K. J. DICK
DATE: OCT. 1988
SCALE: 1" = 4"

DWG. No. – E24-2



GENERAL LAYOUT



RADII	JIG 1	JIG 2
R1	2 3/4"	3 1/2"
R2	5 1/2"	7 1/4"

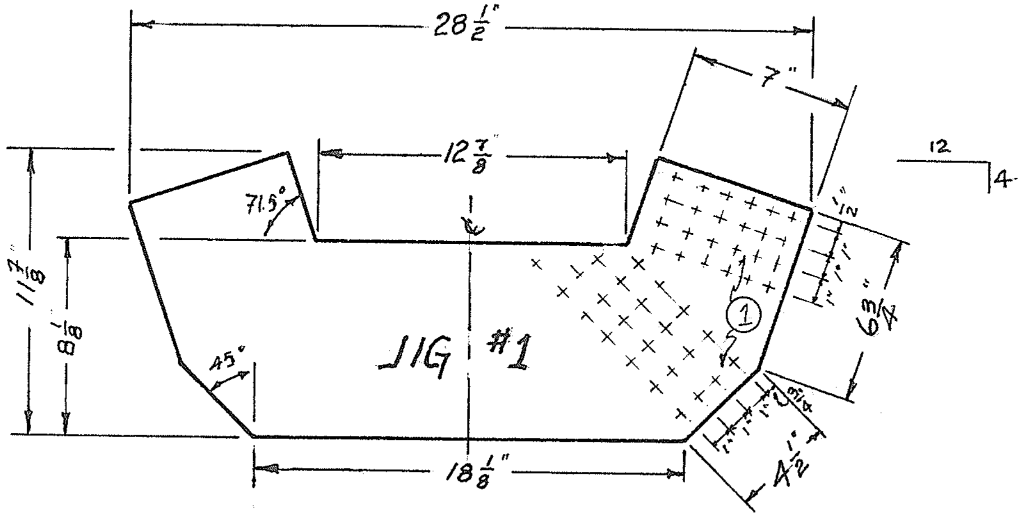
NOTE:

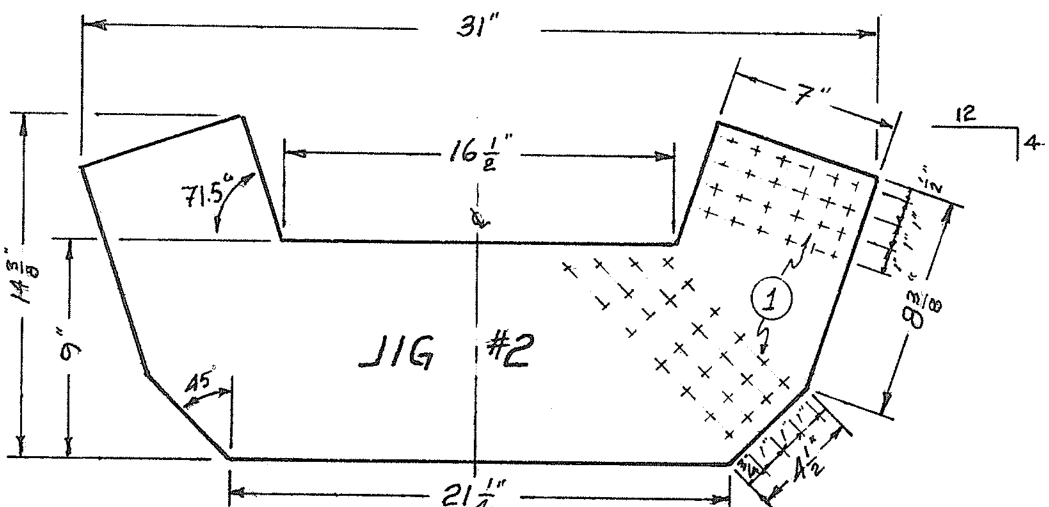
1. DIMENSION VARIES WITH CONNECTOR SIZE:
   JIG #1 = 3 1/2"
   JIG #2 = 5 1/8"
2. HOLES PRE-PUNCHED WITH HEAVY NAIL
3. WITH JIG #1 USE TIMBER 2" - 3" Ø
   WITH JIG #2 USE TIMBER 7 3" Ø

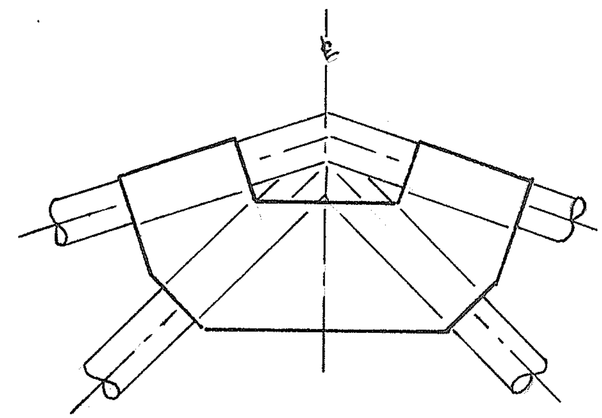
60° CONNECTOR

DRAWN BY: K.J. DICK
DATE: OCT. 1988
SCALE: 1" = 4"

