2-WAY DOUBLE LAYER SPACE GRID

AS ROOF STRUCTURES

BY

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THESIS DECLARATION

I hereby declare that this thesis has been composed by myself and that it is a record of my own reseach work. It has not been accepted in any publication for a higher degree.

All the sources of informations which are not my own originals are specifically acknowledged by means of numbers corresponding to the reference numbers in the list of reference at the back of this thesis.

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THESIS APPROVAL

We hereby recommend that the thesis prepared under our supervision by MR. OYI ABDULLAHI entitiled "2-way double layer space grid as roof structure" be accepted in partial fulfilment of the requirements for the award of Master of Engineering in Civil Engineering of Ahmadu Bello University, Zaria.

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ABSTRACT

The problems of covering large span roofs economically has always been of concern to Engineers, but it was not till after the second world war that solutions to the problem was seriously sought. It has since then, been proved and confirmed that space structures are the solutions to covering large span roofs economically.

This thesis looks into the possibility of choosing a suitable type of space structure that could be adapted for use in Nigeria. Hitherto proprietary systems have been imported into the country for this purpose.

An investigation was carried out into the different type of space structures available. Double layer grid truss was selected thereoff for study. Using the direct stiffeness method of structural analysis, different types of trusses were analysed with the aid of a computer. Parameters like span, height above ground level, depth of truss, support were varied to ascertain a good overall dimensional ratio. Connectors of two types were also designed for the joints of some of the trusses. Models were then tested to conlude the effectiveness of the design criterea used in designing the connectors.

From the results obtained, deductions were made as to both the dimensional ratio that results in a better force distribution and requiring less volume of material for its construction.

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INTRODUCTION.

1.1 GENERAL:

The use of space structures in Nigeria has only found wide application in areas of communications, electric transmission towers and mast construction. Their use as roof covering in the country has been very limited even though it is for this purpose that space structures derived its present world wide popularity particularly in advanced countries.

Space structures may be defined as a three-dimensional assembly of elements resisting loads which can be applied at any point, inclined at any angle to the surface of the structure and acting in any direction.* This structures in which the three dimensional function is realised has been in existence from the earliest times as domes. The discovery of new materials (for example iron) opened a new era for space structure in the 19th century and by the 20th century, the desire of progressive designers to match nature and efficiency displayed in biological engineering (for example)* gave rise to space grid structures. Architects have since developed interest because of aesthetics and the ability of space structures to provide exceptional freedom in design.

Space structures could be classified into three main types;

- skeletal(braced) frame works. for example braced domes,barrel vaults and multi-grid structures.

*Z.S. MAKOWSKI.

-stressed skin systems; for example folded plate structures.

-suspended(cable or membrane) structures. for example cable roof structures.

Space structures have been found to be generally lighter and more economical to construct with the economy increasing as the span of the roof increases. Different materials(steel, alluminium, plastics, timber, inflatable forms excetera) are used in practice, but most commonly used is steel particularly tubular steel because of its light weight matched with high moment of inertia. One type of space structure that has enjoyed a remarkable development is the double layer grid structure.

1.2 STATEMENT OF THE PROBLEM:

In the past the development of space structures have been limited by some factors namely;

-complexity in analysis due to its statically indeterminate nature and the large simultaneous equations coupled with it

-difficulty of connections at joints so as to avoid heavy and cumbersome joint for the so many members meeting at every joint.

-suitability of material to withstand the structural load without itself constituting a high dead weight.

-difficulty in constructions at site.

With the recent trend in the development of electronic computers(mainframes, micros, and minis), the problem of solving for large equations has been to an extent (depending on the computer memory) overcome. The development in the steel industry has gone a long way in overcoming the problem of materials of adequate strength. There has also been some improvement in jointing system particularly for proprietary systems. Since connections at any joint is influenced by both the geometry of members and their internal forces, one cannot lay claim on a universal connector. Hence, depending on the geometry and magnitude of forces in members, there is need for a design of the jointing system particularly in the non proprietary systems. The difficulty in construction has been eased by the development in cranage system. Hence, it is easy to assemble most parts at ground level and have it raised to the desired height.

1.3 JUSTIFICATION FOR STUDY.

Nigeria as a developing nation is undergoing an industrial take off. This therefore means that buildings of very large spans (hangers,airports,stadium, industries,assembly halls,exetera) are being designed and erected frequently. The need to pay an interest by way of research into a possible roofing system which will cover these large spans economically and with least maintenance is very necessary. Most roofs normally carry a uniformly distrubuted load comprising mainly self weight of the roofs and coverings. Space roofs structures are known to be very economical in covering large spans. Double layer grid trusses are not only known to be efficient, they also have a reccuring identical unit parts.This means that trusses can be manufactured and erected in short times resulting in an overall economic production. It is therefore necessary to study the design of double layer grid structures and their jointal connections using a suitable material.

1.4 LITERATURE REVIEW.

There has been research developments in some aspect of space structures. These aspects can be broadly classified into

1. analysis
2. design
3. jointing system.

1.4.1 Analysis

This is one of the areas that has recieved the greatest attention. Most of the research developments centered around investigations on methods of structural analysis by elastic and sometimes plastic methods. Assumption of the flat space trusses as plate and analysing them by differential equations have been considered and used. Methods of analysis by matrix methods of analysis has also been researched into for solutions of skeletal and folded structures. Researches into computer techniques to reduce large matrices that usually results in the matrix methods have been carried out to conserve storage spaces.

Renton(02) discussed the related behaviour of space grid and plates and concluded that due to the repetition of a basic module in space structures, high degree of regularity results. This he said makes for an easily written stiffness equations relating loads to their corresponding deflections and those of adjacent joints. Hence, he said space grid problems could be viewed in terms of finite difference equations which could be reduced to a single differential equation. He concluded that while plane grids (rectangular, triangular and hexagonal mesh) behaves like an isotropic plate and have a resulting biharmonic equation, double layer grids do not, resulting in a non-biharmonic equation. Therefore solutions to equations for various boundary conditions can be found in the biharmonic equation of the plane grid but not in every case of double layer grid, except in some practical cases where the double layer grid is reduced to a sandwiched plate. (see figure 1)

Ignacio and Hernandez (03),(04),as far back as 1961 also realised the necessity of obtaining an exact method of solving for space frame work. They carried out a research on frame works with members in any direction under any system of loads and emphasised taking advantage of the computer. They established a solution by applying the "slope deflection-gyration method" which provides a set of simultaneous equations. They remarked that although methods of successive approximation contributed much to the progress of structural analysis of continous structures,this could be avoided where tools for solving large sets of simultaneous equations are available. Their solutions involve establishing for the members of the frame work;

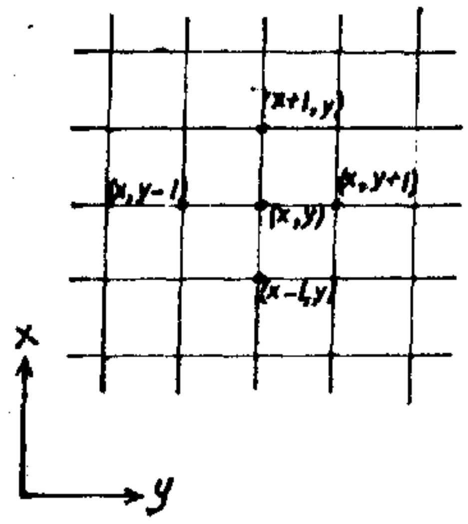
-flexural moment

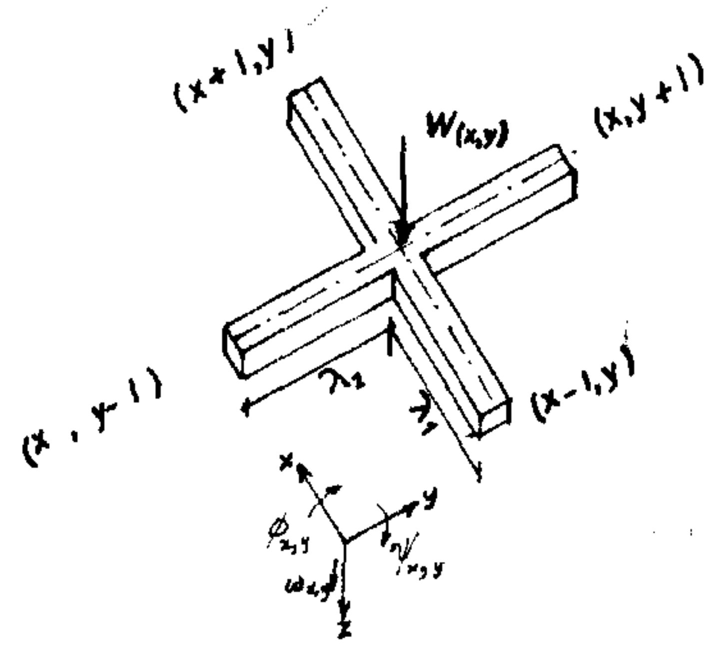
-torsional moment

-shear moment equations and for the joints of the space frame work;

-equilliorium equations of forces

-moment equations.





W = Vertical force
D Bending stiffness per
λ = Length of a beam
ω = Vertical displacement
φ, ψ = Rotations
x, y, z = Coordinate axis
q = A distributed load

The biharmonic equation for plane grids is given by:

$$\frac{\partial^4 \omega_{x,y}}{\partial x^4} + \frac{2\partial^4 \omega_{x,y}}{\partial x^2 \partial y^2} + \frac{\partial^4 \omega_{x,y}}{\partial y^4} = q/D \quad \text{------(1)}$$

For

- rectangular mesh grid;	$D = EI/\lambda$	}	------(2)
- triangular " "	$D \quad EI(\sqrt[3]{3}/4a)$		
- hexagonal " "	$D \quad EI/\sqrt{3}$		

The equation for double layer grids is non biharmonic;

$$\frac{\partial^4 \omega_B}{\partial x^4} + \frac{\partial^4 \omega_B}{\partial y^4} = q/D \quad \text{-------} \quad (3)$$

$D = bh^2 l/2$
B = point under consideration
b = compressive stiffeness of a top bar
l = length of bar
h = vertical seperation of a space grid

FIG 1 RECTANGULAR MESH GRIDS AND ITS HARMONIC EQUATION AS PRESENTED BY RENTON (Q2)·

In deriving the equations, geometrical relation of members with reference to an arbitrary cartesian coordinate system was used. Deformation of joints were classified into angular, and axis displacements. Computation of design moments and shear was carried out by solving the simultaneous equations. In their conclusion, they remarked that the slope-deflection-gyration method provided an exact solution of the space frame work.

Drymiotis and Kazma (05) researched on the use of stiffness method for large complex structures and arrived at the followings;

-that the stiffness method could be employed by dividing the original large structure into what they reffered to as "super members" making sure that no member belong to more than one super-member

-that stiffness equations could be written for every super-member(S-member) in a local reference or right handed cartesian coordinate.

-that displacement for the S-member ends could be calculated by reducing the original S-member stiffness equations.

The force displacement relation for a member of a skeletal structure with end joints i and j (i>j) is written thus.
5(see figure 2)

$$\begin{bmatrix} P_i \\ P_j \end{bmatrix} = \begin{bmatrix} K_{ii} & K_{ij} \\ K_{ji} & K_{jj} \end{bmatrix} \begin{bmatrix} d_i \\ d_j \end{bmatrix} \quad \text{------------(4)}$$

P_i and P_j are members end forces and displacements at ends i and j respectively.

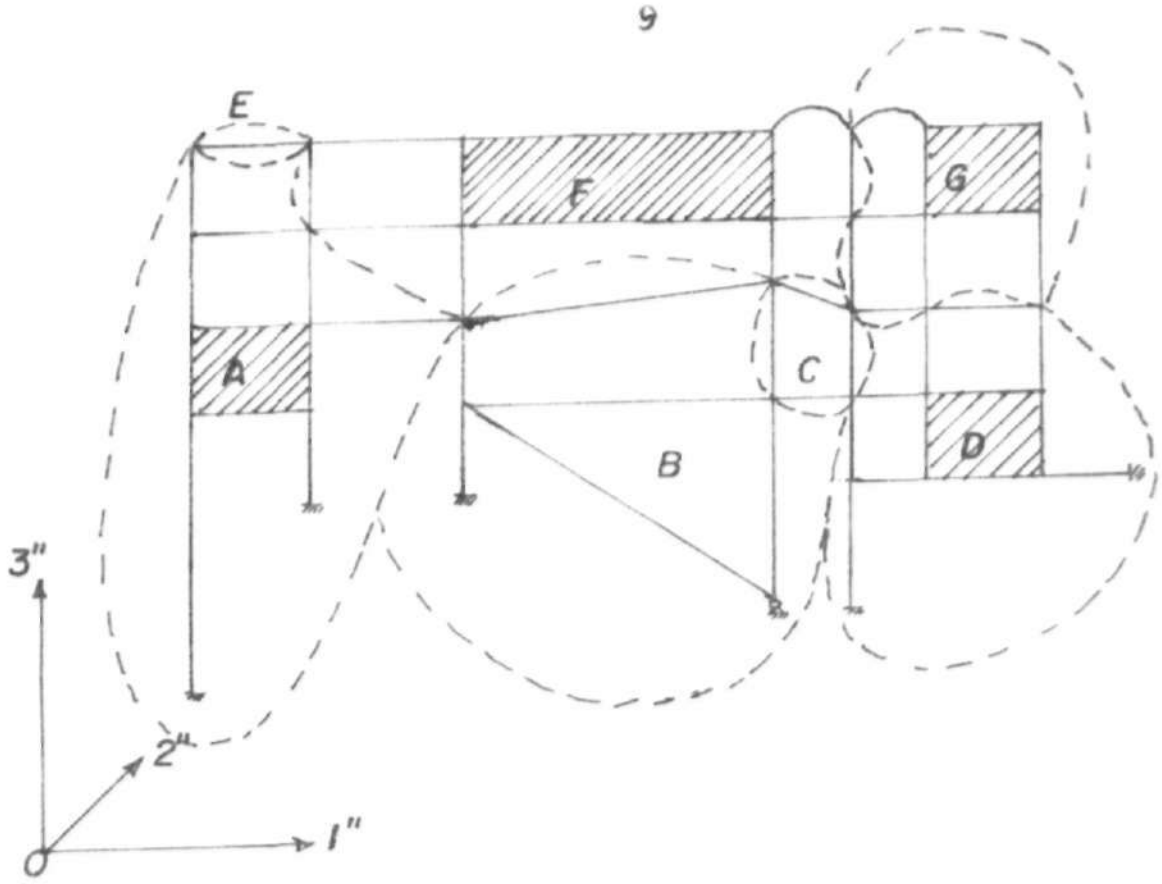
are the members stiffness matrices.

Harrison (06) described a numerical method of analysis which can achieve a first order linear elastic solutions for pin jointed frames. This method is otherwise reffered to as the direct stiffness (deformation) method. He then carried out a research on the basis of 2 assumptions.

-that material used is Hookean and that structural members are straight so that relative deformations within members are proportional to the stress-resultants developed in the course of frame deformation under load.

-frame deformations will not invalidate the equilibrium equations which may then be formulated for the unloaded shape .

Using a second-order solutions (as a check) he observed that the errors resulting from the assumption using first-order solutions are generally small. Hence first order elastic analysis are adequate for normal frames except for structures that are closely and optimally designed using high strength materials where he found that the errors are



Recall equation 5

$$\begin{bmatrix} W^F \\ L^F \end{bmatrix} = \begin{bmatrix} K_{11}^F & K_{12}^F \\ K_{21}^F & K_{22}^F \end{bmatrix} \begin{bmatrix} d^F \\ D^F \end{bmatrix} \quad \text{------} \quad (5)$$

Where W_F^F = *The load vector of the internal joint*

L^F = *The force vector at S-member ends*

d^F = *The displacements vector of the internal joint*

D^F = *The displacement vector of the S member ends*

FIG 2. A Skeletal frame work divided into Drymiotis(03) Super members quite significant and therefore a second order solution should be sought. It should be stresses that a high strength material is more flexible for a designed member than a mild material because of increased slenderness ratio.

Makowski, Z.S and Nooshin, H. (07) also showed in their work that the direct stiffness method is very accurate in analysing space frames. They further suggested ways of overcoming its limitations, which is the enourmous computer storage requirement.

Abatan, A.O (03) showed that the finite element method is a very powerful tool for analysing structures that can be subdivided into a series of elements interconnected at nodes. He also went ahead to discuss equations to be solved in the finite element method and how computer memory problem could be overcome by making the alogrithm dependent on the 3 x 3 matrices for each elements of the entire structure stiffness matrix. He however mentioned that the greatest asset of the finite element method is its ability to cope with irregular bodies.

From the above, it could be appreciated that 3 methods of analysis have been widely used. They are

-plate analogy method entailing differential equation. This method was particularly popular prior to availabilty of compuers with large memories. The method is suitable for plane and thin grids.

-the direct stiffness method. Due to the large amount of matrices involved, it was not seriously considered till the availability of computers with large capacity memories. The method is very suitable and accurate for regular skeletal space structures.

-the finite element method. This is the most recent of the three methods, and is most suitable where irregular boundaries or continua are in question. This method is also very accurate and like the direct stiffness method involves solving for large matrices.

More frequently (particularly in the past) physical model analysis has been used to compare results achieved by direct analysis. The limiting problems to this method has been the trouble involved in load application, particularly where this has to be done in 2 or 3-dimensions. Other limitations according to J. Kolosovski (09) include the difficulty in interpreting experimental results aespecially in folded structures due to existence of combined actions. Also Heinz Hossdorf (10) in his discussion said that with future developments in equipments that could apply loads in 3 dimensions, model analysis will serve as a better method where a problem cannot be investigated analytically. He said that inadequate computer storage capacity is also one of the problems that could make model analysis better.

1.4.2. Design.

Research into the design of members has not recieved very much attention. This is most likely due to the fact that the elastic method of design to specified stress limits, that has been in use has up to now been very satisfactory.But in cases where very high strength steel might be used for construction, another important limiting factor is normally deflection. Due to this, some scholars are now devisng a method that combines in the analysis a design. The structure then satisfies both stress and deflection limits.

Majid,K.I and Okdeh,S. (11) of recent carried out a research in the design of skeletal space structures.They proposed a design method based on the limit state theory, This method which is basically a direct stiffeness method aims at specifying an allowable deflection in a particular direction for a supposed critical joint, and calculacing an optimal area for the structure. The design procedure requires;

-specifying the joint where deflection is expected to be most crtical.

-allocating initial values of a cross section area to a group and, a ratio of member to group crosssectional area.

-calculating new values of group cross sectional area and hence member area.

-reapeting the process until ratio of the difference of the design area is below the specified tolerance level.

-checking that the maximum displacement is within the acceptable limit.

This method was found very satisfactory where optimum cross sectional area for each member is required. But they also pointed out that in most practical cases, members of a space structure are often grouped, and that sometimes a single group member may be desired for economic reasons(this corresponds to where group to member cross sectional area ratio equals one). They therefore concluded that though grouping of members results in a heavier hyperstatic structure, it reduces construction and manufacturing costs.

1.4.3. Jointing systems:

In the research of space structures, there is a unanimous opinion that the difficulty of having to join several members on space structures have limited their use. Most of the researches carried out in the method of jointing has been in the manufactures of the proprietary systems. The result is that a lot has been achieved in terms of proprietary connectors. Some of such proprietary connectors are discussed below.

(i) - Unistrut Corporation:

Hsiao .S.C (12), describe the connecting system made by the company as comprising two parts, the instrut and outstrut connectors(see figure 3a). Both of them are symmetrically opposite. The system invented and patented by Charles.W.Attwood is manufactured by the Unistrutcorporation. The connectors are made by press-forming a 5mm thick plate into an eight way cup with holes and luges punched inwardly or outwardly to recieve the web members.

(ii) Fentiman & sons Triodetic Hub.

Fentiman H.G (13), acesses the economic use of

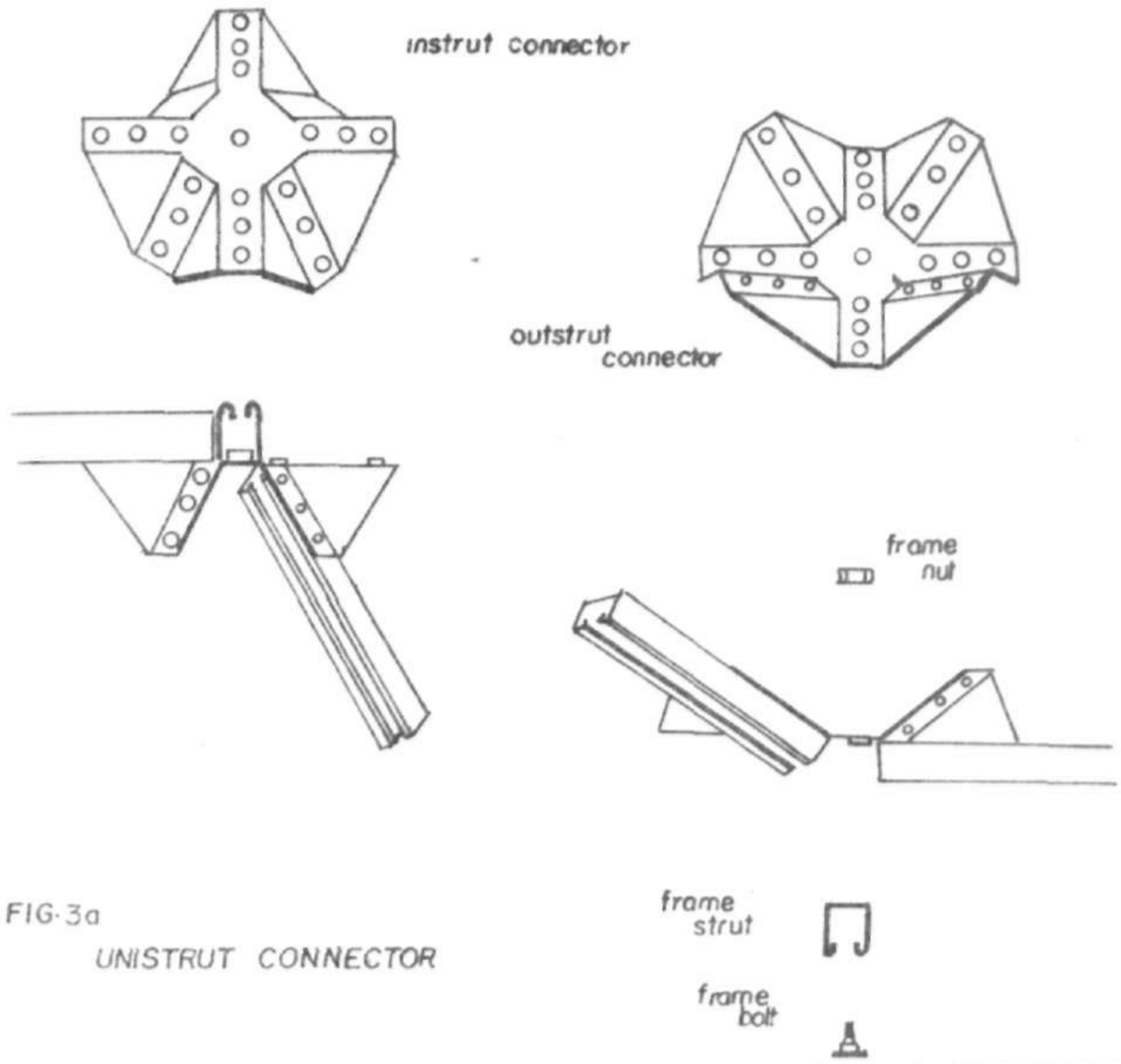


FIG-3a
UNISTRUT CONNECTOR

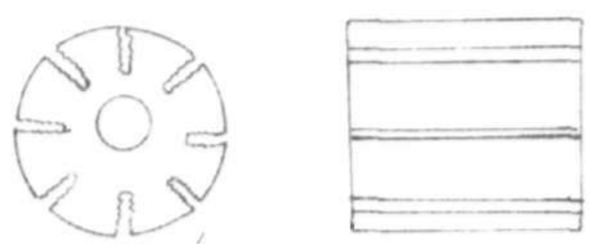


FIG-3b TRIDETIC HUB

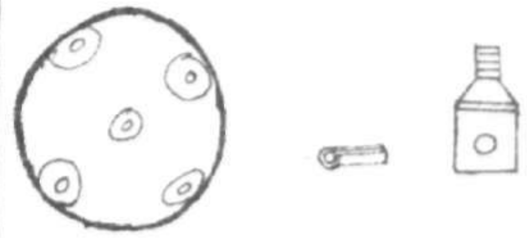


FIG3d MERO JOINT

space frames structures. He went on further to discuss the development of the triodetic hub(by him) which has found consderable use in space structures. The jointing system which is a hub consist principally of a cylidrically shaped extrusion having serrated slots or key ways. The structural members are then cold formed and serrated at their ends to fit into the key of the hub.(see figure 3b) The idea of such a hub was concieved by Fentiman from the impression he got from the gripping strength of the jaws of the tensile testing machines on test spacimens.