DWG. No. - E24-3







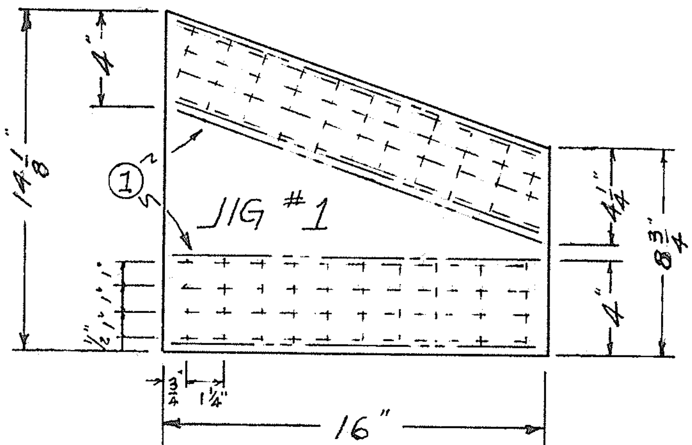
AS FORMED

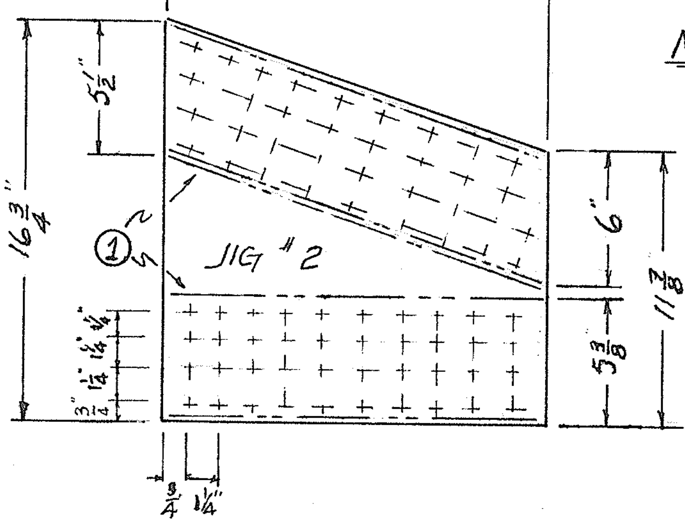
NOTE:
1. PRE-PUNCH HOLES WITH HEAVY NAIL. (REPEAT PATTERN ON OTHER HALF OF CONNECTOR)
2. JIG #1 USE 2" TO 3" ϕ TIMBER
   JIG #2 USE > 3" ϕ

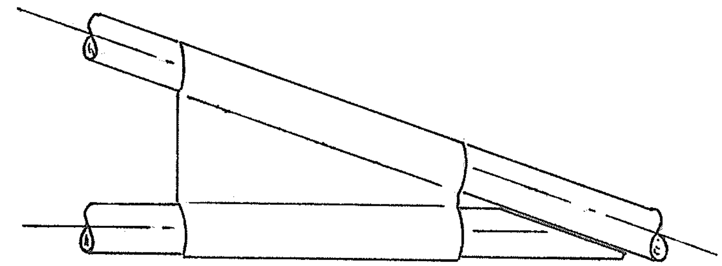
PEAK CONNECTOR

DRAWN BY: K. J. DICK
DATE: OCT. 1988
SCALE: 1" = 8"

Dwg. No. - E24 - 4







AS FORMED

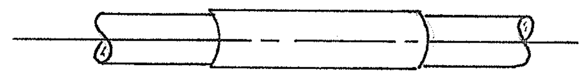
NOTE:

- LEFT & RIGHT CONNECTORS NEEDED.

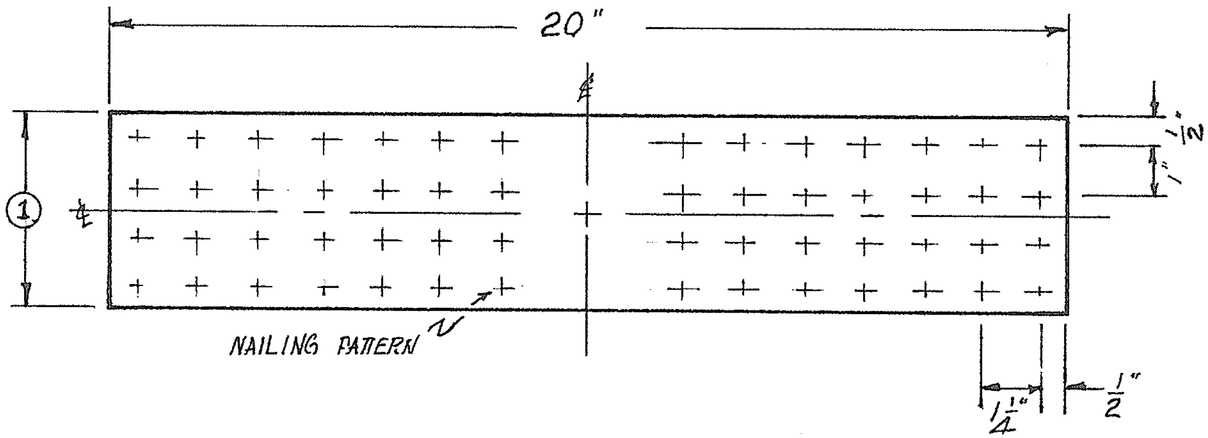
1. PRE-PUNCH HOLES WITH HEAVY NAIL.
2. JIG #1 USE 2"-3" ϕ TIMBER
   JIG #2 USE >3" ϕ TIMBER

HEEL CONNECTOR
DRAWN BY: K. J. DICK
DATE: OCT. 1988
SCALE: 1" = 4"
DWG. No. - E24 - 5

CONNECTOR MATERIAL: 24 Ga. M.S. GALVINIZED SHEET METAL.



AS FORMED



DIM'N / JIG	1
#1	4"
#2	5 5/8"

NOTE: WITH JIG #1 USE 2"-3"ϕ TIMBER
WITH JIG #2 USE >3"ϕ TIMBER

CENTRE CONNECTOR

DRAWN BY: K. J. DICK
DATE: NOV. 1988
SCALE: 1" = 4"

DWG. No. E24-6

APPENDIX 'F'

TENSION/COMPRESSION TEST:
DETERMINATION OF UNIT NAIL LATERAL RESISTANCE

Prior to testing the complete truss, it was deemed useful to examine the resistance of the nails to a lateral or shearing force to estimate their behaviour in-situ. Two nail sizes were examined in combination with a standard test connector. The test procedure and results are outlined below.

SUBSECTION F1 - TEST PROCEDURE AND APPARATUS:

In order to simulate a tension or compression connection two sample pieces of timber were connected as seen in Fig. F1-1. The gap between the two members forced all of the imposed loading to be carried by the nail-connector combination. The connection was considered to have failed in either of two ways:

1. slipping of timber in the connector; and/or
2. buckling of metal connector.

The testing process was as follows:

1. the test specimen was placed in the press frame, diameter of timber was recorded;
2. a load was applied, as steady as possible, by means of

the hand-operated hydraulic jack;

3. when one or both of the failure modes was observed, the reading on the pressure gauge on the hydraulic jack was recorded, along with mode of failure (Note 1, Table F2-1);

4. the force applied at failure was calculated by multiplying the gauge reading (psi) by the area of the ram of the hydraulic jack (in^2), to yield a force in pounds, (Pressure x Area = Force). (see Note 2, Table F2-1);

5. once failure of the specimen had occurred, the load was left on the specimen for an addtional 5-minutes to see if this load could be sustained by the specimen, (see Note 3, Table F2-1);

6. the gauge reading after 5-minutes was recorded and the corresponding load calculated; and

7. the load carrying capacity per nail was calculated based on the sustained 5-minute load, (see Note 4, Table F2-1).

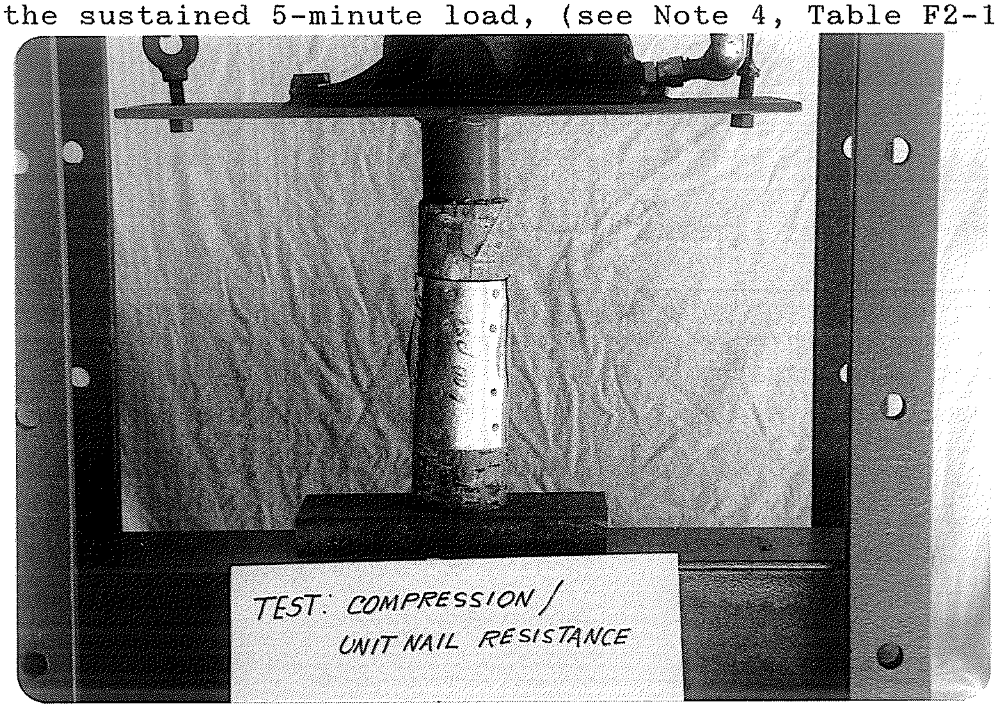


Fig. F1-1: Test Arrangement - Tension/Compression

SUBSECTION F2 - TEST RESULTS:

The results of the testing have been tabulated below.

SPECIMEN	TC-1	TC-2	TC-3	TC-4	TC-5	TC-6
DIA (in)	2.7	2.6	2.3	2.3	2.4	2.3
NAIL (in)	1.5	1.5	1.5	1.0	1.0	1.0
METAL (ga)	24	24	24	24	24	24
# of NAILS	12	12	12	12	12	12
GAUGE RDG AT FAILURE (psi) (1)	600	600	575	350	420	380
LOAD AT FAILURE (lbs) (2)	1062	1062	1018	620	743	672
GAUGE RDG AFTER 5 MIN (psi) (3)	600	600	550	250	375	300
LOAD AFTER 5 MIN (lbs)	1062	1062	975	442	664	531
LOAD/NAIL (lbs) (4)	88	88	81	36	55	44
FAILURE MODE (5)	S	S	S	S	S	S

Notes: 1. Failure as per subsection F1
2. Gauge reading x ram area: psi x 1.77 in^2
3. Load maintained for 5 minutes after initial reading
4. Based on load after 5 min / number of nails
5. B => buckling of connector; S => slippage of timber

Table F2-1: Test Results - 24 Gauge Connectors

APPENDIX 'G'

TEST ON SPECIMENS TO DETERMINE RESISTANCE TO BENDING

DATE : August 28, 1988

LOCATION : Anola, Manitoba

A test apparatus was set up using the press frame (See Fig. G1-1), a short piece of rod welded at right angles to the bolt in the ram and two angle irons as supports at a spacing of 18 inches. Thus, the bending moment was applied to a simply-supported beam with the load at the centre. This implies a bending moment of PL/4 ,

where; P = gauge rdg x 1.77 lbs
L = 18 inches.

The stress at the extreme fibre was then calculated using the relationship; @ = M / S , where:@ = stress (psi)
M = moment (in-lb)
S = section modulus(in3)

Seven samples were tested and the results are tabulated below.

SPECIMEN No. (1)	AVERAGE DIA (2) (in)	GAUGE RDG AT FAILURE (psi)	LOAD AT FAILURE (lbs)	STRESS AT FAILURE (psi)
B1	1.750	200	354	3005
B2	2.500	600	1062	3123
B3	2.000	500	885	5040
B4	2.250	800	1416	5792
B5	2.125	400	708	3382
B6	2.250	500	885	3561
B7	2.000	500	885	5072

Note: 1. All specimens were tested with bark on.
2. Average = sum of end diameters / 2