(iii)-Space Deck Nodus.

After much reaearch work by the Tubes Division of the British Steel Corporation, the nodus jointing system was developed(19)with the aim of producing a joint capable of taking full advantage of high yield strength hollow sections.Some principles were established as pre-requisite for the design of the system. These are

-the joint must carry full loads due to axial forces and bending moment.

-the system should be simple to fabricate

-to permit roof cladding to be laid flush on the grid,(with out the use of purlins) the joint should not project significantly beyond the profile of the chord member to which it is connected.

-the joint should be strong enough to take full advantage of high yield stress structural hollow sections.

Four sizes of the joint were then designed for use with circular hollow section(C.H.S) or rectangular hollow section(R.H.S) or a combination of both.The sizes range can

The range of sizes can accomodate between 50- 120mm diameter hollow sections.

The jointing system itself consist of : (see figure 3c)

-two half casings

-a high strength friction grip bolt

-chord connectors with machined teeth

-fork connectors.

The two half casing are bolted together with the friction grip bolt, while the top and bottom chords are welded to the machined teeth connectors that are then screwed into the couple casing . The web members are also butt-welded to the fork connectors,and the fork end pinned to the side of the casings with four lugs.

The nodus system has found wide usage in Europe and the largest of such structures cover an area of 10500m , with cummulative member length of over 24km weighing in excess of 300 tonnes.

(iv)-Mero joint:

Mengeringhausen (15) as far back as 1943, realised the similarity in natural space lattice structures of atoms and space structures. Employing the concept of their nodal points which are centres of closely grouped spheres of equal diameter, he then assumed the joints of the skeletal space grid to coincide with these nodal points of the lattice structure. He then evolved at what is the Universal 'mero'-connector. The mero connector(see figure 3d) is designed to take up to 18 members from 18 different directions meeting at a joint. The ends of the members are attached to the connector by a single pin bolt which is fixed by a locking pin. The connector is manufactured by Mero-Raumstruktur Gmbh & company of W.Germany.

(v)-Octaplatte Nodes:

Iffland, S.B.J (15) described the oktaplatte system which uses for their nodes, two hemispherical shells and a diaphram welded together to form a hollow steel sphere. The members(normally hollow tubes) are then connected by fillet welding.The system is manufactured by the Mannesmann company of W.Germany.

More often than not proprietary systems may not be employed due to the following reasons.

-where very large free columns is involved thereby requiring a larger module.(but proprietary systems are more suitable and satisfactorily used for finner grids(15))

-when proprietary connectors are inaccessible particularly in countries where they are not marketed.

-in keeping down cost by not importing.

This implies that a fabricated joint must be designed . Lovie P.M (17) said that in designing these joint it is imperative to keep the size of the joint as small as possible because the cost of the joint material is four to five times as expensive as the cost of the structural member. he also showed that the intersecting angle of members to the joint places a limit on how much the size could be reduced . So far adequate criteria guiding their design has not been presented and the choice of connectors have been based on testing the strength of such connectors after fabrication.

Some examples of how Engineers have tackled the problem of jointing system in non proprietary systems are shown below:

(a)-Pauly Pavillion Connector:

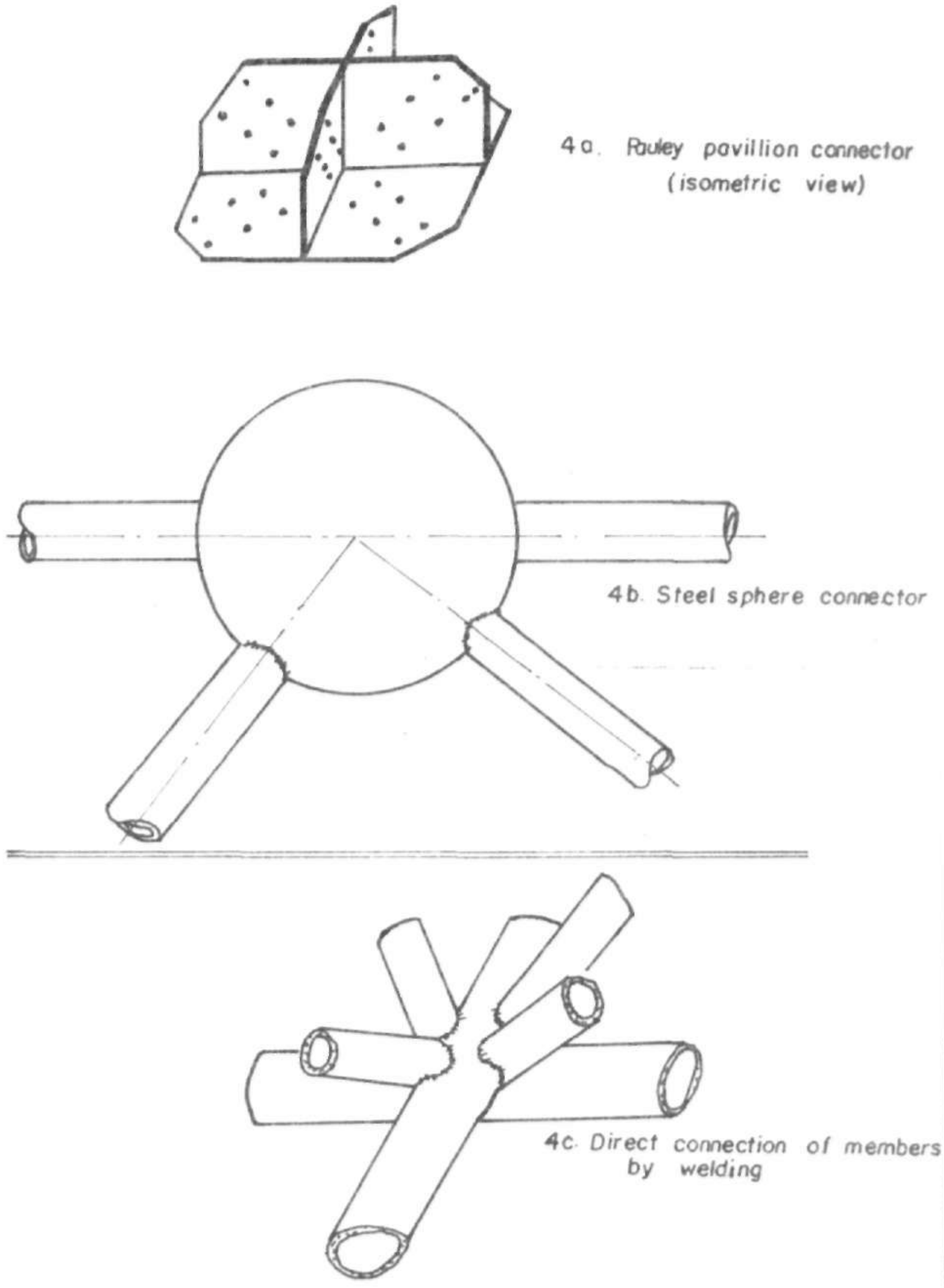
This connector (17) was designed and used in the construction of the space roof truss of Pauly Pavillion at the University of California in Los Angeles(see figure 4a). The structure provides a clear span of 91m-by-122m having a basic module of approximately 10m-by-10m. Eight members are joined at the connector by means of rivetting.

(b)-Denver Exhibition Hall Connector (17):

The exhibition hall which covers an area of 73m-by-209m have in some cases up to 12-members meeting at a joint. The connector can be seen as consisting of two Pauly Pavillion connectors connected back to back.

(c)-In some cases direct welding of tubular members have been done. In a 30m-by-40m two way space grid designed by Poniz.V and Poniz.D (13), members were sized in such a way as to allow for direct welding without introducing an additional connector or gusset.(see figure 4c).

(d)-Bueno .P and Calavera .J (23) designed a connector for use in Madrid. the connector consist of two hemispherical steel casted and later on welded. (see figure 4b). They said, no rational procedure existed in choosing the thickness of such sphere. Therefore thicknesses of such spheres were chosen at will and later tested with two opposite members in tension.



4a. Pauley pavillion connector (isometric view)

4b. Steel sphere connector

4c. Direct connection of members by welding

FIG. 4 Non Proprietary Connectors

Some practical uses of the connector include:

-the roof for Capilla del Instituto 'El Bricanse' in Caceres.

-roofs for the Fabrica de Crevezas Mahov in Madrid

-roof for Pabellon de Deportes del real Madrid.

1.5 TYPES OF ROOF SPACE TRUSSES:

Different types of flat trusses have been employed so far for practical purposes of roof covering or in rear cases as decks, most of which are chosen and pattented so as to conform to their uses (by varying directions of top and bottom members, with respect to the bottom joints, providing skylight, excetera). Skeched below are some of them, their discription and where used. See figure 5.

1. 2-Way Double Layer Lattice Grid:

   This is a simple 2-way lattice grid. It comprises of square on square with no skew diagonals. The diagonal web members lie in the vertical plane and its basic element is a cube,basically this is not a space truss. The advantage of this type is the simplicity on the jointing details due to the geometry of members, even though the torsional resistance is quite negligible. Places of practical use include part of the Murtala Muhammad square in Kaduna.

2. 3-Way Double Layer Lattice Grid: (see figure 5c)

   This is a 3-way lattice grid which like the 2-way grid above is basically not a space truss. The basic element is triangular prism which is extremely stiff and efficient, and adatable to spanning odd-shaped areas. Practical uses of this include

   - Met-Ram three way lattice grid at Feltham erected for testing purposes.
   - Roof covering of Strade Francais, swimming pool, Piscine de Boulogne,Paris.

3. Triangular Based Pyramidal Grid: (see figure 5d)

   This is a system with top and bottom chords square but offset on a diagonal. This has a basic unit of triangular based pyramid which has an efficient load distribution and

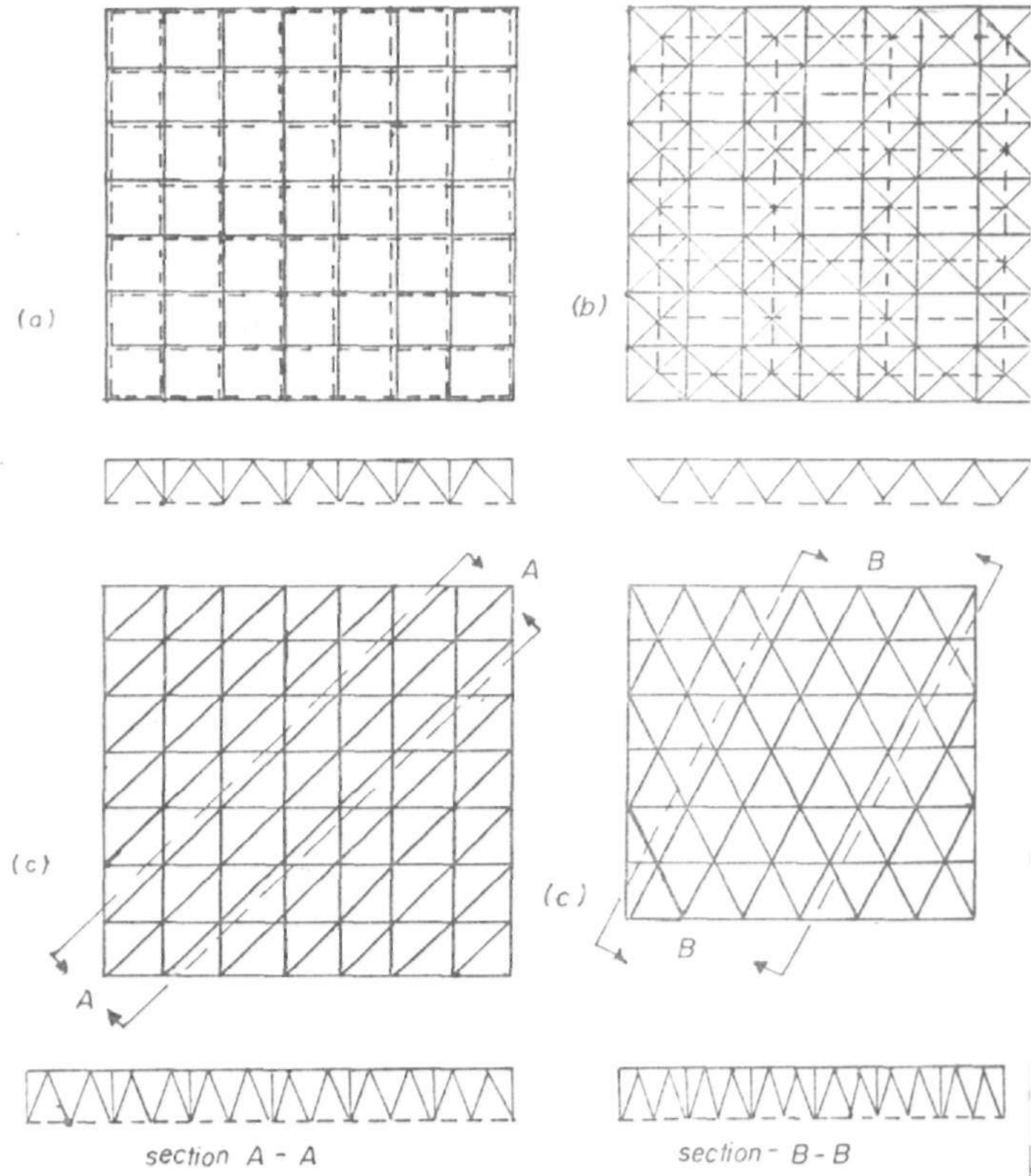


FIG 5. TYPICAL TYPES OF FRAMING SYSTEM

(a) 2-way double layer lattice grid
(b) 2-way double layer space grid with internal openings
(c) 3-way double layer lattice grid

high torsional resistance. Its practical use has been limited by the large number of members and their arrangements at any joint resulting in a complicating jointal system.

4. Square Based Pyramidal Grid: (see figure 5e)

This is a double layer space grid with a square top chord offset over a square bottom chord. The basic unit of the grid is a square based pyramid which is stiff and has a good load distribution system with a maximum of 8-members per joint. This is one of the most commonly used flat space truss for roofing and decking purposes. Its use as decks is common in buildings with extensive sanitary and air conditioning equiptments, examples are laboratories, centre for electronic computers, hospitals and several types of industrial buildings. Practical uses are numerous some of which include

- Architectural pavillion and some dinning halls in Ahmadu Bello University.
- Part of the terminal hall at the new kaduna airport under construction
- A new bank building in Kano.

5. Others. (see figure 5b)

Most other patterns are basically a modification of one of the above which may be done by reducing some members of the top or bottom chord to provide skylight, mirror image reversal and other features. figure 5b is a modification of figure 5e described above, with internal openings.

1.6 SCOPE OF INVESTIGATION.

After a preliminary study of the various types of space roof structures, a selection(based on flat roof trusses) of a type to study for use in Nigeria was done. In selecting, attention was closely paid to the low skilled labour available in Nigeria. There was also the consideration of a necessity to evolve design(s) that could be built locally in the country without resorting to importation of proprietary systems. The scope of investigation could be classified thus:

(i) Analysis of double layer grids of a chosen pattern, varying such parameters as span, height, depth,and support methods (see table 1).

(ii) Design Of Trusses for all the grids analysed in (i) above.

(iii) Design Of Connectors of different shapes after setting out criteria that would evolve connectors of minimum size.

(iv) Testing some of the connectors in (iii) above, after fabrication.

1.6.1: Analysis

In investigating what method of analysis should be utilized for structures as the double layer grids, emphasis was laid on:

- The need for an accurate(or exact) result and secondly

- The recent trend in the use of electronic digital computers(mainframe,minis and micros) in the design office, universities, and recently in some domestic homes were taken into consideration.

Matrix method of structural analysis which provide an exact method and also pocesses the ease for algorithm development

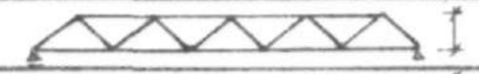
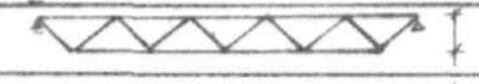
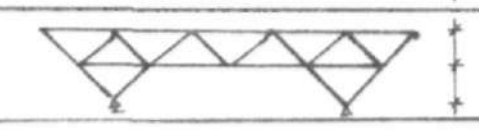
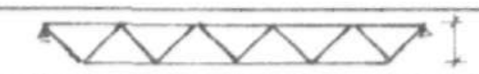
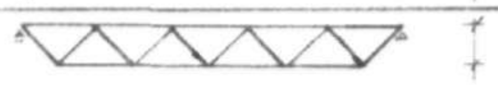
	DIMENSION (m x m x m)	SKETCH	SUPPORT TYPES	DEPTHS (d) m	HEIGHT ABOVE G/L
1	10 x 10 x d		2	1·5, 0·5	5, 10, 15
			1	-0·5	
2	20 x 20 x d		2	2·0	5, 10, 15
			1	-1·0	
3	40 x 20 x d		3	-1·4	5, 10, 15
4	40 x 40 x d		2, 2'	2·0	5, 10, 15
5	50 x 45 x d	"	2		5, 10, 15
6	80 x 80 x d	"	2	2·5	5, 10, 15
			1	-2·5	
7	100 x 90 x d		2	3·0	5, 10, 15
			1	- 3·0, -2·0	

TABLE I; DOUBLE LAYER GRIDS INVESTIGATATED WITH THE THE FOLLOWING PARAMETERS VARYING:

- Dimension (m x m x m)
- Depth (m)
- Support type (1, 2, 2', 3)
- Height above ground level (G/L

development was chosen for use. Matrix structural analysis can be broadly classified into the following:

(a) flexibility (force)method

(b) direct stiffeness method and sometimes

(c) finite element method.

(a) Flexibility(force) Method:

The approach to matrix structural analysis that involves equation written with member forces as unknowns is reffered to as the flexibility method .

$$p = cf$$

$$f = c^{-1}p$$

where

c=compatibility matrix

p=applied joint forces

f=unknown joint forces.

The flexibility method requires a formidable sequence of matrix operations (matrix multiplication, inversion, transposition, excetera).The main advantage is that , solutions of internal forces are produced directly. But the above method does not enjoy a significant utilization in mordern practice. One major reason is the inconvienience of redundant force system, which must be identified and calculated prior to the actual determination of the unknown forces in statically indeterminate structures

(b) Direct Stiffeness Method:

The approach to matrix structural analysis which involves the formation of algebraic equations of analysis on which joint displacements play the role of unknown is known as the direct stiffeness method.

$$Q = Kq$$

$q = K^{-1}Q$

where

Q = generalised force vector

K = stiffness matrix

Q = generalised displacement vector.

This method has been particularly chosen not because of its popularity in practice, but due to the fact that stiffness relations can be derived for elements supported in a statically indeterminate manner whether in the representation of a simple or complex structure.

(c) Finite Element Method:

This method which is the most recent refers to the particular problems of modelling a CONTINUA in the same manner as in the stiffness method. Therefore when an analyst is confronted with the need to perform calculations of stresses and dispacement of structures having continous distribution of materials or having dimension such as plates , curved shells and solids. The most powerful tool for the analysis is the Finite Element Method. Since our structure under the present investigation is very regular, the method has not been looked into for usage.

1.6.2. Summary Of The Direct Stiffeness Method (D.S.M)

The direct stiffness method is valid for linear conservative structures with n - unknowns, n - linearly dependent joint displacements and hence n - degrees of freedom.

The steps are briefly described below.

STEPS:

1. Write out the force-displacement equation for every member(i)-in local coordinates

2. Write out the force-displacement equation for every member(i) in global reference coordinate

3. Relate global member end forces to their corresponding joint displacements.

4. Impose the constraint(R=0) and assemble the constrained stiffness matrix.

Solve the resulting equation to get the required displacements.

6. Subtitute the displacements in (3) above to obtain the global end forces.

7. The end forces are then used to obtain the global member forces by SUPERPOSITION.

8. The member end forces in local coordinates are obtained using the ROTATIONAL TRANSFORMATION.

9. The reactions(where required) are then calculated.

(a) Member force displacement relationship for plane trusses:

(i) In local coordinates:

In trusses, it is generally assumed that each individual member can only be subjected to an axial load.

therefore for any particular truss-member with two ends(a-end,b-end) at at joints say(1 and 2).The force displacement relation in the local coordinate will be as given below

$$\begin{bmatrix} f_{a1} \\ f_{a1} \end{bmatrix} = \alpha\beta \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{bmatrix} u_{a1} \\ u_{a1} \end{bmatrix}$$

Where $\alpha = EI/L$

$\lambda = (L/r)^2$

$r = (I/A)$

The proof for the above result lies in the simple fact that, the truss element is capable of transmitting force in only one direction and hence the displacement of such force in the local coordinate is also in this one direction which correspond to the direction of the member itself.

Therefore

$$\begin{bmatrix} f_{a1} \\ f_{a2} \\ f_{a3} \\ f_{b1} \\ f_{b2} \\ f_{b3} \end{bmatrix} = \alpha \begin{bmatrix} \beta & 0 & 0 & -\beta & 0 & 0 \\ 0 & 12 & 6L & 0 & -12 & 6L \\ 0 & 6L & 4L^2 & 0 & -6L & 2L^2 \\ -\beta & 0 & 0 & \beta & 0 & 0 \\ 0 & -12 & -6L & 0 & 12 & -6L \\ 0 & 6L & 2L^2 & 0 & -6L & 4L^2 \end{bmatrix} \begin{bmatrix} u_{a1} \\ u_{a2} \\ u_{a3} \\ u_{b1} \\ u_{b2} \\ u_{b3} \end{bmatrix}$$

since the forces and the displacements in the remaining directions(2 and 3) are not existing(equals to zero), the columns and rows corresponding to these are deleted resulting in equation 6

Equation 6 can be written in the general form of

$$f = ku \text{ -----------------(8)}$$

where

$$f = \begin{bmatrix} f_{a1} \\ \hline f_{b1} \end{bmatrix} \text{-----------------(9)}$$

$$k = \alpha\beta \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \text{-----------(10)}$$

$$u = \begin{bmatrix} u_{a1} \\ u_{b1} \end{bmatrix} \text{-------------(11)}$$

Recall that $\alpha\beta = AE/L$

Reffering to the above figure, it could be easily seen that for horizontal equillibrium:

$$f_{a1} = -f_{b1}$$

(ii) In global coordinate:

Translational transformation.

For a frame element; The translational ransformation is given as

$\bar{f} = T_{ba}\, f_a$ ------------------------------(12)

where

T_{ba} = Translation matrix.

Therefore

$$\begin{bmatrix} f_{b1} \\ f_{b2} \\ f_{b3} \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ X & -X & 1 \end{bmatrix} \begin{bmatrix} f_{a1} \\ f_{a2} \\ f_{a3} \end{bmatrix} \text{------------(13)}$$

but for trusses, $f_{a2} = f_{a3} = f_{b2} = f_{b3} = 0$, deleting the appropriate rows and columns gives,

$\bar{f}_{b1} = 1f_{a1}$ which implies that $T_{ba} = I.$

similarly $F_{b1} = 1F_{a1}$ (global coordinate).

We can then say that for a truss element, when a force (f_{a1}) acting at a joint (a) is transmitted to joint (b), the magnitude of the force remains the same(see figure below)

ROTATIONAL TRANSFORMATION

This represent the resolution of the elemental axial load unto the global reference axes.

For a frame element,

$F_a = \lambda f$

where

$$\lambda = \begin{bmatrix} \cos\alpha & -\sin\alpha & 0 \\ \sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \text{----------(14)}$$

Therefore

$$\begin{bmatrix} Fa_1 \\ Fa_2 \\ Fa_3 \end{bmatrix} = \begin{bmatrix} \cos\alpha & -\sin\alpha & 0 \\ \sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} f_{a1} \\ f_{a2} \\ f_{a3} \end{bmatrix}$$

which implies that

$$Fa = \lambda f$$

and

$$\lambda = \begin{bmatrix} \cos\alpha \\ \hline \sin\alpha \end{bmatrix} \qquad \text{------------------------(15)}$$

since for a truss elment, $f_{a2} = f_{a3} = Fa_3 = 0$.

In general,

$$\begin{matrix} Fa = \lambda f_a \\ Fb = \lambda f_b \end{matrix} \qquad \begin{bmatrix} Fa \\ Fb \end{bmatrix} \begin{bmatrix} \lambda & 0 \\ 0 & \lambda \end{bmatrix} \begin{bmatrix} f_b \\ f_b \end{bmatrix} \qquad \text{---------(16)}$$

Therefore

$$F = Rf$$

where $R = \begin{bmatrix} \lambda & 0 \\ 0 & \lambda \end{bmatrix}$

reffering to equation (16) and applying the principle of contragriedience

$$u = R^T U$$

$$F = Rf$$

$$f = ku$$

where $K = RkR^T$ -------------(17)

In general form

$$K = \begin{bmatrix} \lambda\lambda^T & -\lambda\lambda^T \\ -\lambda\lambda^T & \lambda\lambda^T \end{bmatrix} \qquad \text{------------------(18)}$$

from equation (15)

$$\lambda = \cos \quad \sin$$

therefore

$$\lambda\lambda^T = [\cos\alpha + \sin\alpha] = 1 \quad \text{-----------------(19)}$$

(b) Member Force Displacement Relationship For Space Truss:

(i) In local coordinates; This is exactly the same as in plane truss, since every element is same whether in plane or in space. Therefore

$$\begin{bmatrix} f_{a1} \\ f_{b1} \end{bmatrix} \alpha 3 \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{bmatrix} u_{a1} \\ u_{b2} \end{bmatrix}$$

(ii) In global coordinates;

- Translational Transformation; This is also the same as for an element in a plane truss ($F_b = 1f_a$)

- Rotational Transformation;

$$\begin{bmatrix} F_{a1} \\ F_{a2} \\ F_{a3} \end{bmatrix} = \begin{bmatrix} \cos\theta x & \cos\theta x' & \cos\theta x'' \\ \cos\theta y & \cos\theta y' & \cos\theta y'' \\ \cos\theta z & \cos\theta z' & \cos\theta z'' \end{bmatrix} \begin{bmatrix} f_{a1} \\ f_{a2} \\ f_{a3} \end{bmatrix}$$

$$F_a = \lambda \quad f_a$$

where

$$= \begin{bmatrix} \cos\theta x \\ \cos\theta y \\ \cos\theta z \end{bmatrix} \quad \text{------------------------------(20)}$$

In general $\qquad F = Rf$

where

$$R = \begin{matrix} \lambda & 0 \\ 0 & \lambda \end{matrix} = \begin{bmatrix} \cos\theta x & 0 \\ \cos\theta y & 0 \\ \cos\theta z & 0 \\ 0 & \cos\theta x \\ 0 & \cos\theta y \\ 0 & \cos\theta z \end{bmatrix}$$

Therefore

$$R = \begin{bmatrix} \cos\theta x & \cos\theta y & \cos\theta z & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\theta x & \cos\theta y & \cos\theta z \end{bmatrix}$$

But $F = K\ U$

$= RkR^{T}\ U$

Therefore $K = RkR^{T}$

where

$$k = a3 \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix}$$

which implies that

$$RkR^T = \begin{bmatrix} \cos\theta x & -\cos\theta x \\ \cos\theta y & -\cos\theta y \\ -\cos\theta x & \cos\theta x \\ -\cos\theta y & \cos\theta y \\ -\cos\theta z & \cos\theta z \end{bmatrix} \begin{bmatrix} \cos\theta x & \cos\theta y & \cos\theta z & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\theta x & \cos\theta y & \cos\theta z \end{bmatrix}$$

$$\begin{matrix} K_{AA} & \lambda\lambda^T & & & K_{AB} & -\lambda\lambda^T \end{matrix}$$

$$K = \begin{bmatrix} \cos^2\theta x & \cos\theta x\cos\theta y & \cos\theta x\cos\theta z & -\cos^2\theta x & -\cos\theta x\cos\theta y & -\cos\theta x\cos\theta z \\ \cos\theta x\cos\theta y & \cos^2\theta y & \cos\theta y\cos\theta z & -\cos\theta y\cos\theta x & -\cos^2\theta y & -\cos\theta y\cos\theta z \\ \cos\theta x\cos\theta z & \cos\theta y\cos\theta z & \cos^2\theta z & -\cos\theta z\cos\theta x & -\cos\theta z\cos\theta y & -\cos^2\theta z \\ -\cos^2\theta x & -\cos\theta y\cos\theta x & -\cos\theta z\cos\theta x & \cos^2\theta x & \cos\theta x\cos\theta y & \cos\theta x\cos\theta z \\ -\cos\theta x\cos\theta y & -\cos^2\theta y & -\cos\theta z\cos\theta y & \cos\theta x\cos\theta y & \cos^2\theta y & \cos\theta y\cos\theta z \\ -\cos\theta x\cos\theta z & -\cos\theta y\cos\theta z & -\cos^2\theta z & \cos\theta x\cos\theta z & \cos\theta y\cos\theta z & \cos^2\theta z \end{bmatrix}$$

$$K_{BA} = -\lambda\lambda^T \qquad\qquad K_{BB} = \lambda\lambda^T$$

To solve the member end forces in local coordinate after knowing the displacements, the following is done.

$$F' = Rf$$

$$f = ku$$

$$aB \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} u$$

But by principle of contrariedience

$$u = R^T J$$

Therefore

$$f = kR^T J$$

$$[f] = [\cos x \quad \cos y \quad \cos z \quad -\cos x \quad -\cos y \quad -\cos z] \begin{bmatrix} Ua_1 \\ Ja_2 \\ Ja_3 \\ Ub_1 \\ Jb_2 \\ Ub_3 \end{bmatrix} \text{-------(21)}$$

Where [Ja Ub], [Ja Ub], [Ja Ub] are the global displacements in the X, Y, Z directions at the a & b ends of member (i) corresponding to the global forces in the X, Y, & Z directions.

1.6.3. Design Of Members and Connectors

(a) Design of members:

In designing the members of every truss, it was decided that, all the members should be of uniform size, and mild steel hollow pipes should be utilized. The main reason for not carrying out an optimum design(which would normally have resulted in a lighter hyperperstatic structure) is the reduction achieved in construction and manufacturing costs when uniform sizes are employed. It therefore means that the members in most critical condition(which in these cases are the compression members) are chosen for design in respect of any truss.

The elastic method of design using the specialized strength limits depending on the slenderness ratio as per the British standard (BS 449) was employed.

(b) Design of truss joints (connectors)

The design of a truss joint could be seen as consisting of two seperate and independent steps

(i) geometric design,

(ii) connection design.

(i) geometric design:

The geometric design of the connectors largely depends on the arrangements, size and number of members meeting at the joint. Therefore the minimum dimension of the connector would be controlled by parameters such as the depth of truss, length of a member, shape and the cross-section of the member.

(ii) connection design:

connections can be broadly classified into two according to

* type of load transfer and

* method of connection.

* Basically the type of load transfer is influenced by the forces in the member being connected, which may subject the connection to either

-shear(axial and eccentric)

-tension(compression) or

-a combination of both.

In truss systems the members are axially loaded. In effect all the joints are loaded in tension or compression. "Unless buckling governs, tension type loading is more critical than compressive loading as far as strength of member and connection is concerned"(20). Therefore the joint with the highest tensile load was chosen for the design of connection in every truss.

* Method of connection could also be classified into two,

-welding(pressure, fusion, and brazing)

-Fastening (bolting and rivetting).

Welding; The process whereby pieces of metals are connected together by means of heat(with or without pressure) is known as welding. The main advantages of welding are economy, simplicity cum compacteness of connection and reduction in handling process during fabrication. A variety of welding processes exist, but in structural steel work, metal-arc welding is most utilized. There are different types of weld(fillet, grove or butt, plug and slot) but most commonly used are fillet and butt welds. Fillet welding is adopted in most designs of connectors and connections for the following reason.

-less skill is required

-the expensive preparation of edges necessary in butt welding is avoided.

-also avoided is the residual stresses accompanied with butt welds.

-roof structures carry static loads, therefore for strength reasons, fillet weld is comparable to butt welds (since fatigue would be absent).

Length of welds were calculated as follows:

$p = 115\ N/mm$

$f_w = P/A$

$A_w = l_w . a$

$a = 0.7t_m$

$f_{w(max)} = p$

therefore

$$l_{w(min)} = \frac{P}{f_{(max)}\, 0.7t_m} = \frac{P}{115(0.7)t} \qquad \text{-----------(22)}$$

where

p = allowable stress in fillet weld.

A_w = required area of weld

l_w = required length of weld

$l_{w(min)}$ = minimum l_w

a = throat of weld

t_m = thickness of the materials to be welded

f_w = stress in fillet weld

$f_{w(max)}$ = maximum f_w

P = the internal force in structural member

Fastening: The design procedure when fasteners are employed is basically the same. Because of the decreasing popularity of rivets, bolts were used in the design. Some advantages of bolts over rivets are

-unskilled labour could be utilized on site.

-demountable structures are possible.

-the costly and sophisticated equipments associated with rivetting is abscent. Only simple tools(spanners, wrenches, excetera) are required.

In the design of bolted connections, the number of bolts required depends on, the existing force in the structural member to be connected, and the allowable(design) stresses of the bolt. Bolts of grade designation 4.6, to BS 4190:1967 were employed in the design.

Generally, bolts are subjected to either shear, crushing(bearing) or tension.

If n = number of bolts reuired for a connection,

n > P/(nsh.A.p) for bolts in shear

n > P/(d t.p) for bolts in bearing.

n > P/(An.p) for bolts in tension.

where

P = design force applied to the connection.

p ,p ,p , = allowable stresses in shearing, crushing, and tension respectively.

A = gross area of bolts in shear

An = net area of bolts in tension.

t = thickness of plate being connected.

The bolts in the connectors designed were found to be in tension. Therefore, safe load values(tabulated) of black bolts between 16mm to

48mm were used.(see appendix 1).

This implies that if F is the safe load of bolt(s) used in KN at allowable stress of $130N/mm^2$ *,

therefore $F > P(KN)$

Another important important parameter in the design are the edge distances of holes to be drilled. The minimum distances as provided in table 21 of BS 449, part 2 1969, were utilized.

* Allowable stresses are taken from table 20 of BS 449, part 2: 1969.

1.6.4 Testing Of Connectors

An attempt at testing each type of the designed connector was made. This is to ascertain the stress distribution and the actual load carrying capacity of the connector. Though designed connector was carried out in some instances for both bolted and welded connector, only the welded connectors were examined for testing . This was done as welded designs are more acceptable as concerning tubular pipes(23).

To effectively carry out the test, a testing stand was designed so that axial load could be applied in the various directions as exist in the actual structure. Mild steel was chosen as the material for modelling, and the truss with the widest range of force distribution was chosen for test.

CHAPTER 2.

LOADINGS AND ANALYSIS

2.1 LOADINGS.

In analysing the double layer grids chosen for investigation, assumption was made that all loads(wind or dead) are acting normal to the roof surface as concentrated loads at the top joints.

Loadings were calculated based on the unit loadings provided in the Nigerian code of practice 1, part 3. The loadings were closely investigated and a moderate possible loading was chosen.

Mild steel with Youngs modulus of elasticity(E) of 2.0 x 10 KN/m was also chosen for use. Below are the types of loadings and those chosen for use in the analysis.

(a) Types of loadings:

:- Dead Load; This is described as the weight of a structure together with all its permanent fixtures and finishes.

(i) Emperically self weight of a steel truss is taken as (8 to 12)L N/mm^2 where L is the truss span

(ii) Roofing sheets: (including normal laps and fastening)

-concrete tile roofing	= 59.00 Kg/m^2
-asbestos cement sheets	= 16.11 ,,
-corrugated galvanised sheet	= 13.7 ,,
(iii) Linnings (approximate)	= 0.05 ,,
(iv) Glazings	= 29.29 ,,
(v) Purlins	= 10.20 ,,

:- Live load; This is the temporary loads expected on the truss. It is normally taken as all other loads other than dead and wind load. The Nigerian Code of practice specifies that, for flat and sloping roofs(with limited acess) up to an angle of 65;

Live load = 25 Kgf per horzontal square meter.

:- Wind load; The wind pressure specified by the Nigerian codes of practice apply to roof structures and therefore may not be used for radio masts, electrical transmission or flood lights towers which are also space trusses. Wind pressure is used to calculate the wind loads which varies with

-the wind velocity at site.

-the shape and slope of structure

-the density of air and protection offered against wind by the surrounding structures.

-height of the structure above the mean ground level.

(b) Loadings Chosen For Use

For all the structures analysed the following loadings were used:

-Dead Load;

self weight of truss	(0.008 to 0.012)L*
roofing sheets	0.134
Linnins	0.05
purlins	0.10
steel	77.00

* L is the average span of the roof

- Live load;

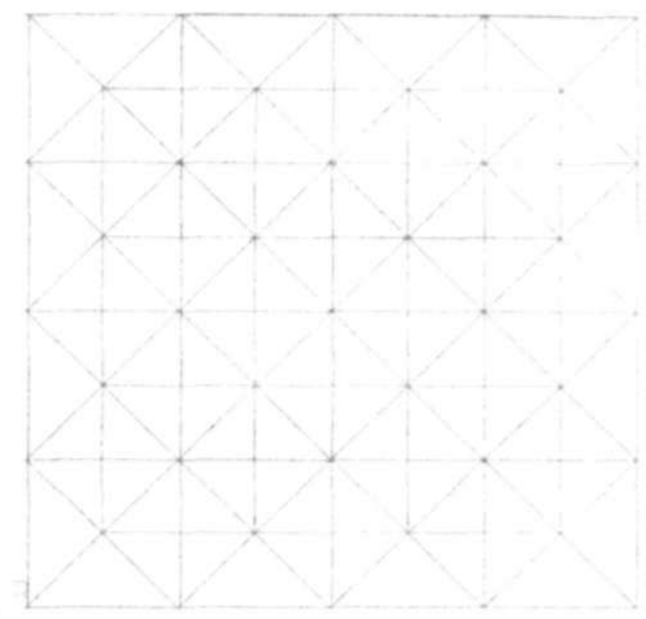
for roof with limited acess	0.25

-wind load;

wind velocity	144 KM/hR
shape factor	1.0
Inclination factor	-1.5

(c) **Trusses Analysed:** The following 8 pages show all the trusses, their plan sketches, and calculations of loads per top joint. At the end a comprehensive table of the calculated loadings and load combinations is drawn out. (see table - 2, page 54).

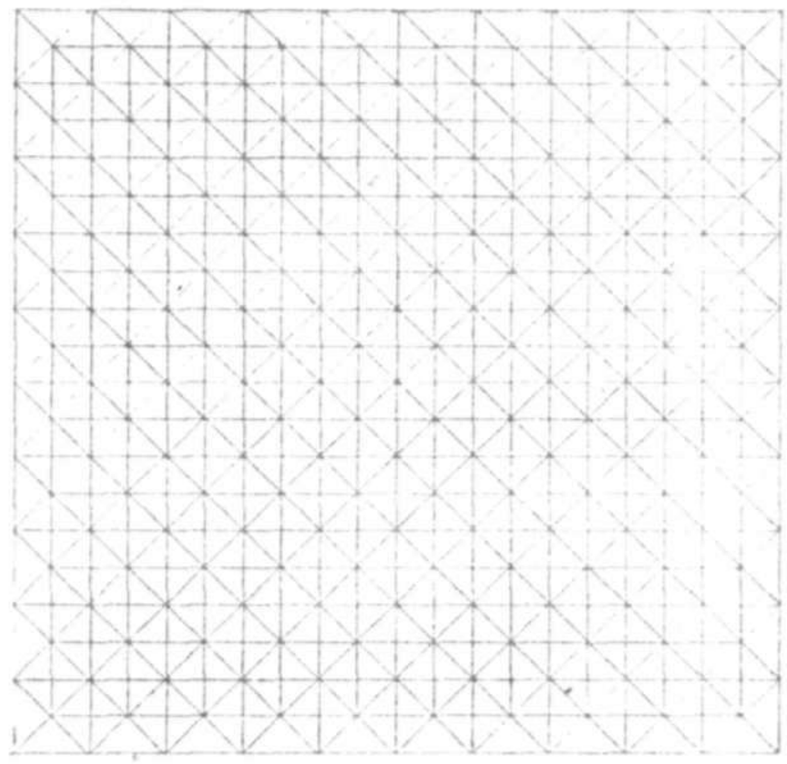
DOUBLE LAYER SPACE GRID - I



DIMENSION = 10.0M X [illegible]

** THE DIMENSION OF THE GRID IS IN METERS BOTH WAYS

DOUBLE LAYER SPACE GRID II



DIMENSION = 10.0M X 10.0M
SPACES PER TOP JOINT = 1.0M
TRIANGLE SIDE = 1.0M

LOADINGS

DEAD LOAD-
- SELF WEIGHT = 0.0001 KN/MM = 0.03 KN/MM^2
- ROOFING SHEETS = 0.134 ..
- LINNINGS = 0.[illegible] ..
- PURLINS = 0.[illegible] ..

0.354

LIVE LOAD- = 0.25 ..

WIND LOAD-
SHAPE FACTOR = 1.0, WIND SPEED 141 KM/HR
INCLINATION FACTOR = -1.5

HEIGHT(M)	WIND PRESSURE(KG/M^2)	WIND LOAD(KN/M^2)
5.0	95.67	[illegible]
10.0	115.00	[illegible]
15.0	140.00	[illegible]

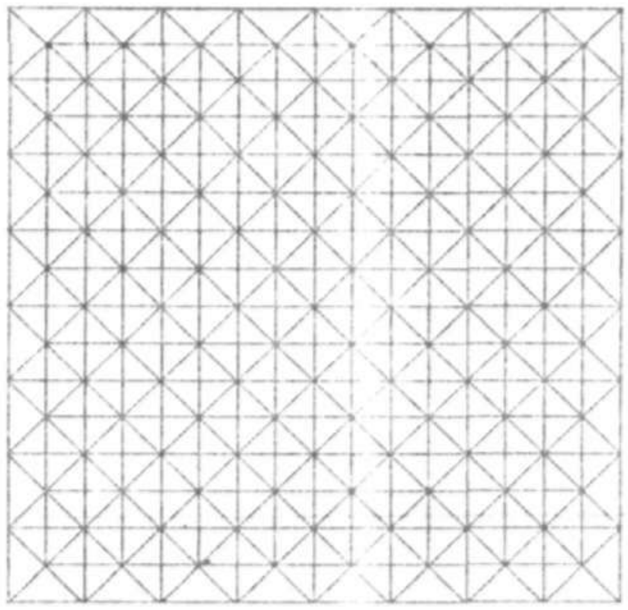
LOAD COMBINATIONS
(A) DEAD + LIVE LOAD = (0.354 + 0.[illegible])1.0 = 0.61 KN
(B) DEAD + WIND LOAD = (0.354 - 1.[illegible])1.0 = -0.34 ..
-1.[illegible] = -1.37 ..
-[illegible] = -1.74 ..
(C) DEAD + WIND + LIVE = (0.604 - [illegible])1.0 = -0.63 ..
-[illegible] = -1.12 ..
-2.[illegible] = -1.43 ..

DESIGN LOADS-

H(M)	LOADS(KN)
5.0	0.61, [illegible]
10.0	0.61, -1.[illegible]
15.0	0.61, -1.74

** THE DIMENSION OF THE GRID IS IN METERS BOTH WAYS

DOUBLE LAYER SPACE CRID - II



DIMENSION
AREA PER TOP JOINT = 6.25M
TRIANGLE SIDE = 2.5M

LOADINGS

DEAD LOAD-
- -SELF WEIGHT = 0.012L KN/MM = 0.24 KN/MM²
- -ROOFING SHEETS = 0.134 ..
- -LINNINGS = 0.[illegible] ..
- -PURLING = 0.10 ..

0.524

LIVE LOAD- = 0.25 ..

WIND LOAD-
SHAPE FACTOR = 1.0, WIND SPEED = 144 KM/HR
INCLINATION FACTOR = -1.5

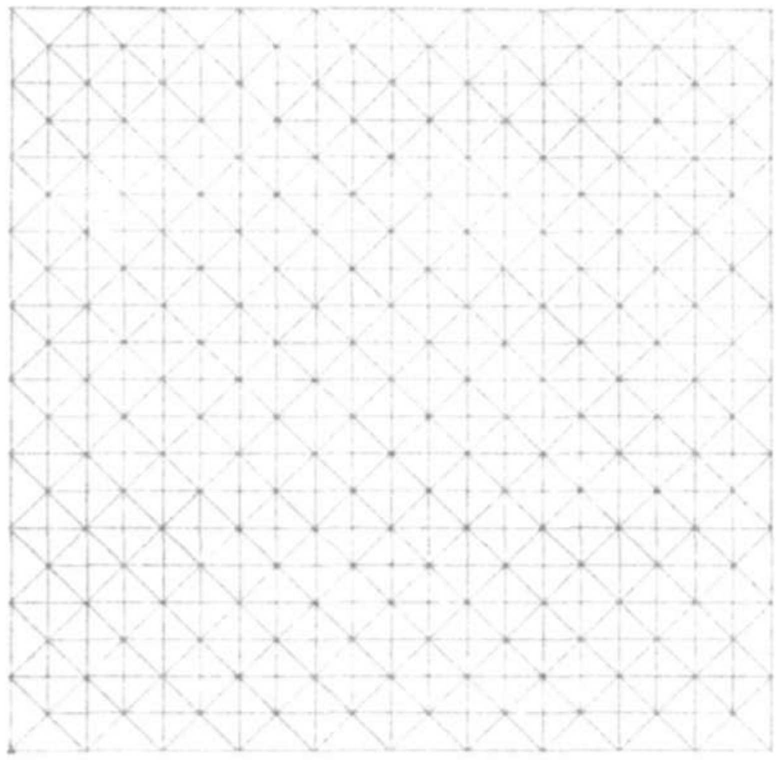
HEIGHT(M)	WIND PRESSURE(KG/M)	WIND LOAD(KN/M²)
5.0	86.67	-1.30
10.0	115.00	-1.73
15.0	140.00	-2.[illegible]0

LOAD COMBINATIONS
(A) DEAD + LIVE LOAD = (0.52 + 0.25)6.25 = 4.84
(B) DEAD + WIND LOAD = (0.52 - 1.30)6.25 = -4.85
-1.73)6.25 = -7.53
-2.10)6.25 = -9.95
(C) DEAD + WIND + LIVE = (0.77 - 1.3)6.25 = -3.2
-1.73)6.25 = -5.9
-2.10)6.25 = -8.2

DESIGN LOADS-

H(M)	LOADS(KN)
5.0	4.94,-4.85
10.0	4.94,-7.53
15.0	4.94,-9.95

DOUBLE LAYER SPACE GRID - IV



DIMENSION = 20.0M X 20.0M
[illegible]
[illegible]

LOADINGS

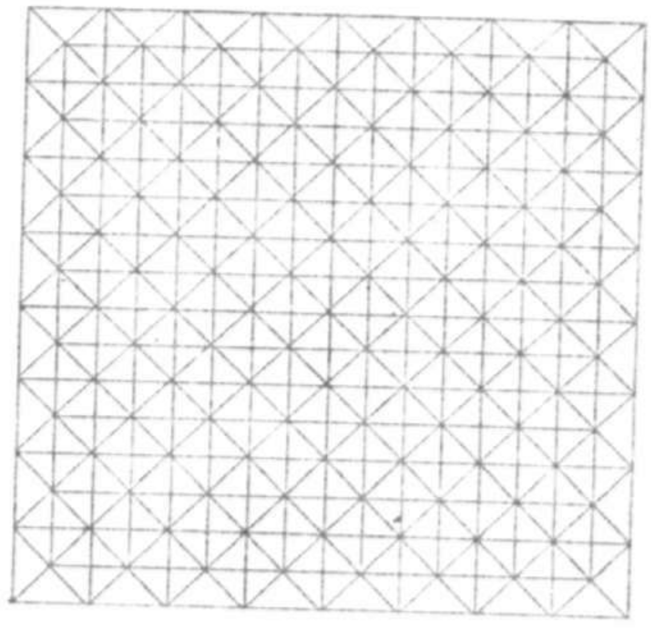
DEAD LOAD-
[illegible] KN/M²
[illegible]

WIND LOAD-
[illegible]

HEIGHT (M)	WIND PRESSURE (N/M²)	WIND LOAD (KN/M²)
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

LOAD COMBINATIONS
[illegible]

DOUBLE LAYER SPACE GRID- VIII



DIMENSION = 30.0M X 90.0M
AREA PER TOP JOINT = 100 SM
TRIANGLE SIDE = 10.0M

LOADINGS

DEAD LOAD-
-SELF WEIGHT = 0.[illegible] KN/MM = 0.06 KN/M²
-ROOFING SHEETS = 0.17[illegible]
-LININGS = 0.[illegible]
-PURLINS = 0.1[illegible]
[illegible]

LIVE LOAD- = 0.25 ..