Table G1-1: Bending Test Results

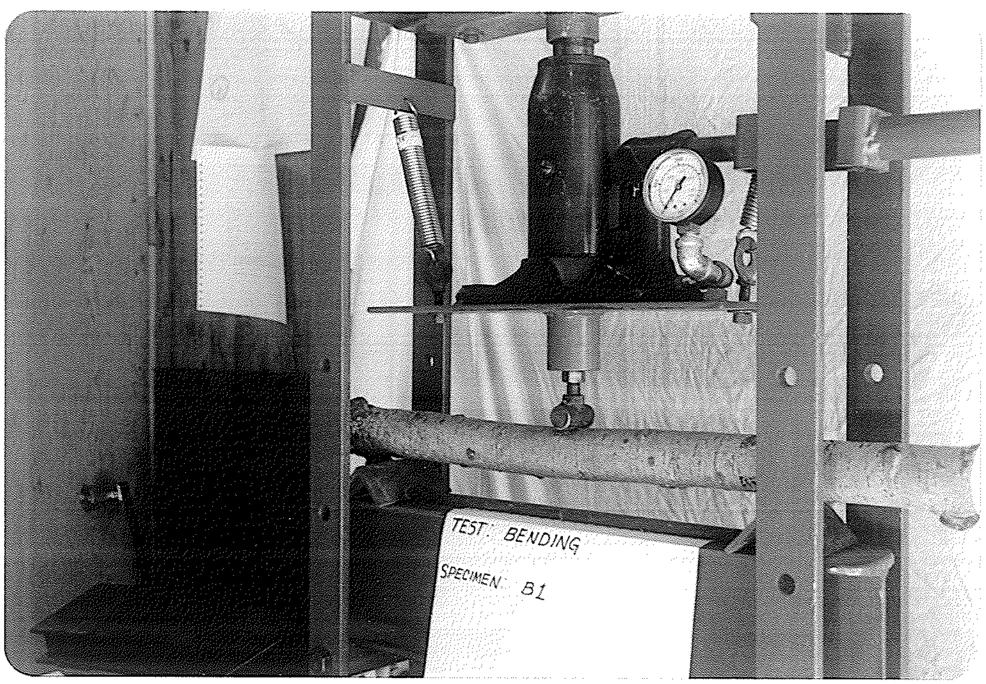


Fig. G1-1: Test Apparatus - Bending

The range of values obtained for the stress at failure appeared a bit higher than was expected. This may be attributed to the criterion used to consider a specimen as having failed. In this case, the poplar specimen had failed if cracking sounds were heard , or splits appeared on the tension side (underside) of the beam. It is suggested, that if the criteria was to consider the beam as failed once the deflection reached a value of the span/180 (18 inches/180 = 0.1 inch) the calculated stress would have been lower.

The stress at failure in Table G1-1 cannot really be considered the allowable stress, but would be closer to the ultimate stress.

APPENDIX 'H'

HYDRAULIC PRESS FRAME
- DESIGN DRAWINGS

Appendix 'H' contains the design drawings and fabricating information for the manufacture of the press frame that was used to produce the metal connectors for the test trusses T1 and T2. Details regarding the material and fabricating techniques employed are contained within the drawings, along with explanatory notes. The following drawings are included:

- Dwg. No. H1-2 - Press Frame ; and
- Dwg. No. H1-3 - Press Frame, General Arrangement.

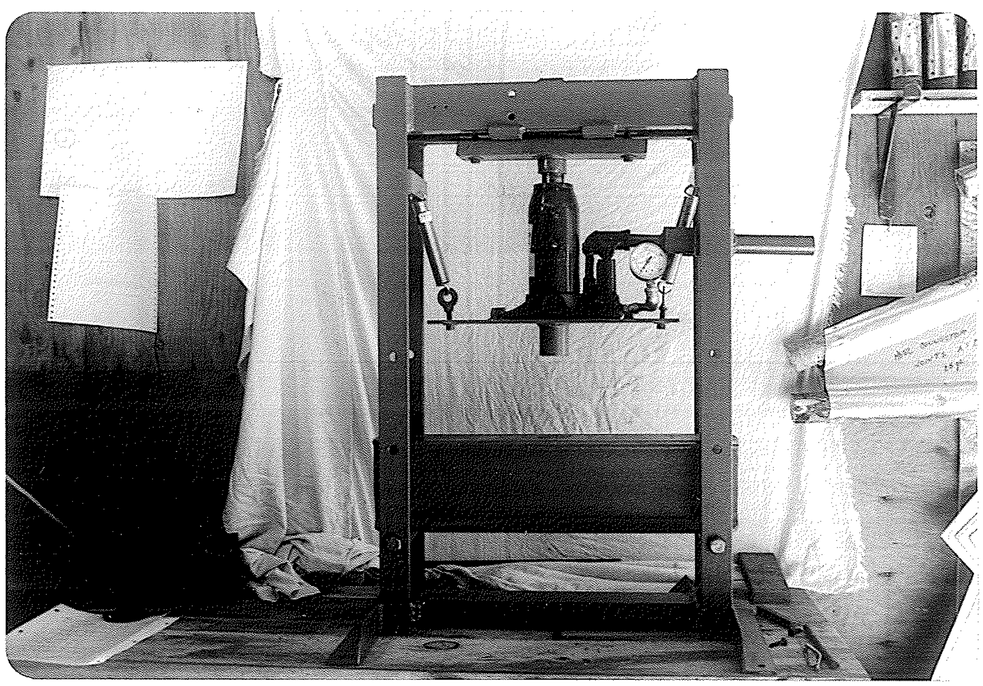
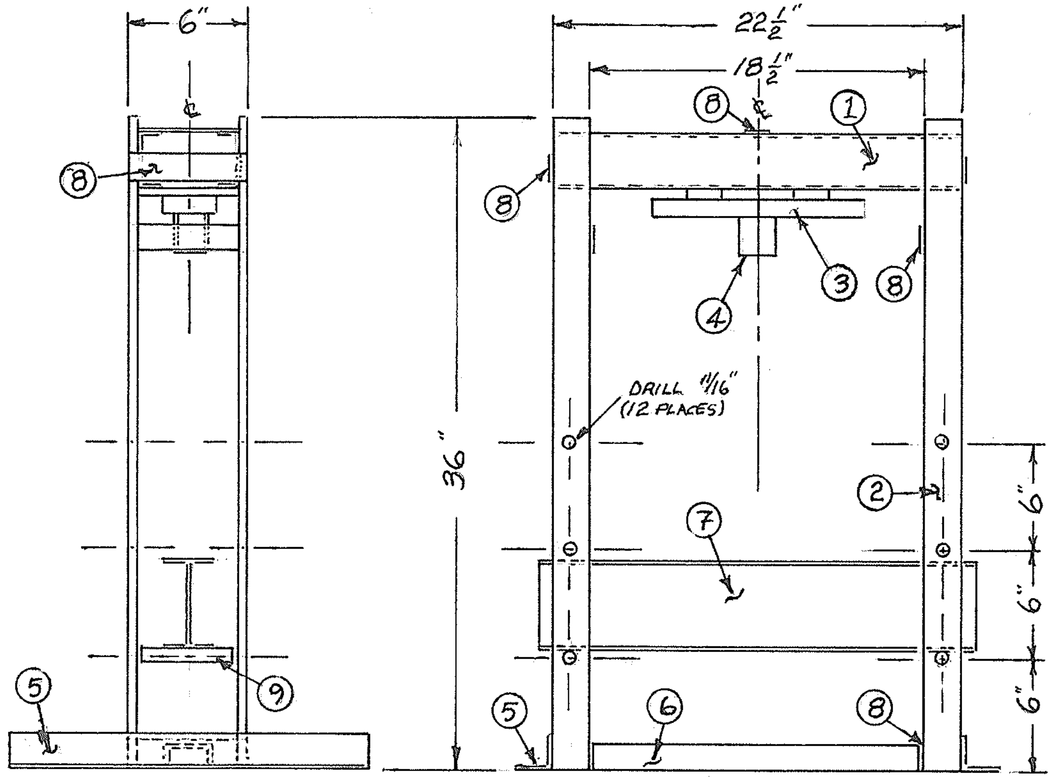


Fig. H1-1: Press Frame - General Arrangement



END VIEW

SIDE VIEW

ITEM No	MATERIAL	No REQ'D
1	3" CHANNEL 22 1/2" LG	2
2	1/2" X 2" X 36" LG. M.S.	4
3	1" X 3" X 12 LG. M.S.	1
4	2" ϕ PIPE - 2" LG.	1
5	2" X 2" X 3/16" ∠ IRON	2
6	3" CHANNEL - 18" LG.	1
7	6" I-BEAM	1
8	1/4" X 1 1/2" X 6" LG. M.S.	6
9	3/4" ϕ PIPE 5" LG.	2

NOTE:

FRAME IS A WELDED STRUCTURE USING 7018 or 6011 ARC WELDING ROD. ITEM 9 IS WELDED TO ITEM 7 AND MOVES AS A UNIT.