WIND LOAD-
SHAPE FACTOR = 1.0, WIND SPEED = 144 KM/HS
INCLINATION FACTOR = [illegible]

HEIGHT(M)	WIND PRESSURE(KG/M²)	WIND LOAD(KN/M²)
5.0	96.67	[illegible]
10.0	115.00	[illegible]
15.0	140.00	[illegible]

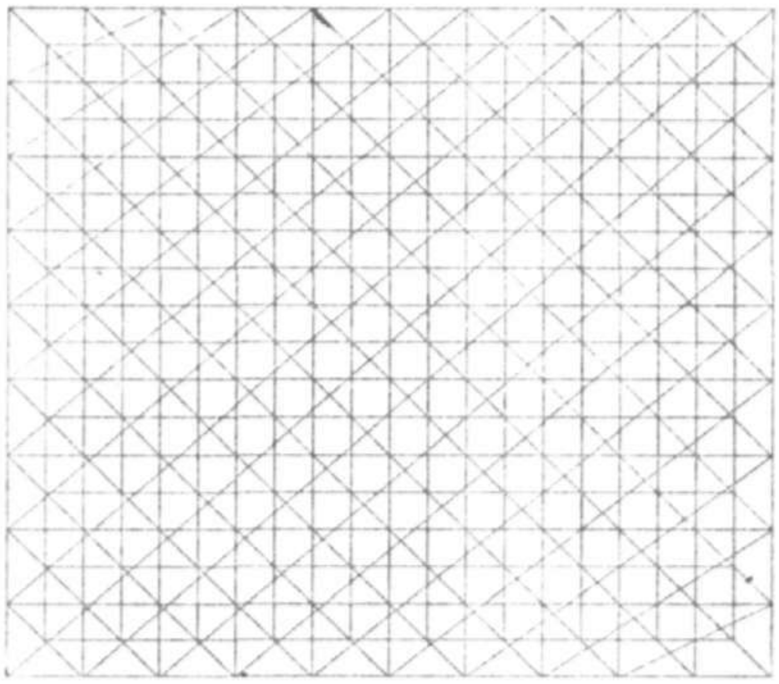
LOAD COMBINATIONS
(A) DEAD + LIVE LOAD = (1.24 + 0.25)100.0 = 149.4 KN
(B) DEAD + WIND LOAD = (1.24 - 1.[illegible])100.0 = [illegible]
-1.73)100.0 = -49.6 ..
-2.10)100.0 = [illegible] ..
(C) DEAD + WIND + LIVE = (1.49 - 1.[illegible])100.0 = [illegible] ..
-1.73)100.0 = [illegible] ..
-2.10)100.0 = [illegible] ..

DESIGN LOADS-

H(M)	LOADS(KN)
5.0	143.4, 5.6
10.0	143.4,-49.6
15.0	143.4, 3[illegible].6

** THE DIMENSION OF THE GRID IS IN METERS BOTH WAYS

DOUBLE LAYER SPACE GRID - IX



DIMENSION = 100.0M X 00.0M
AREA PER TOP JOINT = 100.0M
TRIANGLE SIDE = 10.0M

LOADINGS

DEAD LOAD-
- -SELF WEIGHT = [illegible] = 1.2 KN/M²
- -ROOFING SHEETS = 0.134 ..
- -LINNINGS = 0.05 ..
- -PURLINS = 0.10 ..

148.4

LIVE LOAD- = 0.25 ..

WIND LOAD-
SHAPE FACTOR = 1.0, WIND SPEED = 144 KM/HR
INCLINATION FACTOR = [illegible]

HEIGHT(M)	WIND PRESSURE(KG/M)	WIND LOAD(KN/M²)
5.0	96.67	-1.30
10.0	115.[illegible]	-1.73
15.0	140.[illegible]	-2.10

LOAD COMBINATIONS
(A) DEAD + LIVE LOAD = (148.4 + 0.25)100.0 = 173.4 KN
(B) DEAD + WIND LOAD = (148.4 - 1.30)100.0 = -18.4 ..
(-1.73)100.0 = -24.6 ..
(-2.10)100.0 = -61.6 ..

DESIGN LOADS-

H(M)	LOADS(KN)
5.0	173.4,-18.4
10.0	173.4,-24.6
15.0	173.4,-61.6

** THE DIMENSION OF THE GRID IS IN METERS BOTH WAYS

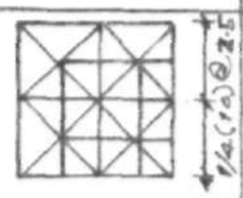
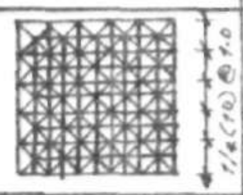
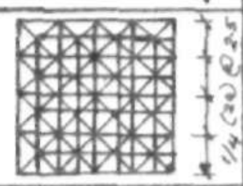
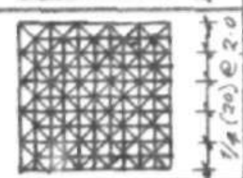
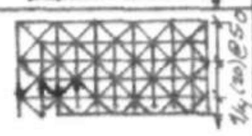
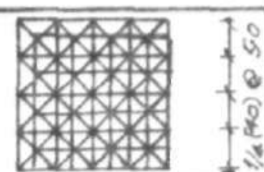
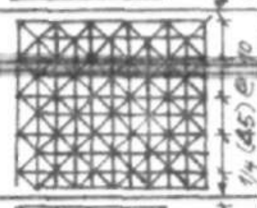
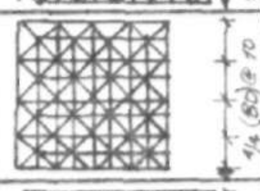
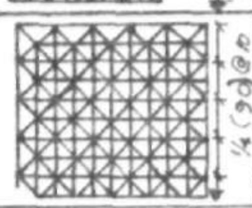
	DIAGRAM (one_quarter shown)	DIMENSION (m x m x m)	H (m)	D·L (KN)	L·L (KN)	W·L (KN)	LOAD COMBINATIONS 1 (KN)	2 (KN)	3 (KN)
1		10 x 10 x D, D = 1·5, = 0·5	5	2·53	1·56	-8·13	4·1	-5·6	-4·03
			10	,,	,,	-10·81	,,	-8·28	-6·71
			15	,,	,,	-13·13	,,	-10·59	-9·03
2		10x 10 x D, D = -0·5	5	0·36	0·25	-1·30	0·61	-0·49	-0·69
			10	,,	,,	-1·73	,,	-1·37	-1·12
			15	,,	,,	-2·10	,,	-1·74	-1·49
3		20 x 20 x D, D = 2·0	5	3·28	1·56	-8·13	4·84	-4·85	-3·29
			10	,,	,,	-10·81	,,	-7·53	-5·97
			15	,,	,,	-13·13	,,	-9·85	-8·29
4		20 x 20 x D, D = -1·0	5	1·78	1·60	-5·2	2·78	-3·42	-2·4
			10	,,	,,	-6·9	,,	-5·14	-4·1
			15	,,	,,	-8·4	,,	-6·62	-5·6
5		40x 20 x d, d = 2D, D = -1·4	5	9·34	4·0		13·34	-11·46	-7·46
			10	,,	,,		,,	-18·34	-14·34
			15	,,	,,		,,	-24·26	-20·26
6		40 x 40x D, D = 2·0	5	19·1	6·25	-32·5	25·34	-13·4	-7·2
			10	,,	6·25	-43·25	,,	-24·15	-17·9
			15	,,	,,	-52·5	,,	-33·2	-27·15
7		50 x 50 x D, D=	5	22·1	6·25	-32·5	28·35	-10·4	-4·15
			10	,,	,,	-43·25	,,	-21·15	-14·9
			15	,,	,,	-52·5	,,	-30·4	-24·15
8		80x 80x D, D = 2·5, = -2·5, = -1·6	5	124·4	25	-130·0	149·0	-5·6	19·4
			10	,,	,,	-173·0	,,	-48·6	-23·6
			15	,,	,,	-210·0	,,	-85·6	-60·6
9		100 x 90 x D, D = 3·0, = -3·0, = -2·0	5	148·4	25	-130·0	173·4	-18·4	
			10	,,	,,	-173·0	,,	-24·6	
			15	,,	,,	-210·0	,,	-61·6	

TABLE 2: OVERALL LOAD COMBINATION

D·L = DEAD LOAD
L·L = LIVE LOAD
W·L = WIND LOAD
1 = D·L L·L
2 = D·L W·L
3 = D·L L·L W·L

2.2 Computer Programme Used For The Analysis Of The Trusses

The programme was modified and adapted from that originally written by Harrison, H. B(25). The modified programme consist of a main programme and six subroutines. The main programme serves as to call the subroutines in the required sequence.

Subroutines:

(a) DATA; This subroutine supplies the computer with the initial data describing the geometry of the structure and the constraints of every joint.

(b) Kpp; This subroutines assembles the stiffeness matrix and with the aid of the stated constraints reduce the matrix to a constrained stiffeness matrix.

(c) LOAD; This subroutine reads the data file for the loaded joints and their magnitude in the three directions.

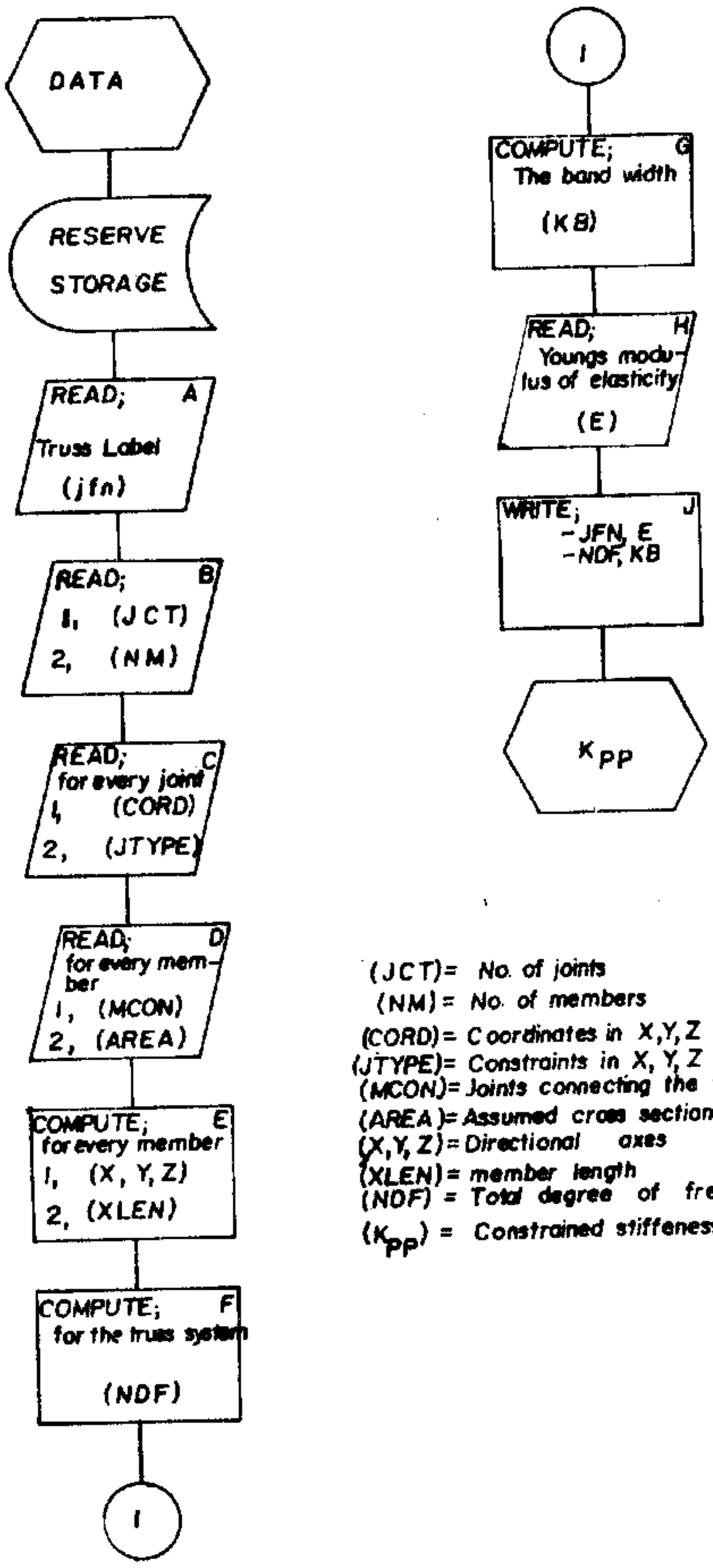
(d) SOLVER; The resulting matrix is solved by using the band with to read in the elements of the matrix. The Tridec elimination process has been used.

(e) DISPL; The required joint displacements results as the solution obtained from the solver.

(f) FORCES; Member end forces in the local coordinates are calculated and hence the reactions at every joint in the three directions.

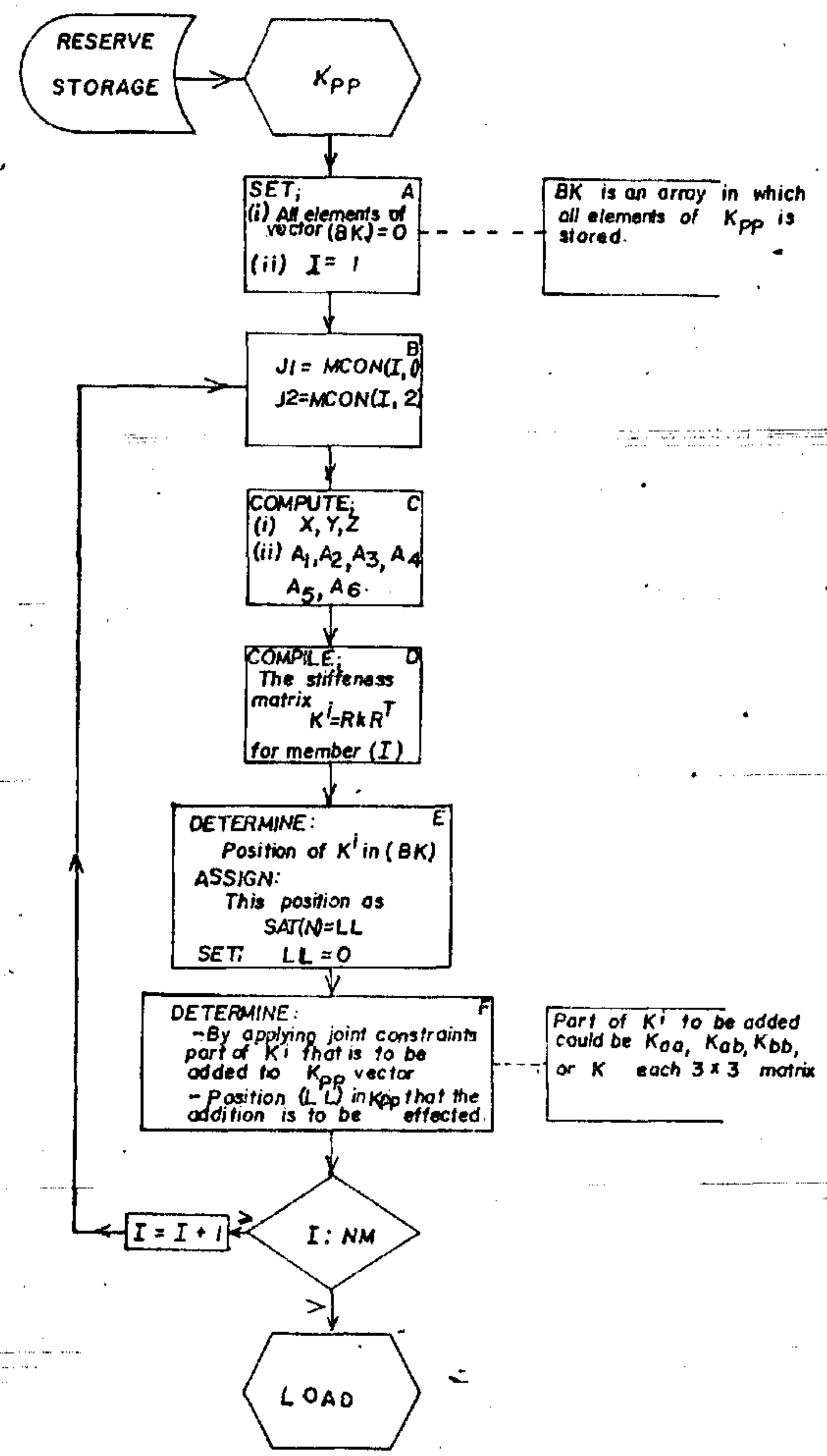
Flow charts has been constructed to explain the programme employed.For every subroutine, a general flow chart is drawn showing the broad steps taken in alphebetical order.

(1). SUBROUTINE DATA

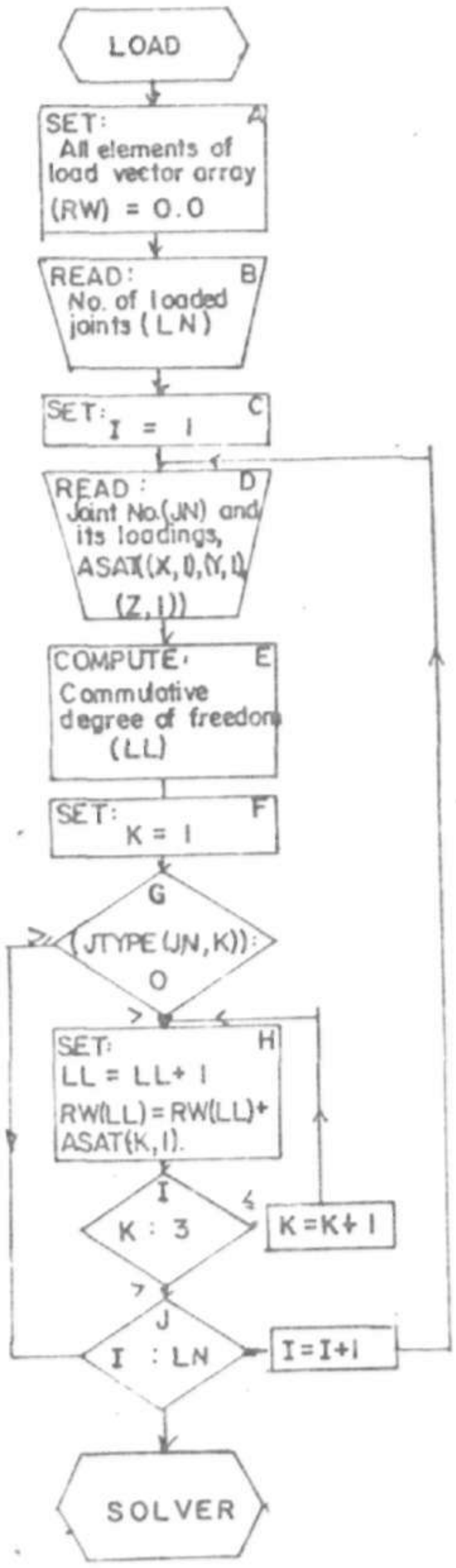


(JCT) = No. of joints
(NM) = No. of members
(CORD) = Coordinates in X, Y, Z directions
(JTYPE) = Constraints in X, Y, Z directions
(MCON) = Joints connecting the two ends of a member
(AREA) = Assumed cross sectional area of a member
(X, Y, Z) = Directional axes
(XLEN) = member length
(NDF) = Total degree of freedom
(K_{pp}) = Constrained stiffeness matrix

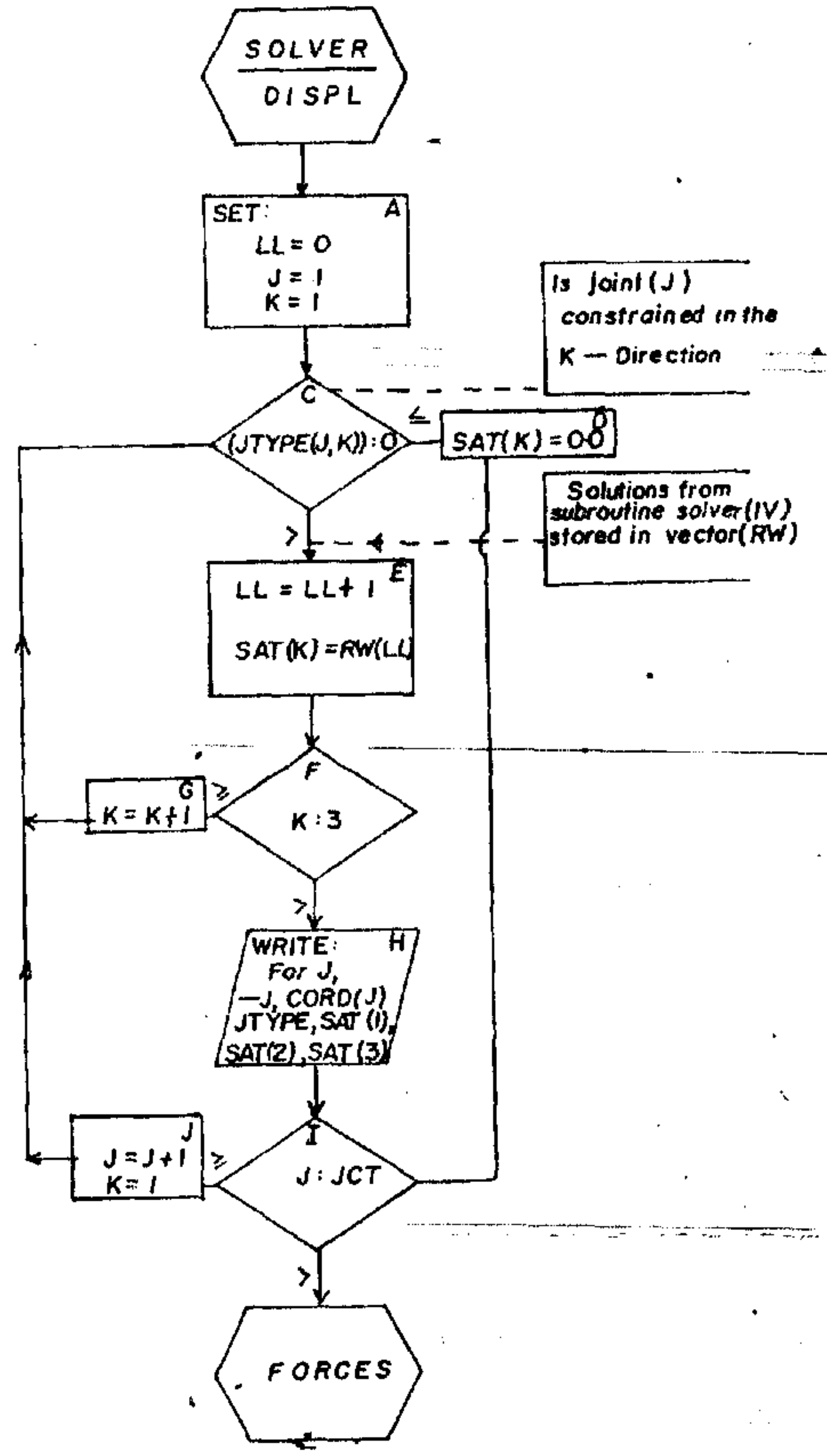
SUBROUTINE KPP



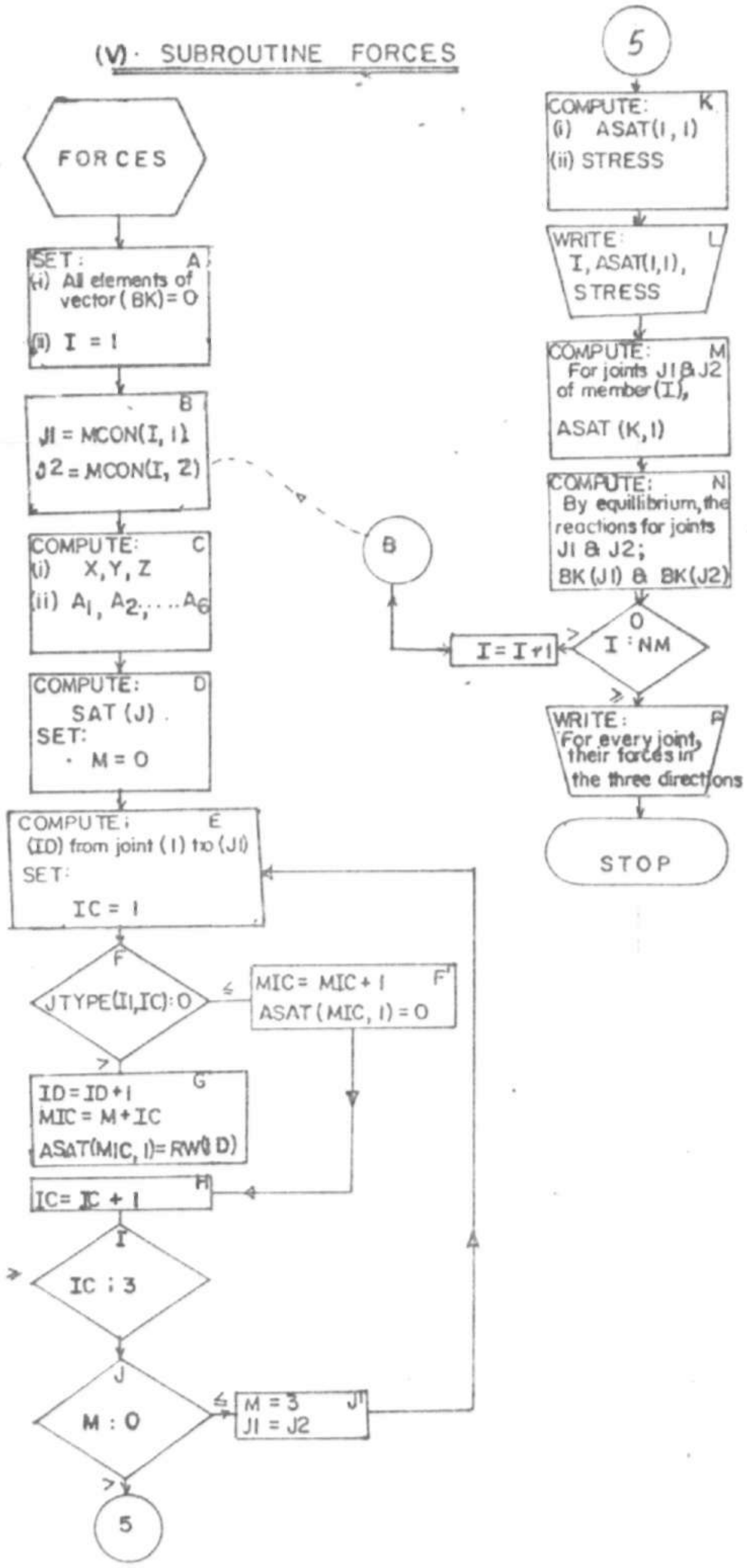
(iii) SUBROUTINE LOAD



(IV). SUBROUTINE DISPL



(V) · SUBROUTINE FORCES



CHAPTER 3.