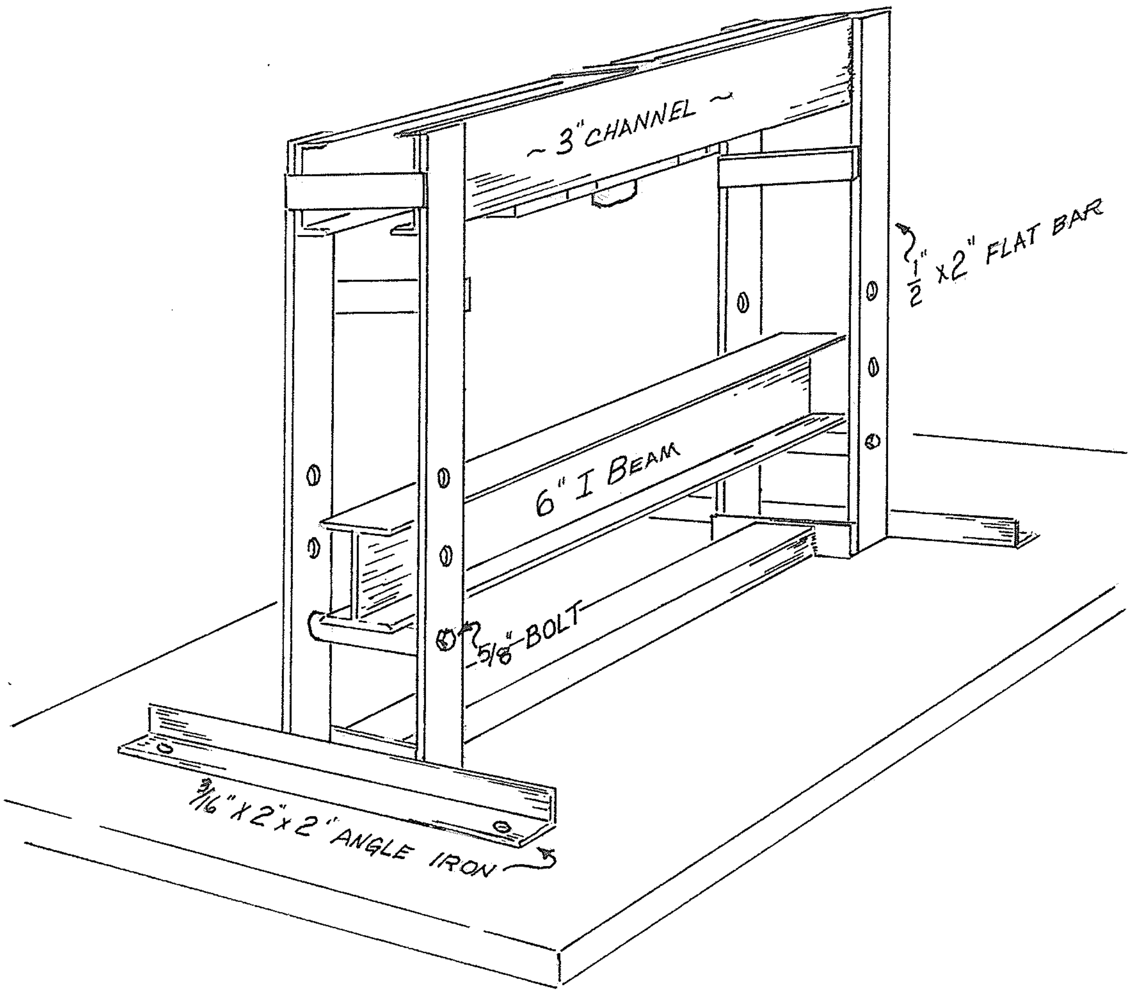
PRESS FRAME

DRAWN BY: K. J. DICK

DATE: OCT. 1988

SCALE: 3/32" = 1"

DWG. No. - H1-2



PRESS FRAME - GEN. ARRG'T.	
DRAWN BY:	K. J. DICK
DATE:	OCT. 1988
SCALE:	NTS
DWG. No. - H1-3	

APPENDIX 'I'

FORMING JIGS

- DESIGN DRAWINGS

The basic drawings and photographs pertinent to the manufacture of Jig #1 and Jig #2 are contained in this appendix. All of the information regarding material and fabrication procedure is presented within the drawing.

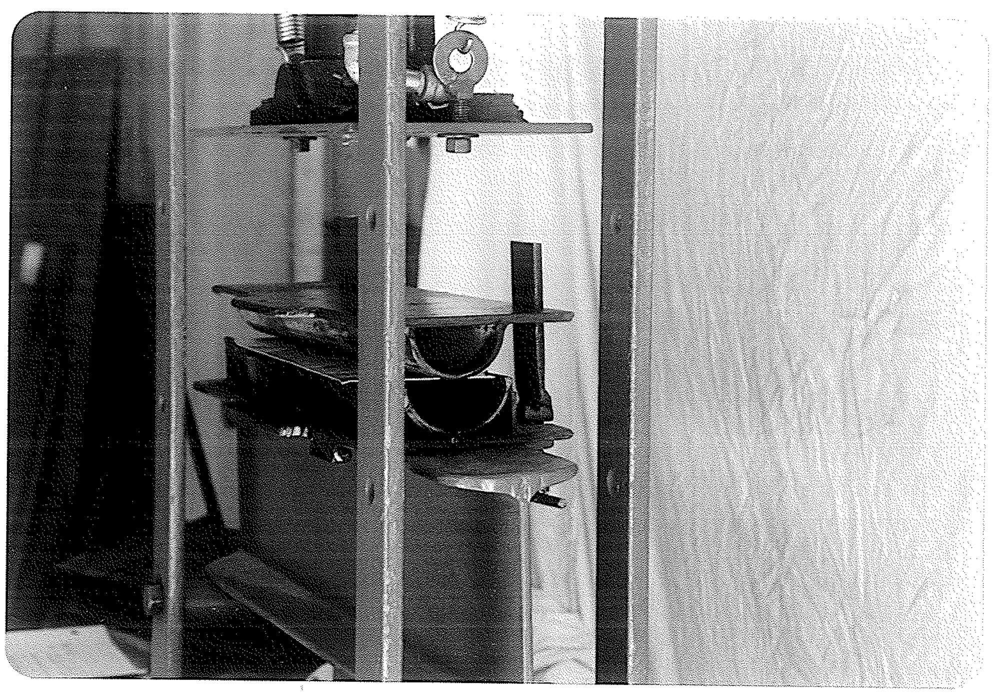
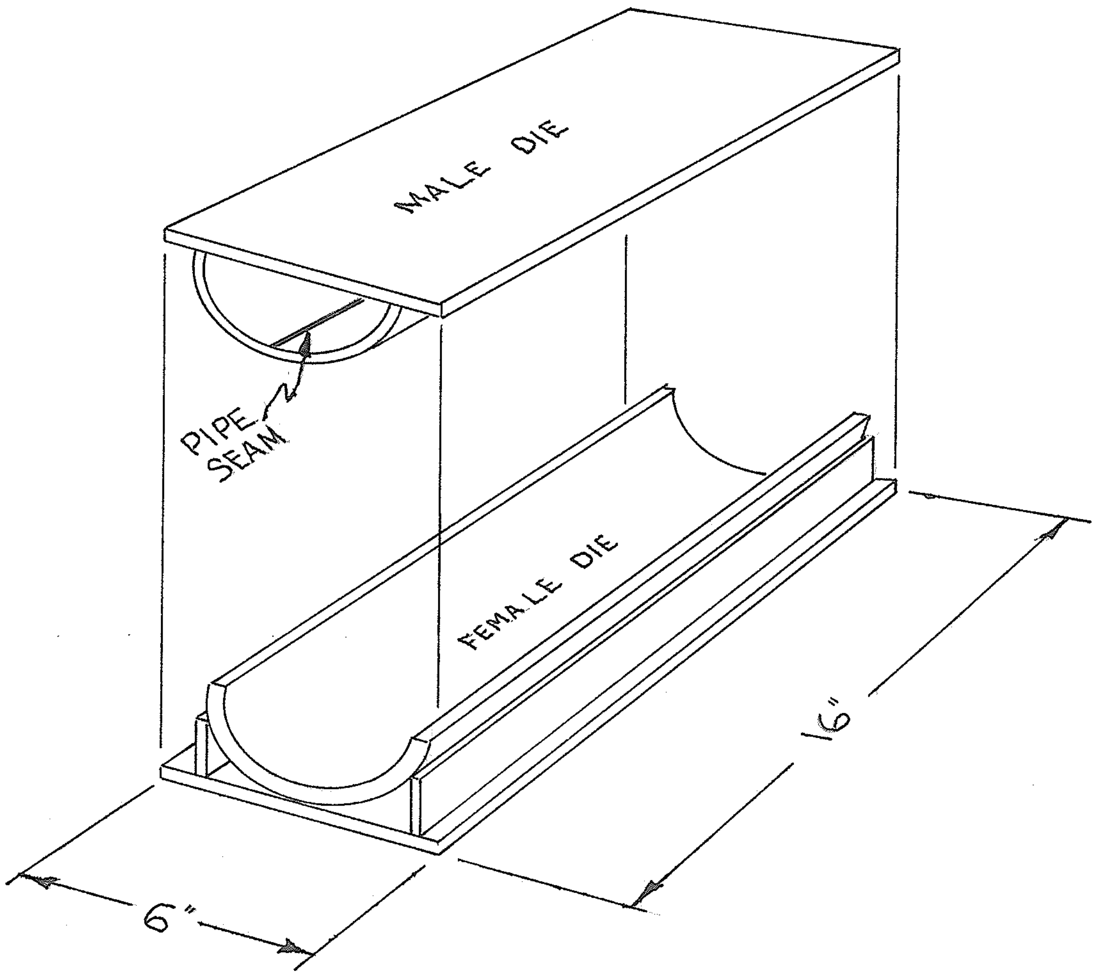


Fig. I1-1: Forming Jig #1



GENERAL ARRANGEMENT

FORMING JIG
DRAWN BY: K. J. DICK
DATE : OCT. 1988
SCALE : NTS
DWG. No. I1-3

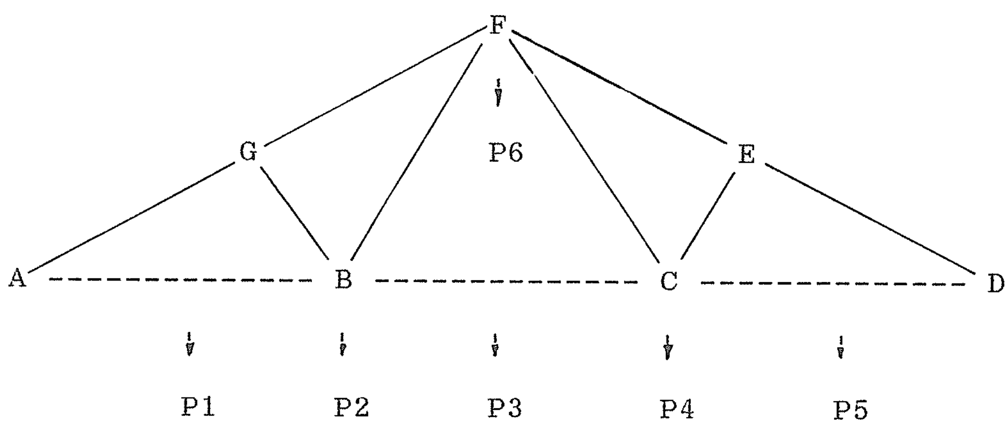
APPENDIX 'J'

DEFLECTION READINGS - TRUSS T2

Date: August 29 - 30, 1988

Location: U. of Manitoba

Table J-1 contains the deflection measurements taken during the testing of truss T2. The points at which deflection was measured are illustrated in Fig. J-1.