DESIGN OF MEMBERS.

The results of the analysed trusses are presented below. A typical quarto-isometric view is futher drawn to show the axial member force and distribution pattern for the smallest spanned truss (see figures 15, 16). The deflection pattern for two of the trusses were also drawn.

The design of members based on these results were futher tabulated for each case(see table 3). While the chosen sections and the total volume of material per truss was also tabulated(see table 4).

TABLE 3: TABLE OF ANALYSIS RESULT AND DESIGN OF MEMBERS.

1. Dimension = 10.0m x 10.0m

No. of joints = 41

No. of members = 128

ANALYSIS LABELS	H (m)	MAX. MEMBER FORCES(kN) Tension	Compression	L (m)	DESIGN C.S.A	λ=L/r	STRESSES(N/mm²) Tensile	Compressive	SELF WEIGHT Assumed	Actual
101,102	5.0	7.77	-5.33	2.32	390	209	19.92	-14.95	12.0	9.20
101,103	10.0	11.49	-5.33	2.32	390	209	29.50	-14.95	12.0	9.20
101,104	15.0	14.71	-6.12	2.50	390	226	37.72	-15.69	12.0	9.20
105,106	5.0	37.29	-27.97	2.50	848	156	43.97	-32.98	12.0	20.20
105,107	10.0	55.14	-27.97	2.50	848	156	65.0	-32.98	12.0	20.20
105,108	15.0	70.59	-27.97	2.50	848	156	33.24	-32.98	12.0	20.20

2. Dimension = 10.0m x 10.0m

No. of joints = 221

No. of members = 300

ANALYSIS LABELS	H (m)	Tension	Compression	L (m)	C.S.A	λ=L/r	Tensile	Compressive	Assumed	Actual
444,445	5.0	3.10	-12.46	1.00	215	102	37.44	-57.70	3.0	12.37
444,446	10.0	3.10	-13.2	1.00	327	103	24.7	-55.6	3.0	13.74
444,447	15.0	3.96	-22.11	1.00	327	103	27.4	-67.6	3.0	13.74

H = Height of the truss above ground level

L = Length of member used in the design

C.S.A. = Cross sectional area of the designed member

L/r = Slenderness ratio of designed member

TABLE 3: CONTINUED

3. Dimension = 20.0m x 20.0m

No. of joints = 145

No. of members = 512

ANALYSIS LABELS	H (m)	MAX. MEMBER FORCES(KN) Tension	Compression	L (m)	DESIGN C.S.A	$\lambda=L/r$	STRESSES(N/mm^2) Tensile	Compressive	SELF WEIGHT Assumed	Actual
201,202	5.0	26.07	-26.02	2.5	848	156	30.70	-30.00	96.0	86.40
201,203	10.0	40.48	-26.02	2.5	848	156	47.70	-30.00	96.0	86.40
201,204	15.0	52.96	-26.81	2.5	848	156	62.45	-31.60	96.0	86.40
205,206	5.0	36.84	-36.76	2.5	905	147	40.70	-40.62	96.0	92.20
205,207	10.0	57.19	-36.76	2.5	905	147	63.20	-40.62	96.0	92.20
205,208	15.0	75.34	-36.72	2.5	905	147	83.25	-40.62	96.0	92.20

4. Dimension = 20.0m x 20.0m

No. of joints = 221

No. of members = 800

555,556	5.0	36.36	-45.35	2.0	848	125	43.47	-53.47	64.0	97.24
555,557	10.0	36.36	-63.15	2.0	1020	104	36.14	-66.82	64.0	116.98
555,558	15.0	36.36	-84.53	2.0	1210	89	29.40	-69.90	64.0	138.30

5. Dimension = 20.0m x 40.0m

No. of joints = 120

No. of members = 416

410,420	5.0	125.60	-146.20	4.00	2510	113	50.04	-58.30	288.0	285.66
410,430	10.0	201.01	-146.20	4.00	2510	113	80.10	-58.30	288.0	285.66
410,440	15.0	265.33	-146.20	4.00	2510	113	105.93	-58.30	288.0	285.66

TABLE 3: CONTINUED

6. Dimension = 40.0m x 40.0m

No. of joints = 145

No. of members = 512

ANALYSIS LABELS	H (m)	MAX. MEMBER FORCES (kN) Tension	Compression	L (m)	DESIGN C.S.A	$\lambda = L/r$	STRESSES (N/mm^2) Tensile	Compressive	SELF WEIGHT Assumed	Actual
401,402	5.0	133.10	-251.31	5.00	3470	103	38.40	-72.00	768.0	618.32
401,403	10.0	139.39	-251.31	5.00	3470	103	40.30	-72.00	768.0	618.32
401,404	15.0	331.77	-251.31	5.00	3470	103	95.60	-72.00	768.0	618.32
405,406	5.0	147.01	-261.69	5.00	3470	103	42.40	-75.40	768.0	618.32
406,407	10.0	249.30	-261.69	5.00	3470	103	71.30	-75.40	768.0	618.32
406,408	15.0	344.78	-261.69	5.00	3470	103	99.40	-75.40	768.0	618.32

7. Dimension = 50.0m x 45.0m

No. of joints = 200

No. of members = 720

501,502	5.0	450.62	-186.63	5.0	3470	103	129.86	-49.90	1350.0	871.64
501,503	10.0	450.62	-336.33	5.0	4100	87	109.90	-82.00	1350.0	1029.49
501,504	15.0	450.62	-482.35	5.0	4600	78	97.96	-104.86	1350.0	1155.77
511,512	5.0	693.23	-306.00	5.0	4600	78	151.79	-66.30	1350.0	1115.73
511,513	10.0	693.23	-521.20	5.0	5230	68	133.50	-99.66	1350	1263.19
511,514	15.0	693.23	-747.40	5.0	6510	55	107.25	-114.81	1350	1579.27

TABLE 3: CONTINUED

8. Dimension = 80.0m x 90.0m

No. of joints = 145

No. of members = 512

ANALYSIS LABELS	H (m)	MAX. MEMBER FORCES(KN) Tension	Compression	L (m)	DESIGN C.S.A	$\lambda = L/r$	STRESSES(N/mm^2) Tensile	Compressive	SELF WEIGHT Assumed	Actual
801,802	15.0	1313.33	-2292.13	10.00	20300	70	64.70	-112.90	5472.0	7002.40
810	15.0	2376.60	-1199.05	10.00	17000	95	139.80	-70.50	5472.0	5864.30**
811	15.0	3649.00	-1908.60	10.00	25100	71	145.40	-76.00	5472.0	8534.70

10. Dimension = 100.0m x 90.0m

No. of joints = 200

No. of members = 720

1001	15.0	1524.56	-3444.86	10.00	32100	72	47.49	-107.32	10800.0	15732.64
1101	15.0	3538.94	-1541.76	10.00	25100	71	142.99	-61.42	10800.00	12301.50
1201	15.0	3230.50	-2341.10	10.00	22700	71	144.52	-103.13	10800.0	10916.308**

** Signifies the truss with the best dimension ratio.

TABLE 4. COMPREHENSIVE RESULT OF ALL THE ANALYSED/DESIGNED TRUSSES

	DIMENSION (m x m)	NJ/NM	DT (m)	H (m)	TRIANGLE SIDE(m)	SUPPORT TYPE	MAX. MEMBER FORCES(KN) TENSION	COMPRESSION	DESIGNED SECTION(t,D:A)	TOTAL VOL. (m^3)
1(a)	10x10	41/128	1.5	5.0	2.5	2	7.77	-5.83	4 , 35 : 390	0.120
(b)	10x10	41/128	1.5	10.0	2.5	2	11.49	-5.83	4 , 35 : 390	0.120
(c)	10x10	41/128	1.5	15.0	2.5	2	14.71	-5.12	4 , 35 : 390	0.120
(d)	10x10	41/128	0.5	5.0	2.5	2'	37.29	-27.97	6 , 51 : 348	0.262
(e)	10x10	41/128	0.5	10.0	2.5	2'	55.14	-27.97	6 , 51 : 348	0.262
(f)	10x10	41/128	0.5	15.0	2.5	2'	70.59	-27.23	6 , 51 : 348	0.262
(g)	10x10	221/300	0.5	5.0	1.0	1	8.1	-12.46	2.5, 30 : 216	0.161
(h)	10x10	221/300	0.5	10.0	1.0	1	8.1	-13.16	4 , 30 : 327	0.244
(i)	10x10	221/300	0.5	15.0	1.0	1	8.96	-22.11	4 , 30 : 327	0.244
2(a)	20x20	145/512	2.0	5.0	2.5	2	26.07	-26.02	6 , 51 : 348	1.122
(b)	20x20	145/512	2.0	10.0	2.5	2	40.48	-26.02	6 , 51 : 348	1.122
(c)	20x20	145/512	2.0	15.0	2.5	2	52.96	-26.31	6 , 51 : 348	1.122
(d)	20x20	145/512	2.0	5.0	2.5	2'	36.84	-36.76	6 , 54 : 905	1.198
(e)	20x20	145/512	2.0	10.0	2.5	2'	57.19	-36.76	6 , 54 : 905	1.198
(f)	20x20	145/512	2.0	15.0	2.5	2'	75.34	-36.76	6 , 54 : 905	1.198
(g)	20x20	221/300	-1.0	5.0	2.0	1	36.36	-45.35	6 , 51 : 348	1.266
(h)	20x20	221/300	-1.0	10.0	2.0	1	36.36	-68.15	6 , 60 : 1020	1.523
(i)	20x20	221/300	-1.0	15.0	2.0	1	36.36	-84.53	6 , 70 : 1210	1.807

NJ/NM = Number of joints/Number of members

DT = Dept of the truss

H = Height of truss above ground level

TOTAL VOL. = Total volume of steel required for the structural members

t , D : A = Thickness , Diameter : & cross sectional area of members of the truss.

TABLE 4. CONTINUED

	DIMENSION (m x m)	NJ/NM	DT (m)	H (m)	TRIANGLE SIDE(m)	SUPPORT TYPE	MAX. MEMBER FORCES(KN) TENSION	COMPRESSION	DESIGNED SECTION(t,D:A)	TOTAL VOL. (m^3)
3(a)	40x20	120/416	-1.4	5.0	4.0	3	192.1	-223.60	8 ,108 : 2510	3.719
(b)	40x20	120/416	-1.4	10.0	4.0	3	307.4	-223.60	8 ,108 : 2510	3.719
(c)	40x20	120/416	-1.4	15.0	4.0	3	406.6	-223.60	8 ,108 : 2510	3.719
4(a)	40x40	145/512	2.0	5.0	5.0	2	133.10	-251.81	8 ,146 : 3470	8.050
(b)	40x40	145/512	2.0	10.0	5.0	2	239.89	-251.81	8 ,146 : 3470	8.050
(c)	40x40	145/512	2.0	15.0	5.0	2	331.77	-251.81	8 ,146 : 3470	8.050
(d)	40x40	145/512	2.0	5.0	5.0	2'	147.01	-261.69	8 ,146 : 3470	8.050
(e)	40x40	145/512	2.0	10.0	5.0	2'	249.30	-261.69	8 ,146 : 3470	8.050
(f)	40x40	145/512	2.0	15.0	5.0	2'	344.78	-261.69	8 ,146 : 3470	8.050
5(a)	50x45	200/720	-2.0	5.0	5.0	1	450.62	-186.63	8 ,146 : 3470	11.320
(b)	50x45	200/720	-2.0	10.0	5.0	1	336.38	-336.38	8 ,171 : 4100	13.370
(c)	50x45	200/720	-2.0	15.0	5.0	1	450.62	-432.35	8 ,191 : 4600	15.010
(d)	50x45	200/720	-1.25	5.0	5.0	1	698.23	-305.00	8 ,191 : 4600	14.490
(e)	50x45	200/720	-1.25	10.0	5.0	1	698.23	-521.21	8 ,216 : 5230	16.470
(f)	50x45	200/720	-1.25	15.0	5.0	1	698.23	-747.40	8 ,267 : 6510	20.510
6(a)	80x80	145/512	2.5	5,10,15	10.0	2	1313.33	-2292.18	16 ,419 : 30300	90.940
(b)	80x80	145/512	-2.5	5,10,15	10.0	1	2376.6	-1199.05	13 ,318 : 17000	76.160
(c)	80x80	145/512	1.6	5,10,15	10.0	1	3649.8	-1908.60	20 ,419 : 25100	110.34
7(a)	100x90	200/720	3.0	5,10,15	10.0	2	1524.56	-3444.36	26 ,419 : 32100	204.32
(b)	100x90	200/720	-3.0	5,10,15	10.0	1	3588.94	-1541.76	20 ,419 : 25100	159.76
(c)	100x90	200/720	-2.0	5,10,15	10.0	1	3280.60	-2341.1	13 ,419 : 22700	141.71

FIG. 14

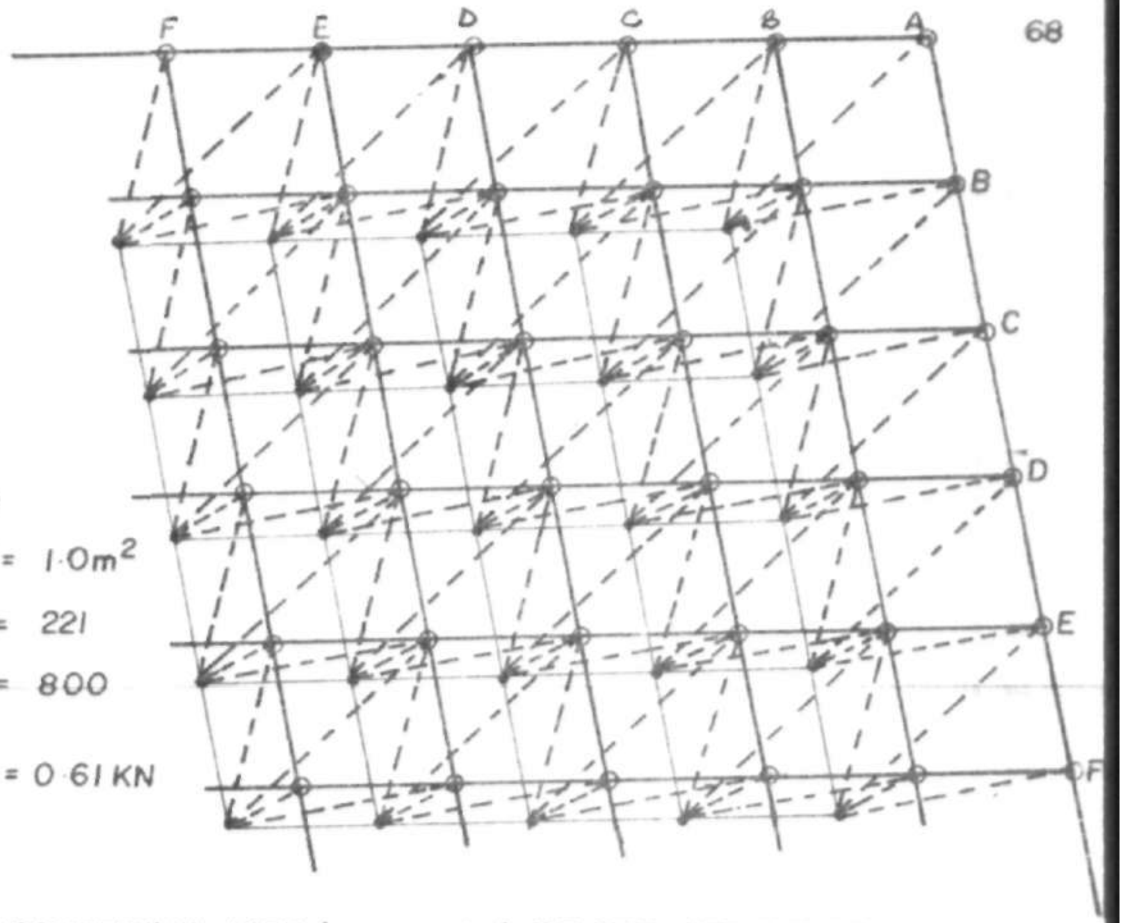
DIMENSION; 10 x 10 m

AREA PER TOP JOINT = 1·0m²

Nº OF JOINTS = 221

Nº OF MEMBERS = 800

POINT LOAD PER TOP JOINT = 0·61 KN



ISOMETRICAL VIEW (one quarter) OF THE STRUCTURE

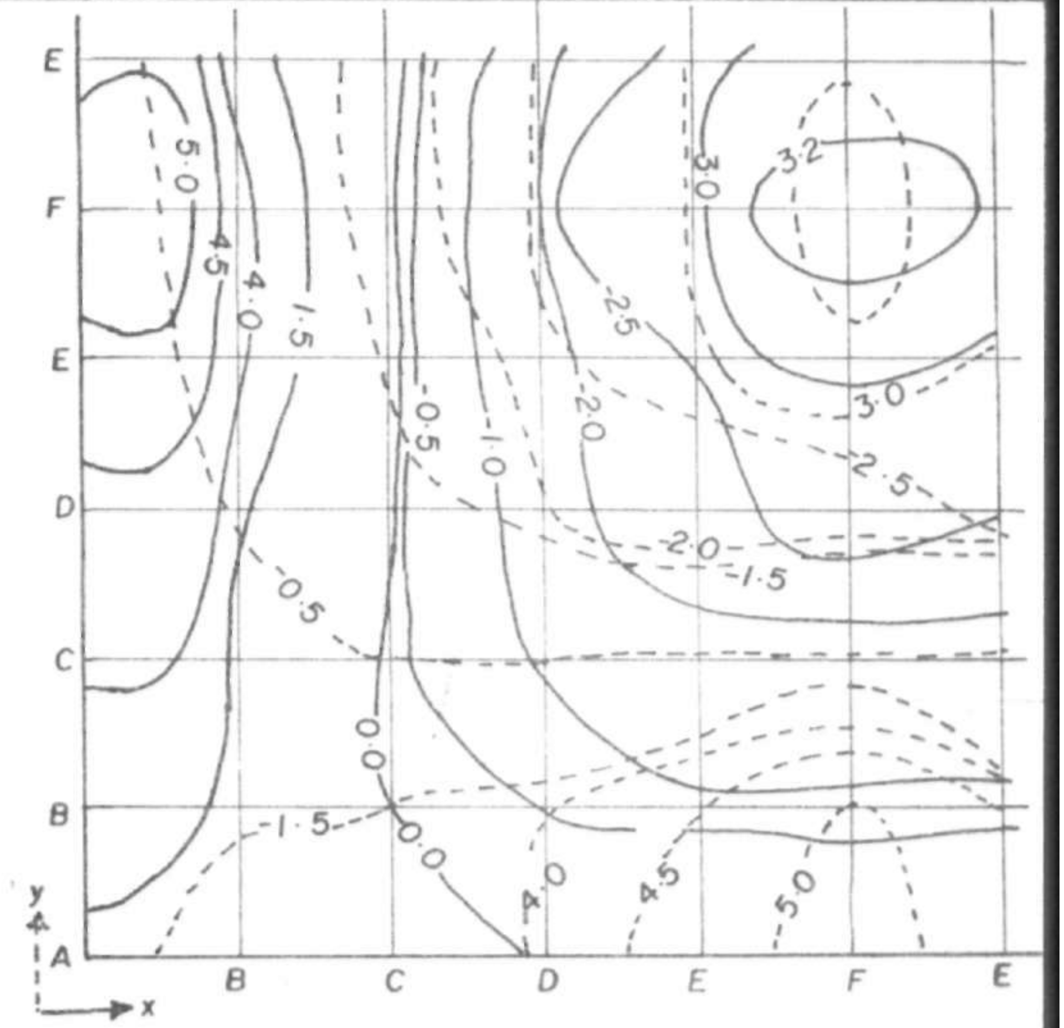
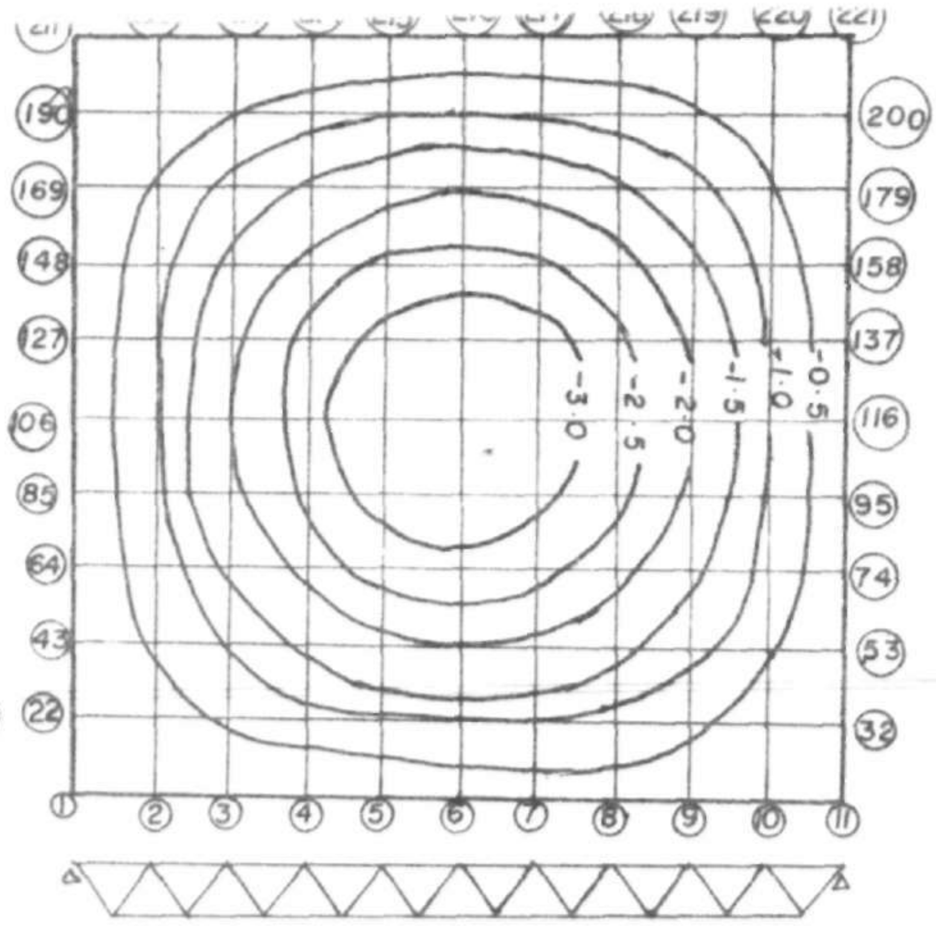