P = Point of deflection reading

Fig. J1-1: Deflection Reading Location Points

LOAD (lbs)	TIME (hr) (3)	DEFLECTION READING (1)(2) (in. to nearest 1/16") P1	P2	P3	P4	P5	P6
0	0	0	0	0	0	0	0
300	0.5	0	3/16	1/4	3/16	1/8	1/4
520	0.75	5/16	3/8	3/4	5/8	3/8	1/4
520	1.00	5/16	3/8	3/4	5/8	3/8	1/4
520	3.66	5/16	3/8	3/4	5/8	3/8	5/16
520	4.00	5/16	3/8	3/4	5/8	3/8	3/8
595	4.16	5/16	1/2	1	5/8	1/2	1/2
595	5.50	5/16	9/16	1	5/8	1/2	1/2
595	22.50	5/16	9/16	1	5/8	1/2	1/2
595	22.80	5/16	9/16	1	5/8	1/2	1/2
670	23.66	3/8	9/16	1	11/16	1/2	1/2
745	24.00	3/8	9/16	1	3/4	1/2	1/2
820	25.66	- - failure - -					
TOTAL DEFLECTION		3/8	9/16	1	3/4	1/2	1/2

Note: 1. Each reading taken twice, then averaged.
2. All readings indicate deflection downwards.
3. Indicates cumulative elapsed time from initial zeroing for deflection readings on truss.

Table J1-1: Deflection Readings - Truss T2

It can be seen from the table that the truss sustained a load of 520 pounds for more than 24 hours, plus additional incremental loads for less time.

APPENDIX 'K'

POLE TAPER DATA

The tables in this appendix contain information regarding the taper of the poplar poles that were used in the two test trusses, T1 and T2.

The measurements taken correspond to the individual member label, for example AGF, and go from left to right. If, as in this example, there is a panel point within the member, the diameter of the member is given at that panel point. Web members will have only two dimensions, butt and top end diameters. All measurements were obtained with vernier calipers and taken in such a manner as to record the least diameter at that point. The bark had been removed from the poplar poles at the panel points prior to applying the connectors, therefore, the measurements in Tables K1-1 and K1-2 represent the bare wood diameters.

TEST TRUSS T1

MEMBER LABEL	BUTT DIAMETER (in)	PANEL POINT DIAMETER (in)	TOP DIAMETER (in)	AVERAGE DIAMETER (in)
AGF	2.500	2.250	2.000	2.250(1)
DEF	2.500	2.438	1.875	2.270
ABH	2.500	2.250	2.000	2.250
DCH	2.500	2.250	2.000	2.250
BG	2.250	- -	2.000	2.125(2)
BF	2.250	- -	2.000	2.125
CE	2.250	- -	2.187	2.218
CF	2.250	- -	1.875	2.062
		AVERAGE MEMBER DIAMETER -		2.193

Note: 1. Average diameter = Sum/3
2. Average diameter = Sum/2

Table K1-1: General Measurements - Truss T1

TEST TRUSS T2

MEMBER LABEL	BUTT DIAMETER (in)	PANEL POINT DIAMETER (in)	TOP DIAMETER (in)	AVERAGE DIAMETER (in)
AGF	2.437	2.187	1.875	2.166(1)
ABH	2.250	2.500	2.125	2.125
DEF	2.437	2.312	2.000	2.249
DCH	2.437	2.250	2.125	2.270
BG	2.437	- -	2.500	2.468(2)
BF	2.375	- -	1.875	2.125
CE	2.375	- -	2.312	2.343
CF	2.375	- -	2.125	2.250
		AVERAGE MEMBER DIAMETER -		2.249

Note: 1. Average diameter = Sum/3
2. Average diameter = Sum/2

Table K1-2: General Measurements - Truss T2

APPENDIX 'L'

LOADING REQUIREMENTS FOR SIERRA LEONE

The metal connecting system and timber truss members were designed to withstand the imposed loading due to condtions in Sierra Leone. The loads of concern in this instance, are the dead loads caused by; roofing material, timber truss members and purlins. The calculations related to the loading per truss are based on;

Truss span	-	24'-0"
Truss spacing	-	4'-0" centres

1. Roofing Material:

The common type of roofing material encountered in Sierra Leone is galvinized corrugated iron sheeting (C.I. sheet). If 24 ga. sheeting is used the loading becomes;

Weight of sheeting	=	1.46 psf
Based on 24' span = 24 x 4 x 1.46	=	<u>140.16 lbs/truss</u>

2. Truss Timber:

The loading due to the timber members is based on;

Specific gravity (N. Aspen)	-	0.41
Lineal Feet Required/Truss	-	70.00
Nominal Diameter of Timber (in)	-	2.00 ,

from which the following are obtained;

Timber weight	= 0.41 x 62.1	= 25.6 lbs/ft^3
Total volume	= 70 x (3.14((2/12)^2)/4)	= 1.527 ft^3
Total weight	= 1.527 x 25.6	= 39 lbs/truss

3. Purlin Material:

Using the same material as for the truss members the loading is based on;

Spacing of purlins (ft)	-	2.0 centres
Purlins required	-	13
Nominal diameter (in)	-	1.5 ,

which yields;

Total volume = 13 x 4 x (3.14((1.5/12)^2)/4) = 0.638 ft^3

Total weight = 0.638 x 25.6 = 16 lbs/truss

From the above calculations the total dead weight imposed on the truss becomes;

TOTAL LOAD PER TRUSS = 140 + 39 + 16 = 195 lbs/truss

In addition to the dead loads caused by roofing material and timber framing, the live loads due to snow, wind, and construction personnel loads are normally considered when determining the total load the structure must support. Snow is not a problem in Sierra Leone, but wind and construction personnel must be considered.

Wind loading may cause external or internal(suction) pressures to act upon a structure. The two governing equations used in determining these pressures are presented (Limit States Design of Wood Structures, 1986, pp.118 - 125).

EQN-L1 External Pressure:

$$Qe = q\ Ce\ Cg\ Cp$$

where;

- Qe = external pressure acting normal to the surface
- q = reference velocity pressure
- Ce = exposure factor = 0.9
- Cg = gust factor = 2.0
- Cp = external pressure coefficient

EQN-L2 Internal Pressure:

$$Qi = q\ Ce\ Cpi$$

where;

- Qi = specified internal pressure
- q = reference velocity pressure
- Ce = exposure factor at mid-height of building = 0.9
- Cpi = internal pressure coefficient

Using these relationships the pressures acting on the roof can be calculated. Since no data are available for Sierra Leone an average wind pressure, q, of 5.0 psf

(0.25 kPa) is used in the calculations, corresponding to a wind velocity of approximately 45 mph 72 (kph). As most of the buildings are constructed in well sheltered areas this is seen to be a conservative estimate. A variety of wind combinations can act upon a roof, however, in this instance only the worst case will be presented here. Fig. 4-2 illustrates the pressure coefficients that act upon the roof. (after LSD of Wood Structures, 1986, p.120).