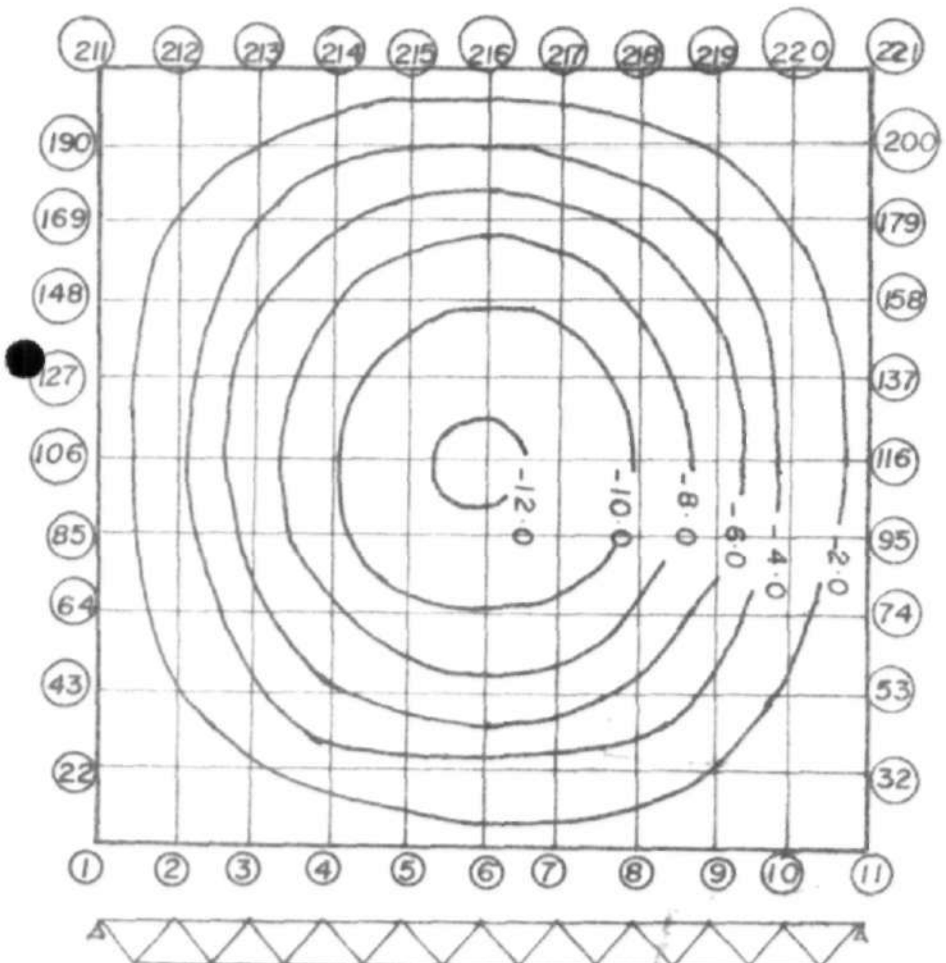
FIG. 15
FORCE DISTRIBUTION IN TOP PLANE IN X(solid line) AND Y(dotted line) DIRECTIONS



DIMENSION = 10 x 10 x 0.5
GRID SHOWN = Top layer members
SUPPORT TYPE = 1
LOADINGS = Dead+ Live Load
MEMBER AREA = 327.0 mm^2

FIG 16 TOP LAYER GRID

DEFLECTION (mm) IN Z - DIRECTION



DIMENSION = 20 x 20 x 1.0
GRID SHOWN = Top layer members
SUPPORT TYPE = 1
LOADINGS = Dead + Live Load
MEMBER AREA = 848.0 mm^2

CHAPTER 4.

DESIGN AND TESTING OF CONNECTORS

In the design of connectors, the joint most dangerously loaded was chosen in every truss already analysed. Some cases are shown schematically in table 5. The table shows for every truss

-the dimensions

-height above ground level (H) in meters

-the joint most dangerously loaded

-the axial forces of members converging at such joint

-the chosen design load for that joint.

TABLE 5

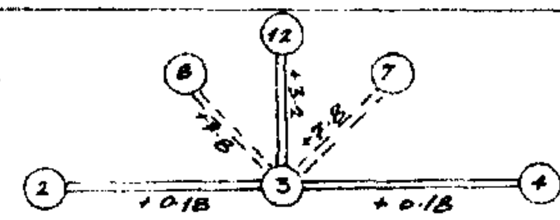
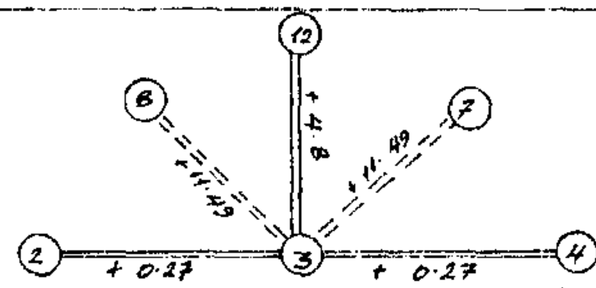
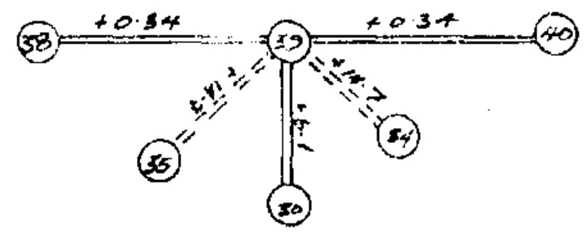
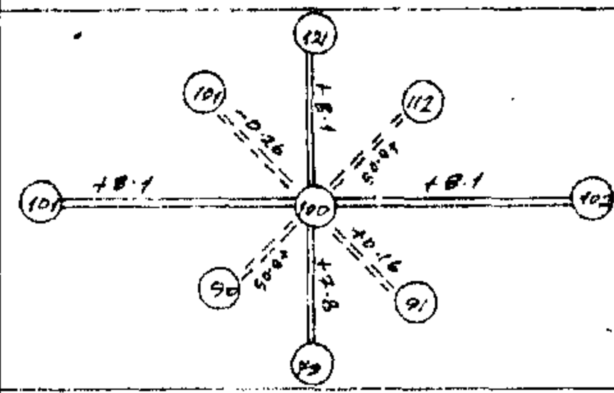
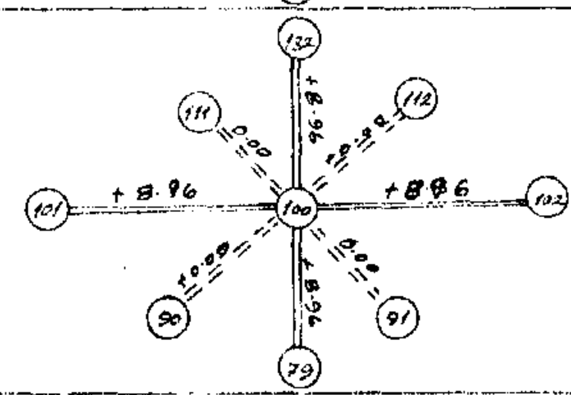
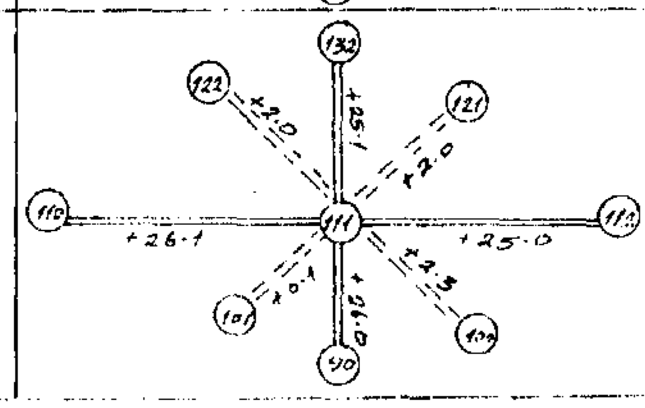
1	2	3	4	5
DIMENSIONS meters	HEIGHT meters	JOINT No & TYPE	DIAGRAM & AXIAL FORCES (KN)	DESIGN LOAD (K
10 x 10 x 1·5	5	3,19,23,39 bottom joints		7·8
10 x 10 x 1·5	10	"		11·49
10 x 10 x 1·5	15	"		14·7
10 x 10 x 0·5	5, 10	100, 101, 121 122, 1· bottom joints		8·1
10 x 10 x 0·5	15	111 top joint		8·96
20 x 20 x 2·0	5	81, 82		26·1

TABLE 5 Contd.

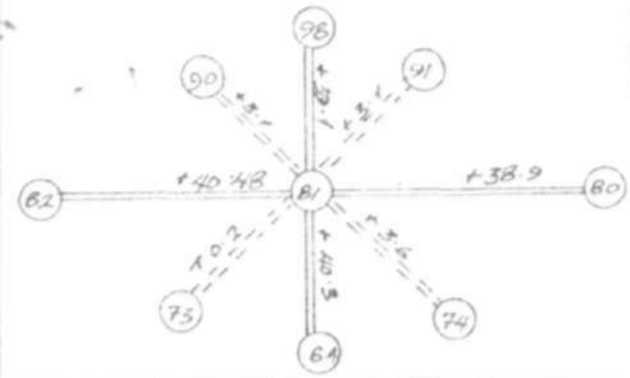
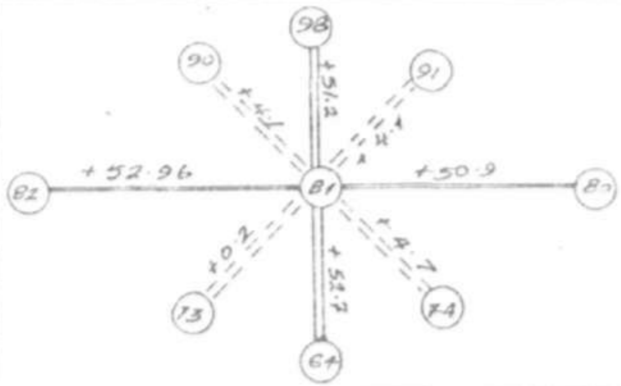
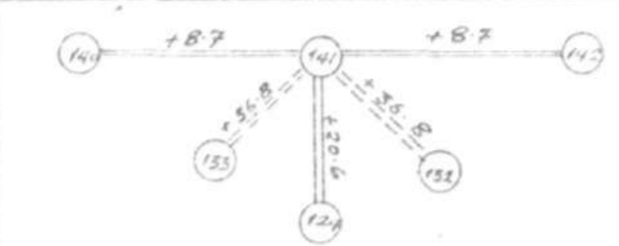
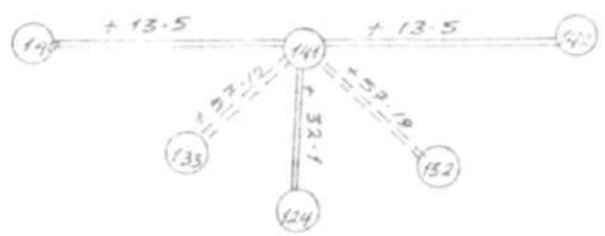
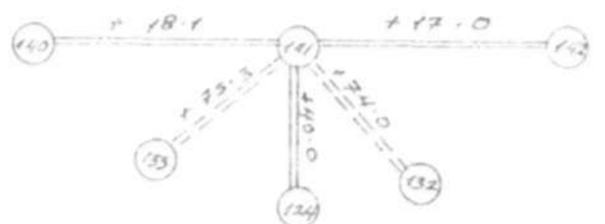
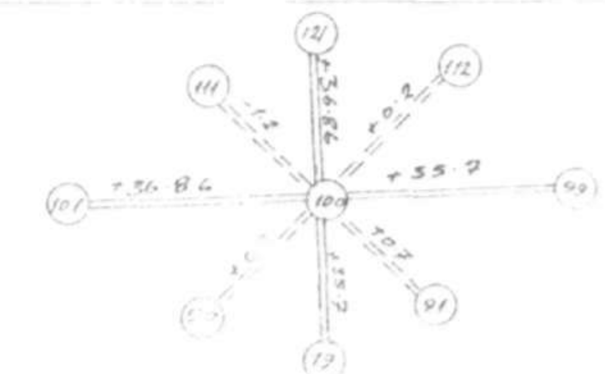
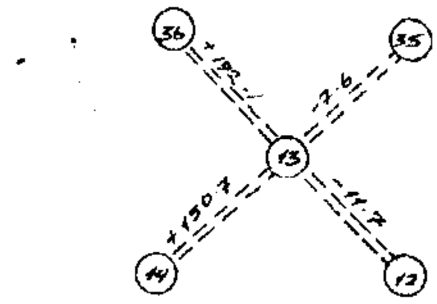
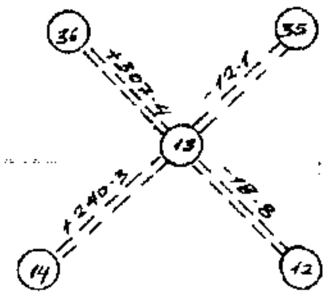
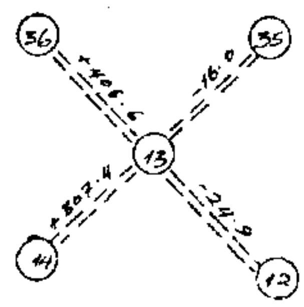
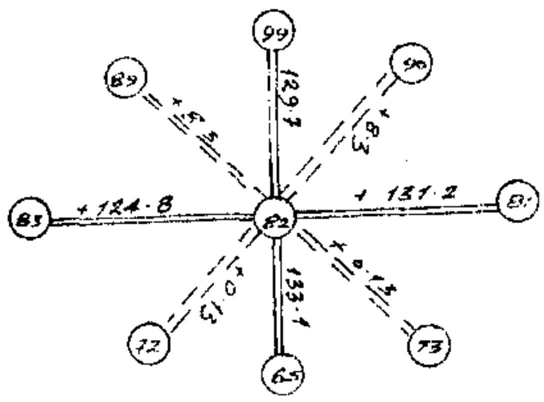
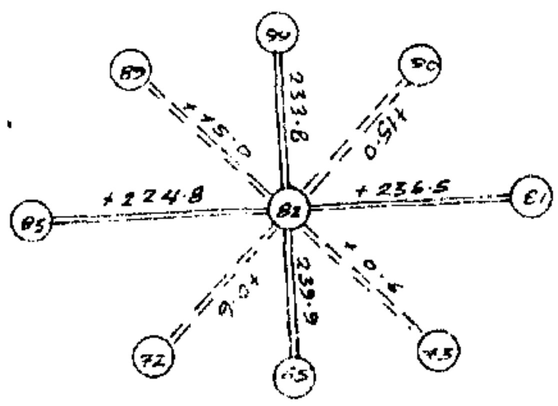
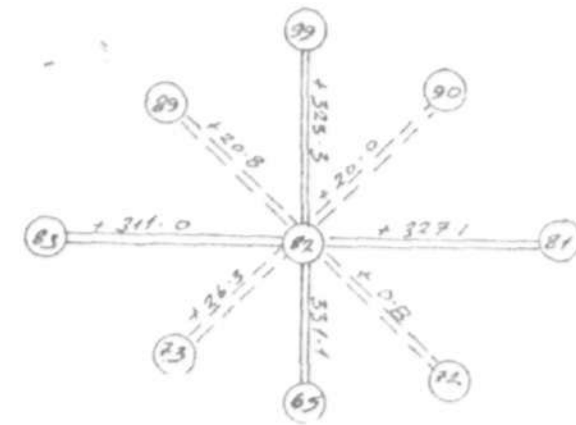
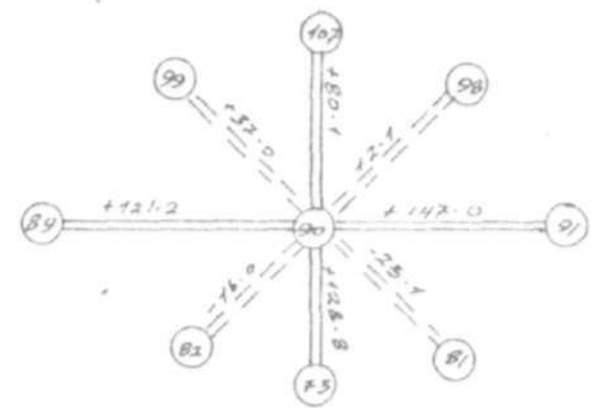
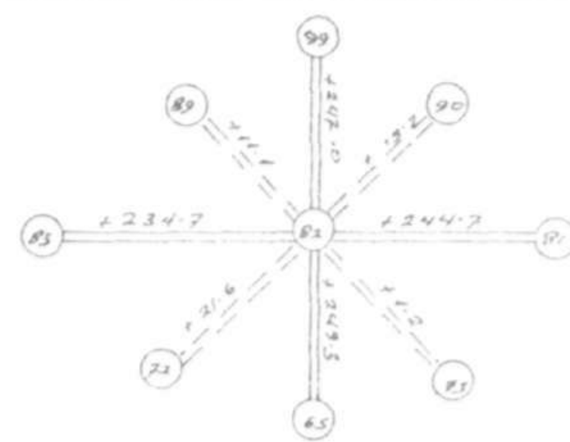
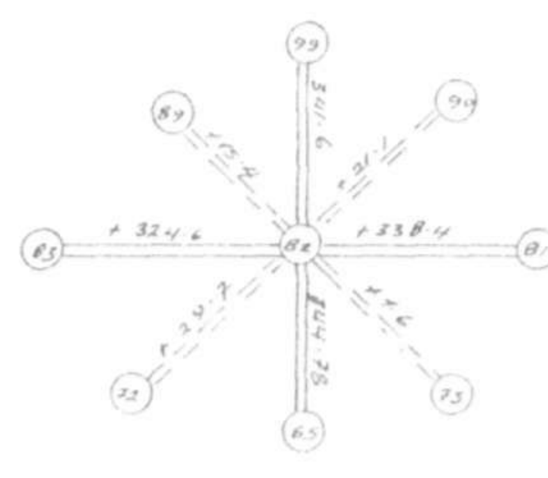
1	2	3	4	5
20 × 20 × 2.0	10	81, 82		40.48
20 × 20 × 2.0	15	81, 82		52.96
20 × 20 × 2.0	5	141		36.8
20 × 20 × 2.0	10	141		57.19
20 × 20 × 2.0	15	141		
20 × 20 × 1.0	5, 10, 15	101, 100 121, 122		36.86

TABLE 5 Contd.

1	2	3	4	5
* 40 x 20 x 1.4	5.0	13, 22, 78, 108		192.1
40 x 20 x 1.4	10	13, 22, 78, 108		307.4
40 x 20 x 1.4	15	13, 22, 78, 108		406.6
40 x 40 x 2.0	5.0	82		133.1
40 x 20 x 2.0	10.0	82		239.9

** Joint with the widest variation chosen for test

ABLE 5 Contd.

1	2	3	4	5
X 40 X 2.0	15.0	82 Top joint		331.1
X 40 X 2.0	5.0	90 Top joint.		147.01
40 X 40 X 2.0	10.0	82 Top joint		249.3
40 X 40 X 2.0	15.0	82 Top joint		344.78

4.1 DESIGN OF PROTOTYPE CONNECTORS

4.1.1 Connector - 1

The geometric design was based on that used for Pauley Pavillion (refer back to figure 4, page 17). But connection of members was designed as welded only. Dimensions of the connectors were arrived at after determining the required length of weld for the connection. See figure 18 below.

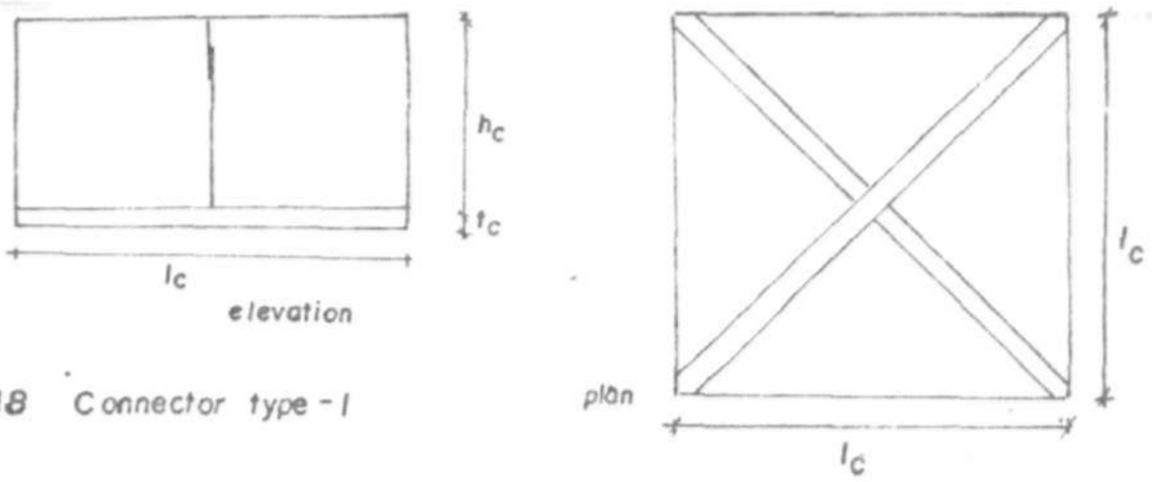


FIG 18 Connector type - I

Reffering back to the previous assumption (i.e that all the joints are pinned), it was therefore necessary to have the centroidal axes of members meeting at a point. This implies that the following must hold true for the connectors.

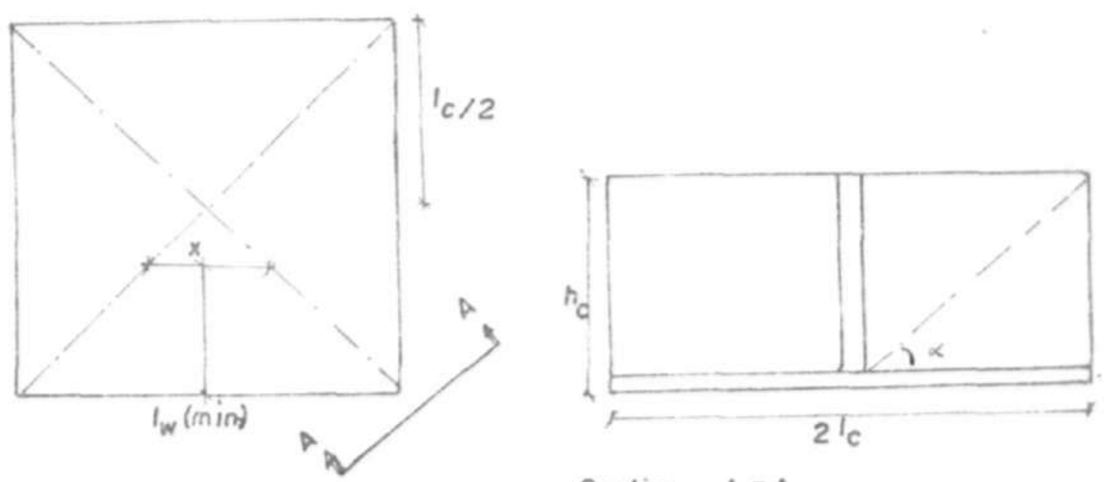


FIG 19 Dimensional analysis of connector - I

$x > Dm$

therefore

$x = Dm$

by simple proportion

$lc/(lc/2) = x/(lc/2 - lw_{min})$

therefore $lc - 2lw_{min} = x$

which implies that for a single weld

$lc = Dm + 2lw$

$hc = (lc/2)\tan\alpha$

and for a multiple weld (where possible)

$lc_{min} = Dm + 2lw/n$ where n = the number of weld runs.

$hc = lc/2 \tan\alpha$

where is a function of member goemetry.

Below is a comprehensive table (table 6) showing the following:

(P) - the maximum tensile force acting on the connector(i.e the designed force)

(α) - the angle in degrees between the struts and the horizontal

(H) - the height of the truss above ground level in meters

(Dm) - the diameter of the structural member in millimeter

(lw_{min}) -the minimum required length of weld in millimeter

(lc) - the computed length of connector in millimeter

(hc) - the computed height of the connector in millimeter

TABLE: 6 DESIGNED DIMENSIONS OF PROTOTYPE CONNECTOR-1

	DIMENSION (m x m x m)	H (mm)		DESIGNED FORCE(KN)	Dm (mm)	tm (mm)	lw (mm)	lc (mm)	hc (mm)	lc' (mm)	hc' (mm)
1	10x10x1.5	5.0	40.3	7.77	35	4	25	90	38	50	21
	10x10x1.5	10.0	40.3	11.49	35	4	36	110	47	55	23
	10x10x1.5	15.0	40.3	14.71	35	4	46	130	55	60	26
	10x10x0.5	5.0	15.8	37.29	51	6	78	210	30	95	14
	10x10x0.5	10.0	15.8	55.14	51	6	115	285	40	110	16
	10x10x0.5	15.0	15.8	70.59	51	6	147	350	50	130	19
	10x10x0.5	5.0	35.3	8.10	30	2.5	41	115	41	55	18
	10x10x0.5	10.0	35.3	3.10	30	4	26	85	30	45	16
	10x10x0.5	15.0	35.3	3.96	30	4	28	90	32	45	16
2	20x20x2.0	5.0	48.53	26.07	51	6	54	160	91	80	46
	20x20x2.0	10.0	48.53	40.48	51	6	84	220	125	95	54
	20x20x2.0	15.0	48.53	52.96	51	6	110	275	156	110	63
	20x20x2.0	5.0	48.53	36.84	54	6	77	210	119	95	54
	20x20x2.0	10.0	48.53	57.19	54	6	119	295	167	115	66
	20x20x2.0	15.0	48.53	75.34	54	6	156	370	209	135	77
	20x20x-1.0	5.0	35.23	36.36	51	6	77	210	74	95	34
	20x20x-1.0	10.0	35.23	36.36	60	6	77	215	76	100	36
	20x20x-1.0	15.0	35.23	36.36	70	6	77	225	80	110	39

TABLE 6. CONTINUED.

DIMENSION (m x m x m)	H (mm)		DESIGNED FORCE(KN)	Dm (mm)	tm (mm)	lw (mm)	lc (mm)	hc (mm)	lc' (mm)	hc' (mm)
3. 40x20x-1.4	5.0	26.3	192.10	108	8	300	710	176	260	65
40x20x-1.4	10.0	26.3	307.40	108	8	480	1070	265	350	87
40x40x-1.4	15.0	26.3	406.60	108	8	635	1380	342	430	107
40x40x2.0	5.0	29.5	133.10	146	8	207	565	160	250	71
40x40x2.0	10.0	29.5	239.89	146	8	373	900	255	335	95
40x40x2.0	15.0	29.5	331.77	146	8	516	1130	334	410	116
40x40x2.0	5.0	29.5	147.01	146	8	229	610	173	265	75
40x40x2.0	10.0	29.5	249.30	146	8	383	925	252	345	98
40x40x2.0	15.0	29.5	344.78	146	8	536	1220	345	420	119
4. 50x45x-2.0	5.0	26.2	450.62	146	8	700	1550	382	500	124
50x45x-2.0	10.0	26.2	450.62	171	8	700	1575	388	525	130
50x45x-2.0	15.0	26.2	450.62	191	8	700	1595	393	545	135
50x45x-1.25	5.0	13.4	698.23	191	8	1085	2365	395	735	123
50x45x-1.25	10.0	13.4	698.23	216	8	1085	2390	398	760	127
50x45x-1.25	15.0	13.4	698.23	257	8	1085	2440	407	810	135
5. 30x30x2.5	*[ALL]	13.4	1313.33	419	16	1020	2465	411	935	156
30x30x-2.5	[ALL]	13.4	2376.60	313	18	1645	3610	602	1145	191
30x30x-1.6	[ALL]	12.5	3649.00	419	20	2270	4965	549	1560	173
6. 100x95x3.0	*[ALL]	21.3	1524.56	419	26	730	1985	369	790	155
100x95x-3.0	[ALL]	21.3	3588.94	419	20	2235	4895	955	1540	301
100x96x-2.0	[ALL]	15.2	3290.60	419	18	2270	4965	676	1560	213

* [ALL] signifies that the height is for all cases(5.0, 10.0, 15.0)m

4.1.2 Connector - 2

The geometric design was based on a simple octahedral ring (made to recieve 8 members at 8 different directions). Connection of members to the connector was design for both welded and bolted situations. Below is the skeletal diagram.

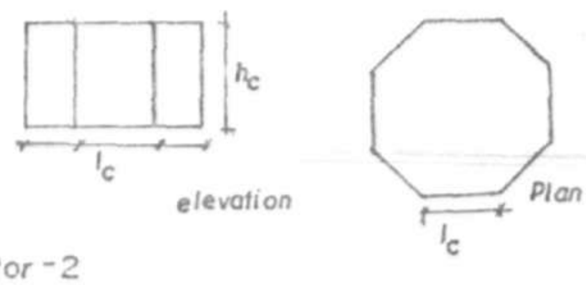


FIG.20 Sketch of connector - 2

Welding: the required length of weld in each case was as already calculated in table 5. A check was though made in every case to ascertain that the circumference of the member section(which is the length of weld in this case) exceeds the minimum lengtj of weld.

i.e Cm > lw .ere Cm = the circumferencial length of the member(Dm).

Bolting: The amount of bolts required in each cases was calculated as stated in chapter one. The bolt was assumed to be secured to the end of the structural member via a plate welded to the member.

Dimensioning: The geometrical dimensions of the connector was derived by ensuring the following;

- that the length of the connector (lc) was adequate. i.e lc>Dm
- that the hieght of the connector (hc) was enough so as to ensure that the centroidal axes of the members meet at the a point within the connector. (see figure 21).
- that the concentric diameter of the connector (Dc) is

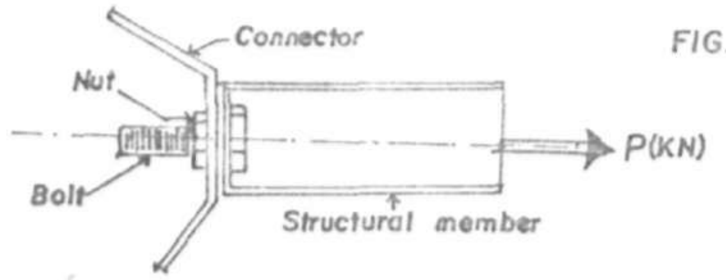


FIG.21 Member connection in connector - 2

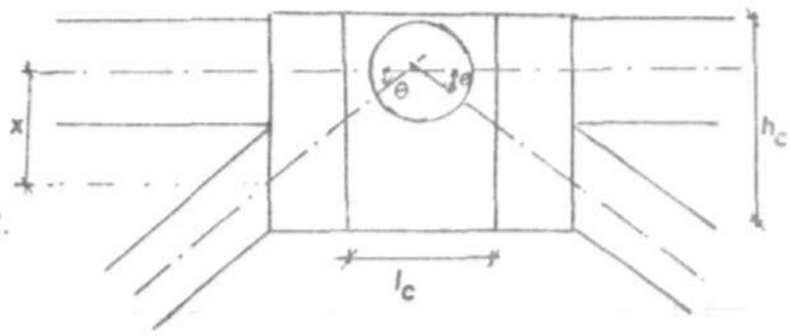


FIG.22 Dimensional analysis of connector - 2.

i.e

$$(lc/2) = r - [r.\cos(\alpha/2)]$$
$$= r - [r.\cos(22.5)]$$
$$= r - 0.854r$$

therefore

$$lc = 0.7654r$$
$$\text{OR } r = 1.31(lc)$$

which implies that

$$Dc = 2.62(lc).$$

note: that for an octagon, angle (α) is equal to 45

The computed dimensions for connector-2 is tabulated in table 7.

In all cases the thickness of the connector should be greater or equal to that of the member it is connecting. But reference should always be made to gusset thicknesses recommended by Mukhanov[21] as shown in the appendix.

TABLE: 7 DESIGNED DIMENSIONS OF PROTOTYPE CONNECTOR-2

	DIMENSION (m x m x m)	H (m)		DESIGNED FORCE(KN)	Dm (mm)	tm (mm)	Cm (mm)	lc (mm)	Dc (mm)	hc (mm)	ASSUMED BOLT SIZE*	ASSUMED BOLT No.	SAFE LOAD(KN)
1.	10x10x1.5	5.0	40.3	7.77	35	4	109.9	40	104.8	35	15	1	20.4
	10x10x1.5	10.0	40.3	11.49	35	4	110.0	40	104.8	35	16	1	20.4
	10x10x1.5	15.0	40.3	14.71	35	4	110.0	40	104.8	35	16	1	20.4
	10x10x-.5	5.0	15.8	37.29	51	6	160.2	55	144.1	75	22	1	39.4
	10x10x-.5	10.0	15.8	56.14	51	6	160.2	55	144.1	75	27	1	59.7
	10x10x-.5	15.0	15.8	70.59	51	6	160.2	55	144.1	75	30	1	72.9
	10x10x-.5	5.0	35.3	8.10	30	2.5	94.3	35	91.7	00	16	1	20.4
	10x10x-.5	10.0	35.3	3.10	30	4	94.3	35	91.7	00	16	1	20.4
	10x10x-.5	15.0	35.3	8.96	30	4	94.3	35	91.7	00	16	1	20.4
2.	20x20x2.0	5.0	48.5	26.07	51	6	160.2	55	144.1	140	20	1	31.9
	20x20x2.0	10.0	48.5	40.48	51	6	160.2	55	144.1	140	24	1	45.9
	20x20x2.0	15.0	48.5	52.96	51	6	160.2	55	144.1	140	27	1	59.7
	20x20x2.0	5.0	48.5	36.84	54	6	169.7	60	157.2	150	22	1	39.4
	20x20x2.0	10.0	48.5	57.19	54	6	169.7	60	157.2	150	27	1	59.7
	20x20x2.0	15.0	48.5	75.34	54	6	169.7	60	157.2	150	33	1	90.2
	20x20x-1.0	5.0	35.3	36.36	51	6	160.2	55	144.1	105	22	1	39.4
	20x20x-1.0	10.0	35.3	36.36	60	6	133.5	65	170.3	125	22	1	39.4
	20x20x-1.0	15.0	35.3	36.36	70	6	219.91	75	196.5	145	22	1	39.4

TABLE 7. CONTINUED

DIMENSION (m x m x m)	H (m)		DESIGNED FORCE(kN)	Dm (mm)	bm (mm)	Dn (mm)	lc (mm)	Dc (mm)	hc (mm)	ASSUMED BOLT SIZE*	ASSUMED BOLT No.	SAFE LOAD(kN)
3. 40x20x1.4	5.0	26.3	192.10	108	8	339.29	115	301.3	190	27	4	238.8
40x20x1.4	10.0	26.3	307.40	108	8	339.29	115	301.3	190	33	4	360.8
40x40x1.4	15.0	26.3	406.60	108	8	339.29	115	301.3	190	35	4	424.3
40x40x2.0	5.0	26.3	133.10	146	8	458.59	155	406.1	270	22	4	157.6
40x40x2.0	10.0	26.3	239.99	146	8	458.67	155	406.1	270	30	4	291.6
40x40x2.0	15.0	26.3	331.77	146	8	458.67	155	406.1	270	33	4	360.8
40x40x2.0	5.0	29.5	147.01	146	8	458.67	155	406.1	270	22	4	157.6
40x40x2.0	10.0	29.5	249.30	146	8	458.67	155	406.1	270	30	4	291.6
40x40x2.0	15.0	29.5	344.78	146	8	458.67	155	406.1	270	33	4	360.3

4.2 TESTING OF CONNECTORS:

4.2.1 Purpose & Extent;

.he testing of the connectors were to be carried out so as to clarify the following points:

(a) - the adequacy of the design criterea used.

(b) - the load capacity and the factor of safety thereoff.

(c) - the stress distribution/displacements under full load.

4.2.2 Testing Stand;

The type of loading pattern applicable to the connectors on site necessitated a special testing stand that would satisfy the following:

(a) -the possibility of mounting the connector and loading it axially in 3 different directions.

(b) - the possibility of mounting a system of measuring the load applied.

With the above considerations a testing stand of the shape shown in figure 23 was arrived at. The dimensions of which depended on the size of model to be tested.

4.2.3 Model Material;

A modelling material is normally chosen after a due consideration is given to its properties as it suits or compares with the prototype. The possible modelling materials could be classified as: metals, plastics, gypsum plaster, paper boards etc. But the properties whichs serve as a guiding criterea for a particular choice includes ;

- modulus of elasticity
- poissons ratio

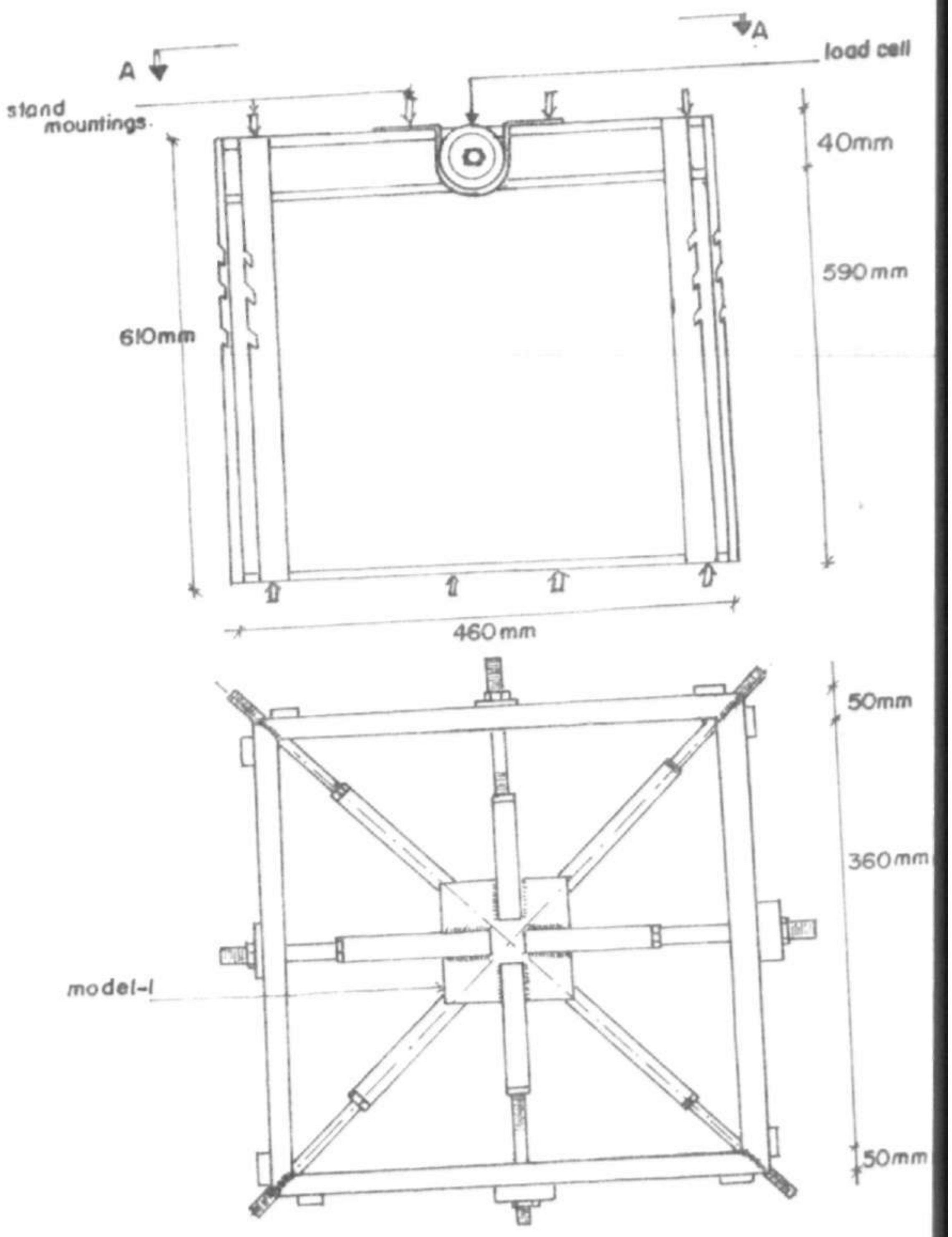


FIG. 23 SCHEMATIC DIAGRAM OF THE TEST STAND SHOWING, A CONNECTO & ITS LOADING SYSTEM.

- workability of the material
- thermal condutivity and
- creep.

After giving due consideration to the above, mild steel was chosen for use as the model material. Some of the reasons for its choice are:

(a) the properties compares with that of the prototype.

(b) the prototype is devoid of complicated curvatures, therefore machining of the metal is possible.

(c) its weldeability

Although plastics(e.g perpex) is easily workable and would require smaller loading appliances to produce measurable strains, its vast difference in properties with regards to Hooke's law compared with the prototypes led to its not been found very suitable.

4.2.4 Model Dimensions & Scale:

Scales of models are normally chosen so as to compromise between the properties of the material to be measured and the ease of fabrication of the size.

The available loading system in the laboratory used couple with its measuring instruments made it very difficult fo select a good scale. Therefore a ratio of model to prototype of 1 : 10 was chosen. This resulted in a very small model dimension as evident in table 9 shown below.

TABLE 3

PROPERTY	PROTOTYPE DIMENSIONS(mm)		MODEL DIMENSIONS(mm)	
	connector-1	connector-2	model-1	model-2
1. Material thickness	8	8	0.8	0.8
2. Connector length	710	115	71.0	11.5
3. Connector height	176	190	17.6	19.0
4. Member diameter	108	108	10.8	10.8
5. Member thickness	8	8	0.8	0.8
6. Weld thickness	8	8	0.8	0.8

4.2.5 Model Fabrication;

The two models to be tested were manufactured in the departmental workshop. Pricipally each model consist of 2 distinct portions. One being the connector and the other being the structural member that transmits the axial force to the connector.

A metal sheet of 0.8mm thickness and a circular hollow section(C.H.S) of external diameter 10.5mm with thikness of 0.8mm, were utilized in the fabrication of the model. The fabrication process for each of the model is as follows:

For Model - 1;

- the cutting out to precision, the 3 pieces that make up the connector. These are the base plate (71.0mm x 71.0mm in dimension) and two other pieces each measuring 17.6mm x 100.0mm.
- the slotting of the 2 equal pieces above at their mid-section so as to fit together.
- the scribing out of straight lines on the 3 pieces where the

connecting members are to be later on welded.

- the welding of the 3 pieces together.a

- the cutting out of adequate lengths of the C.H.S(4 long ones and 4 short ones). A length is considered adequate, if it is long enough to effectively transmit the applied forces and small enough to fit into the designed test stand.

- the welding of these pieces onto the connector, making sure it centers on the initially scribed lines.

For model-2:

- the cutting out to precision, 8 equal pieces(11.5mm x 19.0mm) that make up the connector

- the setting and welding of 2 pieces a time so that the internal angle between them is 135 .

- the subsequent welding of all other edges together to form an octehedral ring with a diameter of 30.1mm.

- the cutting out of adequate length of C.H.S(4 long ones and 4 short ones).

- the shaping (bevelling) of an end face of each of the 4 long pipes to attain the required angle of 26.3 with the horizontal.

- the welding of these pipes to the connector. The short ones welded close to the top and on the alternate faces of the connector. While the longer ones are welded to the lower part of the connector on the remaining alternate faces, ensuring that the bevelled faces flushes on the connector to make the required angle of 26.3 with the horizontal.

4.2.6 Test Set Up And Instrumentation:

In setting up the test stand, the following was to be fullfilled:

(a) -effective load application

(b) -effective measurement of the required strains/displacement.

(c) -effective measurement of the applied load.

(a) Load Application:

In appliying load, the existing load application devices in the laboratory(these are mainly heavy hydraulic jacks) were not found suitable. A system was then devised, whereby the connection members are fitted to a long stud through a nut and screw system. The nut is then screwed at the end to effect load application. Refer back to figure 23.

(b) Strain/Displacement measurement:

To measure accurately strains and displacements resulting from applied loads or stress, electronic instrumentation devices are best employed.

An electronic instrumentation system consist of a number of components which together measures and record the results. Instrumentation devices normally used in above measurements are tranducers. A transducer is a device which when actuated by energy in one transmission system, supplies energy in the same form or in another form to a second transmission system. There are various types, its choice depending on the physical quantity to be measured, principle of measurement and the required accuracy.

The phsical quantities that were required are strains of the plate in connector-1 and the displacements of the octahedral ring from its neutral axis in cnnector-2. To measure the strains, strain guages(which are passive transducers) were used. There are different types of strain guages as regards size(length), shape and material. With regards length, we have sizes ranging from PL5(5mm length) to PL20(20mm length), and depending on material, we have wire grid or foil. The number of directions strains are required decides whether a longitudinal guage or a rosette type guage is to be used.

Basically a strain guage converts mechanical displacement into a change of resistance. The measurrement of which is by a specially adapted Wheatstone Bridge. Sensitivity of a strain guage is described in terms of the guage factor(K). K is the unit change in resistance per change in length.

i.e $K = (\delta R/R)/(L/L) = 1 + 2\mu$

where

R = Nominal guage resistance

δR = The change in guage resistance

L = Guage length

L = Change in length

μ = Poissons ratio

But for most metals;

$0.25 < \mu < 0.35$

$1.5 < K < 1.7$ in most cases.

Below is a circuit diagram of a typical guarded Wheatstone Bridge. (figure 24)

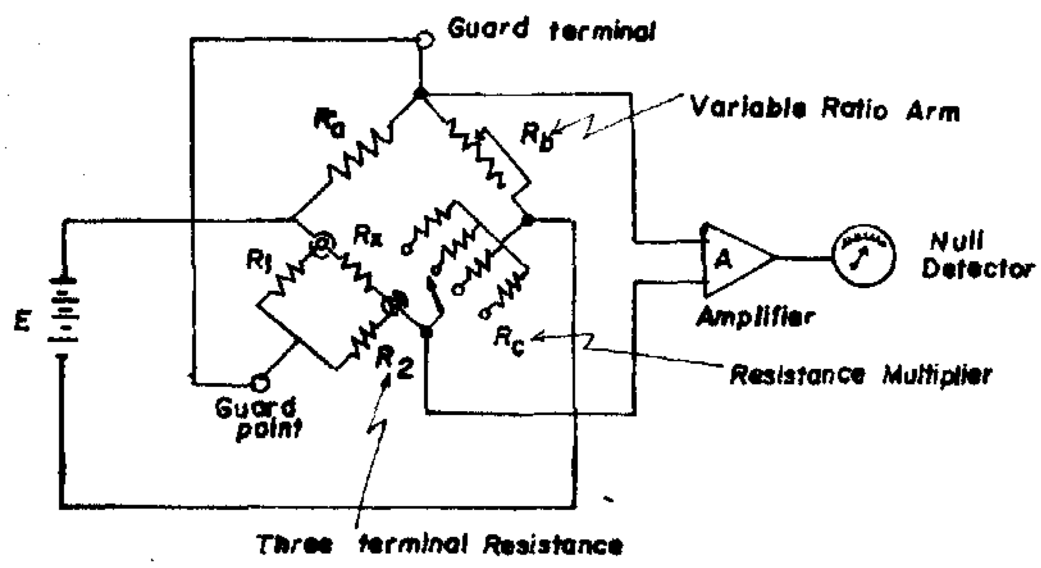


FIG 24. A Typical guarded Wheatstones Bridge

(c) Measurement Of The Applied Load:

The system of load application used required that, small sized measuring devices be employed. This was to avoid the possibility of an extra load being constituted by the measuring device.

The devices available and suitable were load cells. But due to the unavailability of enough quantity (6 in number) in their required sizes, the maximum load that could be applied was limited to only 20KN.

The above reason was what led to the choice of very small models as a solution for effective load measurements.

The load cells were mounted on the test frame so as to eliminate the possibility of its constituting any additional weight. To have an accurate measurement, the cells were initially caliberated with the aid of the Universal Testing Machine. The results were plotted graphically(appendix 2) and later used to obtain the magnitude of the applied loadings.

(d) Procedure Of Measurements:

For each of the connector, the points where stresses and displacement were expected to be critical were selected for monitoring.

For model-1, two strain guages of PL5 sizes were affixed to opposite faces of such points. This was to help in separating the axial strains from the bending strains. The procedure of affixing each strain guage consist of:

-cleaning the surface where the guage is to be affixed, to ensure effective bonding and transmission.

-mixing of the bonding cement(P-2 Drug A) with the hardener(P-2 Drug B).

-applying the mixed cement thinly and evenly on the cleaned surface, the underface of the strain guage is then affixed to it. The transparent paper supplied with the strain guage is then placed and pressed slightly on the surface of the strain guage. This is to prevent any air trap in the glue. When the glue hardens, the transparent paper is removed.