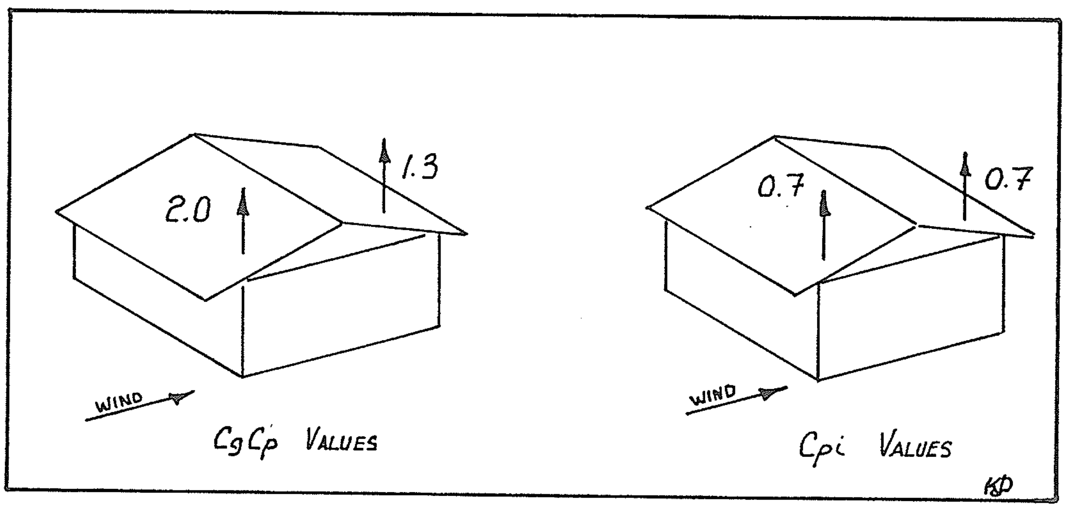


Fig. L1-1: Wind Pressure Coefficients

Using a roof slope of 20 degrees and a q-value of 10.4 equations 1 and 2 yield;

EQN-L1: $Qe = q\ Ce\ Cg\ Cp$

$Qe = 5.0 \times 0.9\ Cg\ Cp$

$Qe = 4.5$ psf $CgCp$

EQN-L2: $Qi = q\ Ce\ Cpi$

$$Qi = 5.0 \times 0.9\ Cpi$$

$$Qi = 4.5\ \text{psf}\ Cpi$$

Substituting the values from Fig. L1-1 into equations 1 and 2 yields the net pressures on the windward and leeward side of the roof. The net pressure is the algebraic sum of the internal and external pressures. (The negative sign (-) indicates that the pressure is causing a net suction effect on the roof.)

	WINDWARD	LEEWARD	SUM
NET PRESSURE (psf)	(-)12.15	(-)9.00	
TOTAL LOAD ON ROOF (lbs)	(-)580	(-)430	(-)1010

Table L1-1: Wind Pressures - Loading

Since the wind loading creates a suction effect, the net truss loading due to wind and dead weight is determined to be 195 - 1010 = -815 lbs. (i.e. upwards) As can be the case with many structures provision must be made to ensure that this suction or lifting force is counteracted. This is accomplished by securely fastening the top plate to the outside bearing wall which provides enough mass to counteract the forces created by the wind. The truss is then attached to this top plate by toenailing, bolting or tying. In Sierra Leone the top plate is usually joined to the wall with reclaimed metal packing straps. The straps are looped around the top plate and through the mud block wall two to three courses down. Fig. L1-2 illustrates this procedure.

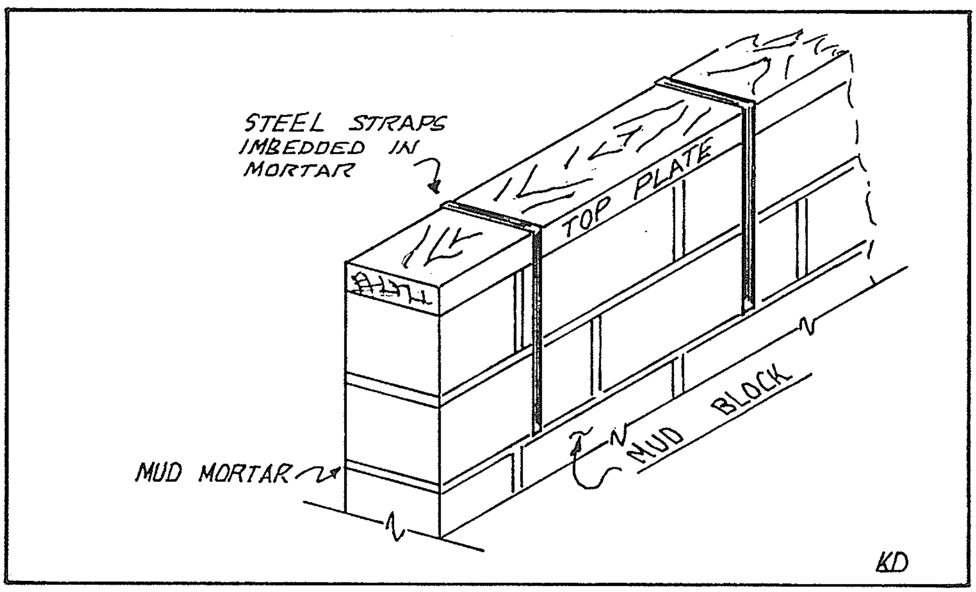


Fig. L1-2: Top Plate Attachment to Mud Block

Such a procedure, two courses down, produces approximately 330 pounds of additional weight at each support end. The calculation is shown below.

Resisting Weight Provided by Mud Block:

Block Size	:	6" x 8" x 16" long (0.44 cu.ft.)
Weight of Mud	:	125 pounds per cubic foot
Truss Spacing	:	4' - 0" centre to centre

Total weight, based on two courses, becomes:

6 blocks x 0.44 cu.ft./block x 125 lbs/cu.ft. = 330 lbs .

Total weight, based on three courses, becomes:

9 blocks x 0.44 cu.ft./block x 125 lbs/cu.ft. = 495 lbs .

In this case, wrapping metal strapping down three courses would provide sufficient resistance to uplift forces.

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