-soldering the two terminals of the strain guage to a long electrical chord that would act as lead to the switch box.

-preparing another mix which is spread thickly and evenly on the soldered terminals. This is to prevent any possible short-circuitary and damage to the guage.

-labelling of each guage at the flexible chord for identification.

The above procedure was completed for each of the selected point and allowed to dry for 2 days after which the model was mounted and tested.

For model-2, the model was too small to employ the use of strain guages in monitoring the strains. Therefore, displacements rather than strains were monitored. Displacement transducers were clamped with the aid of a device unto the testing stand at the appropriate places.

In testing the models, the models were initially mounted freely and their zero load reading taken. Subsequently axial loads were applied 2KJ at a time and allowed to attain a steady state for at least 10 minutes before the readings were taken.

4.3 DATA ANALYSIS AND RESULTS:

4.3.1 Data Analysis;

-For strains:- the axial strainswas separated from the bending strains thus:

Let,

Mi = bending strains at point (i) due to construction and loading inaccuracies.

Fi = axial strains at point (i) due to the applied load.

Ai = strain measurement at one side of point (i).

Bi = strain measurement at the other side of point (i).

Then reffering to figure 24 below,

$$Ai = Fi + Mi$$

$$Bi = Fi - Mi$$

$$Ai + Bi = 2Fi$$

$$:- \quad Fi = (Ai + Bi)/2$$

similarly

$$Ai - Bi = 2Mi$$

$$:- \quad Mi = (Ai - Bi)/2$$

4.3.2Results:

The results of the test carried out is as tabulated in table 8 below

RESULT OF TEST FOR MODEL - 1.

AXIAL STRAINS

POINTS / LOAD	1	2	3	4	5	6	7	8
0	14.6	14.4	15.7	15.4	15.1	15.0	15.0	15.4
2	14.6	14.3	15.7	15.3	15.1	15.0	15.9	15.5
4	14.6	14.2	15.7	15.4	15.1	15.0	16.0	15.5
6	14.6	14.4	15.7	15.5	15.1	15.1	16.0	15.8
8	14.6	14.6	15.7	15.7	15.1	15.0	16.0	16.0
10	14.6	14.6	15.7	15.7	15.1	15.1	16.0	16.1

BENDING STRAINS

POINTS / LOAD	1	2	3	4	5	6	7	8
0	-0.24	0.45	-1.03	0.46	0.63	0.09	-1.52	-
2	-0.45	0.20	-1.20	0.25	0.82	0.32	-1.30	-
4	-0.52	0.12	-1.18	0.23	0.70	0.57	-1.36	-
6	-0.50	-0.09	-1.07	0.56	0.57	0.82	-1.65	-
8	-0.77	0.18	-0.34	0.53	0.67	0.90	-1.67	-
10	0.92	0.15	-0.31	0.50	0.90	0.93	-1.72	-

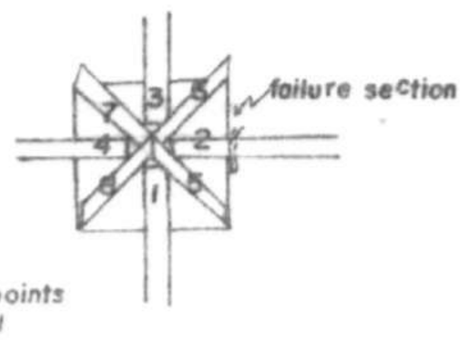


FIG. 25
Measuring points for model-I

TABLE 9: RESULT OF TEST FOR MODEL - 2

LOAD APPLIED (KN)	DISPLACEMENTS (mm) 1	2	3	4
0	0	0	0	0
2	0.1	0	0	0
4	1.3	0.9	0.4	0.3
6	1.7	1.2	0.7	0.6

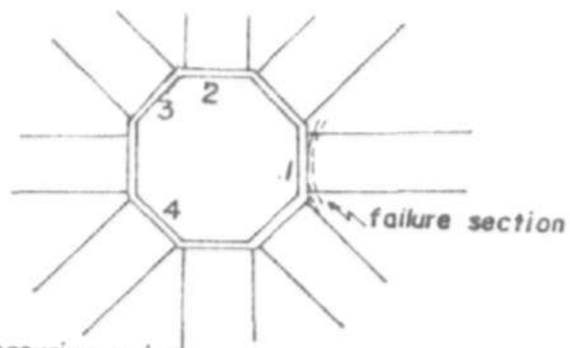


FIG. 26
Measuring points for model - 2

CHAPTER 5.

OBSERVATIONS, COMMENTS & RECOMMENDATION

From the results obtained in the previous chapters , observations and comments are made below as regards the following:

-method of analysis

-loadings and design of members

-dimensional variation

-design and testing of connectors.

5.1 OBSERVATIONS:

5.1.1 Method Of Analysis:

It was observed that the ease of analysis is a function of the number of degrees of freedom of the structure. The lower the number of joints, the less time taken by the computer for execution. E.g for a 41 jointal structure with about 90 degrees of freedom, the average time was found to be about 5 minutes. Whereas for a 221 jointal structure with about 500 degrees of freedom, the average computer execution time was found to be about 30 seconds.

It was also impossible to analyse a structure with a degree of free dom of over 32000, as the computer can not make available a memory size of more than 35 to 37000 spaces at any time.*

*The computer used was the 'Control Data Cyber 72(64k words) Operating System: NOS' available at Ahmadu Bello University.

5.1.2 Loadings And Design Of Members;

Reffering back to chapters 2,3 and tables 1, 2, 3, 4, the following observations were made about the loadings:

(a) That the assumption of emperical formula [(8 to 12)L N/mm] in calculating the self weight was rational as it compares to the actual self weight after design. The variations thereoff are in a pattern.

(b) That for trusses of lower spans(10, 20, 30)m, assumption value of 8L (N/mm) gave rise to a nice variations in design results for a variation of truss height above ground level. Whereas the assumption of higher values led to a design result unvarying with variation of truss height.

(c) That for trusses of medium span(30, 40, 50), the assumption value of (8 to 10)L (N/mm) gave rise to design result varying with height variation. Whereas an assumption of a higher value did not.

(d) That for trusses of large span(over 60, 70)m, assumption value of (10 to 12)L (N/mm) gave no noticeable variation for variations of truss height.

(e) For lower span and assumption of 8L (N/mm), both load combinations 1 (dead load + live load) and load combinations 2(dead load + wind load) were determining factors in the design. Assumption of higher value leads to a case where load combination 2 makes no contribution to the design.

(f) For medium spans, assumption of (8 to 10)L N/mm allows the the two load combinations to make contributions to the design, while a higher value allows for only load combination 1.

(g) For large spans, assumption of (8 to 12)L N/mm allows for only load combination 1 to be the designing factor.

5.1.3 Dimensional Variation:

With reference to chapters 1, 2, 3, and tables 2, 3. The results obtained by varying the depth to span ratio(d/L), and support types of each of the truss shows the following:

(a) That for lower spans, a (d/L) ratio of about (1/10) tend to result in a lighter structure, while a ratio of about (1/20) or lower produces a heavier structure.

(b) That for medium spans, a (d/L) ratio of about (1/20) or lower produces a heavier structure, while a ratio of about (1/40) results in a lighter structure.

(c) That for large spans, a (d/L) ratio of about (1/40) and lower; (1/60) and higher gives a heavier structure while between these range, a lighter structure is achieved. On the whole, the curve tend to take the shape as in figure 27 for a lighter structure.

(d) That generally support type 1 gave a better force distribution and resulted in a lighter structure than other support types tried.(see figure 23)

5.1.4 Design And Testing Of Connectors:

Based on the obtained results in chapter 4, the following observations were made:

(a) That connector type-2 tend to increase the roof size more than connector type-1.

(b) That for a small model, it is difficult to attain adequate precision.

(c) That welding of plates with thicknesses below 2mm is very difficult and leads to incomplete fusion -for oxy-acetylene welding, while its not possible to arc weld. This is because the minimum welding rods available is 1.5mm.

(d) In testing small models, bending strains developed are significant, -up to 12% of the axial strain.

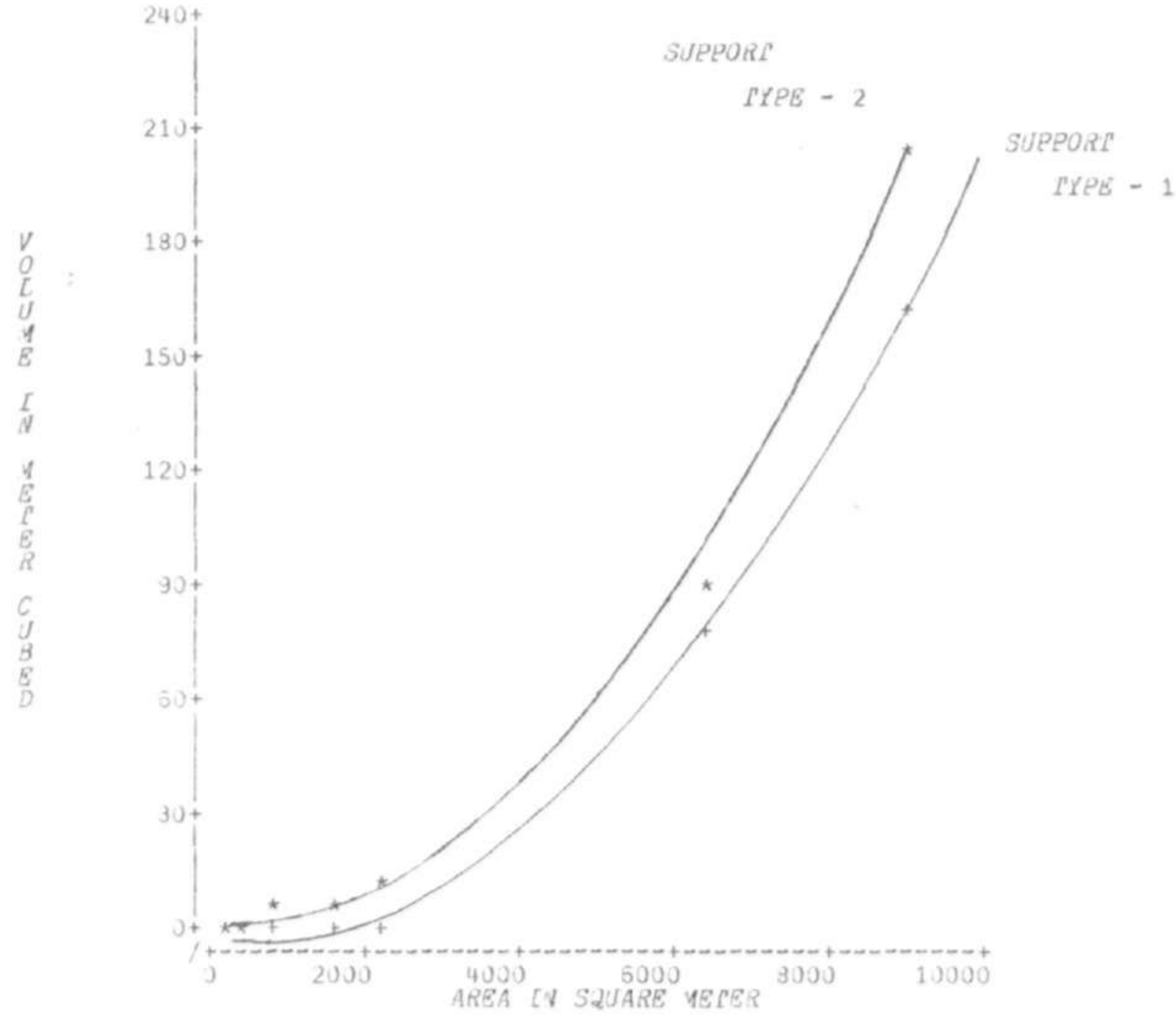


FIGURE 28. GRAPH OF VOLUME VERSUS AREA COVERED FOR SUPPORT TYPE 1 AND 2.

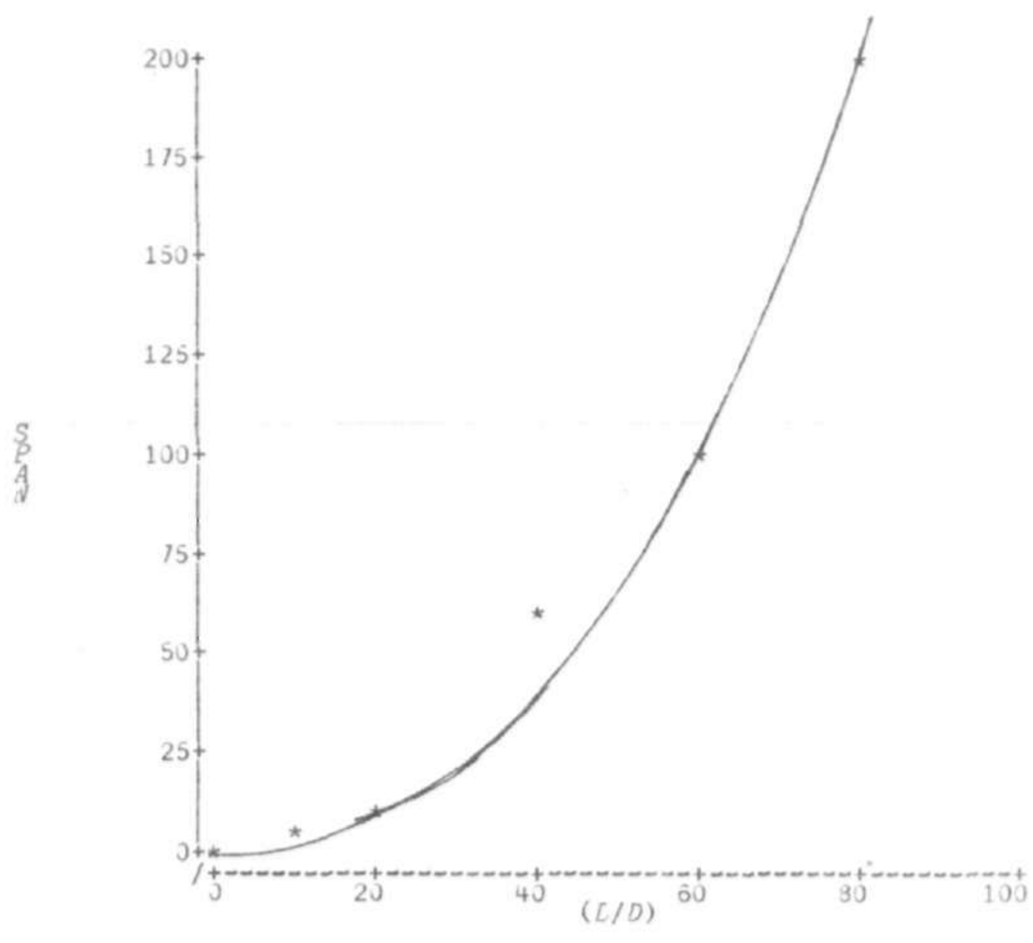


FIG. 21 ECONOMIC VARIATION OF SPAN/DEPTH RATIO AS THE SPAN OF THE TRUSS INCREASES

5.2 CONCLUSIONS AND RECOMMENDATION:

Based on the investigation of analysis, design and test carried out, the following deductions are made as to the suitability of using space structure for the purpose of roof covering in Nigeria.

5.2.1 CONCLUSION;

(a) The initial assumption of (3 to 12)L N/mm for the self weight of the truss is reasonable, provided it is applied with care as regards the different spans.

(b) A computer (preferably large memory size) is essential to successfully analyse the structure.

(c) The use of Direct Stiffeness Method of analysis gives satisfactory results.

(d) The use of support type-1 gave a better force distribution than the rest.(figure 23)

(e) The elastic method of design to BS 449 has earlier been proved satisfactory [23]

(f) Structural members loaded in compression are usually in more critical conditions. While the tensile loaded members put the joints in more critical conditions.

(g) The use of small models for testing purposes does not give appreciable result.

(h) However the type of loading system used is possible for higher load application, where a torquing machine is available.

(i) The complete manufacture of skeletal space roof system is possible without rersorting to importation of proprietary systems.

5.2.2 RECOMMENDATION:

(a) It is recommended that as much as possible, a finner grid should be employed.

(b) While support types 2 and 3 are adequate, support type 1 should be employed.

(c) In assuming self weight of the truss before analysis, 8LN/mm should be assumed for small spanned (< 30m) roofs, 10L N/mm should be employed for medium spanned(> 30m, < 60m) roof, while for spans larger than 60m, (10 t0 12)L N/mm should be employed. L is taken as the average of the 2 spanning directions.

(d) Circular hollow sections should be used since compression members are in more critical situations.

(e) For connector fabrication, the recommended gusset plate size by Mukhanov[26]* is suitable, and thicknesses below 6mm should be avoided.

5.3 SUGGESTIONS FOR FUTURE RESEARCH:

(a) A future research is necessary as regards the economic use of space trusses for roof covering in Nigeria. This could be done by embarking on design and cost analysis for both plane and space trusses covering equal areas

(b) Where a comprehensive steel laboratory equipment is available, it would be a good research to carry out a full scale test, to properly define the jointal behaviour of space structures.

* See Appendix 1

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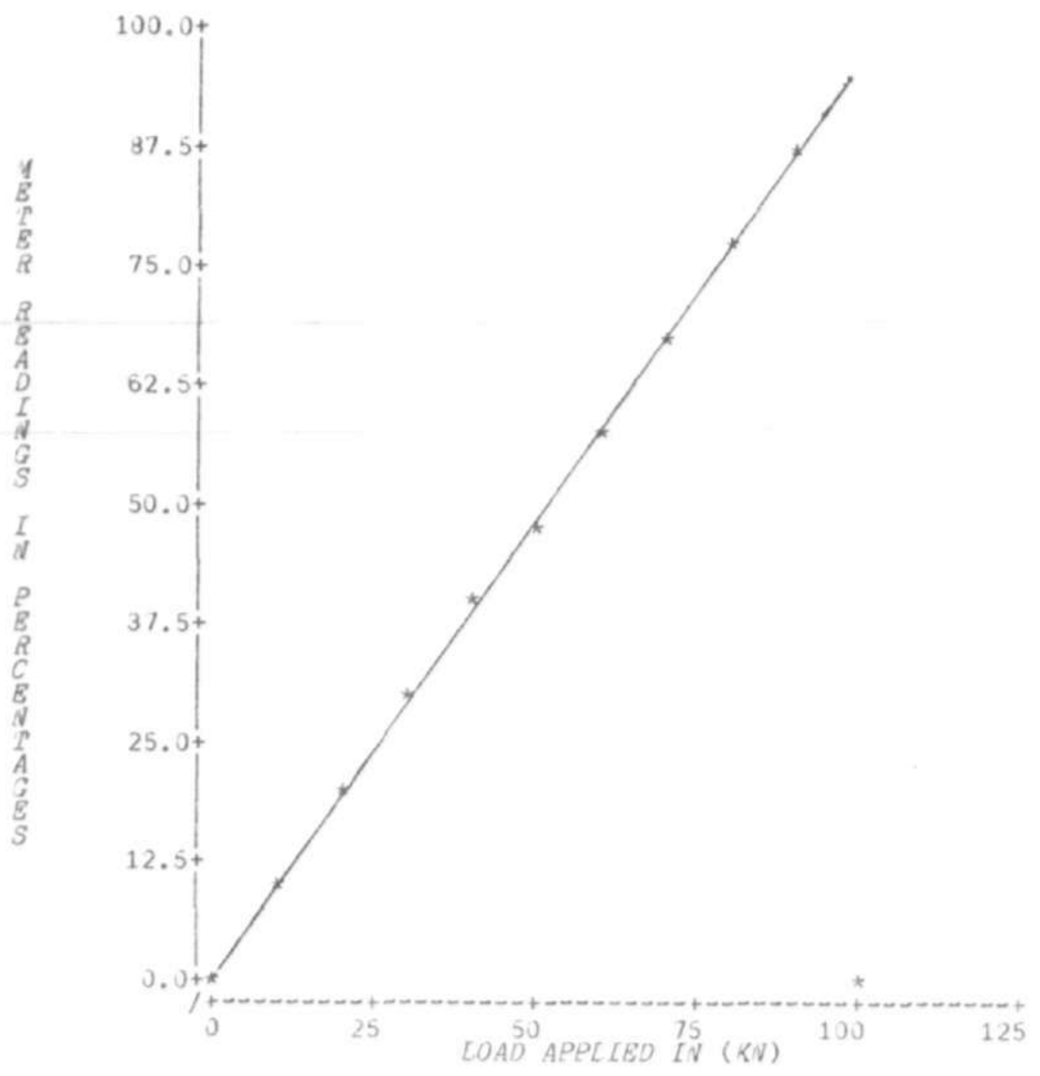
APPENDIX 1:

Thicknesses of gusset plate as recommended by K. MUKHANOV

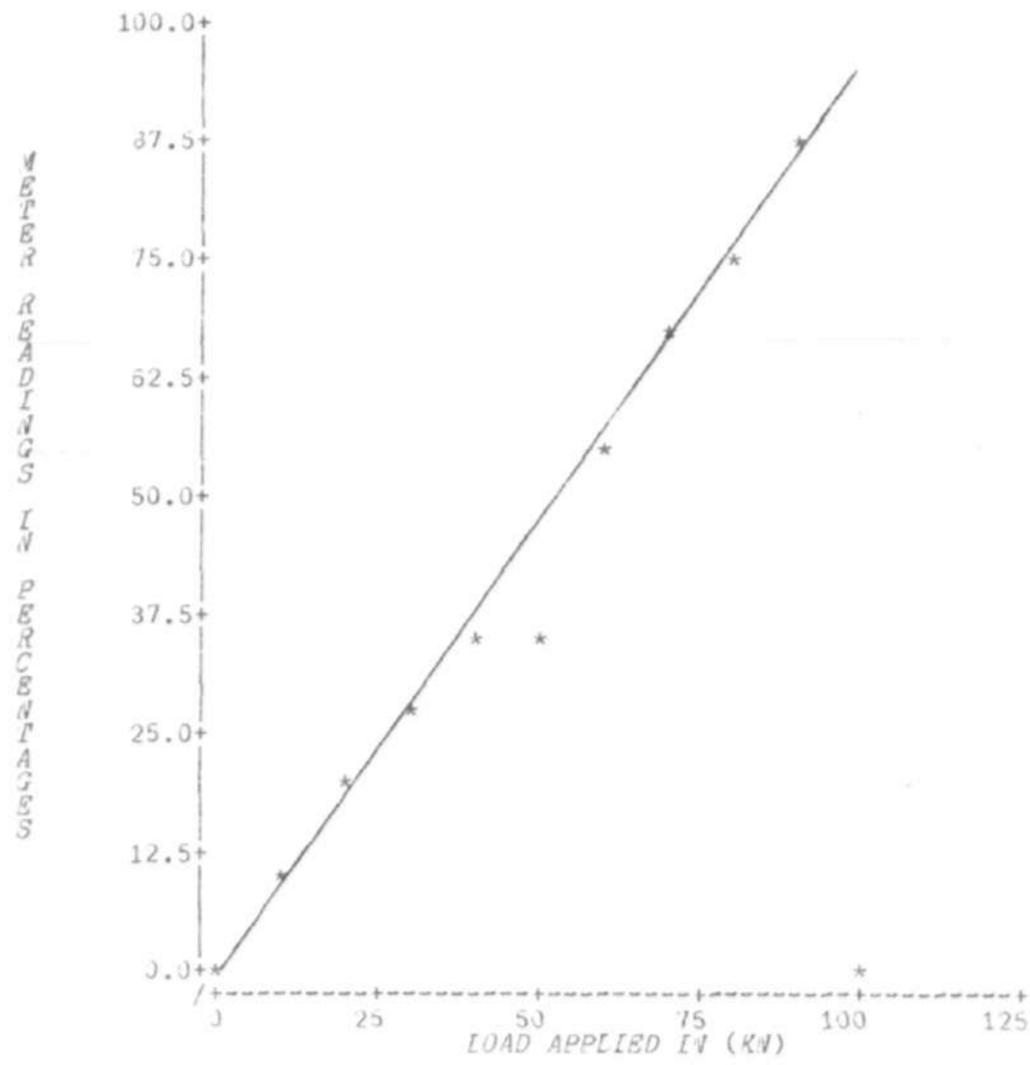
MAX. DESIGN FORCE IN MEMBER(KN)	0–200	200–450	450–750	750–1150	1150–1650	1650–2250	2250–3000
THICKNESS OF GUSSET(mm)	8	10	12	14	16	18	20

APPENDIX 2



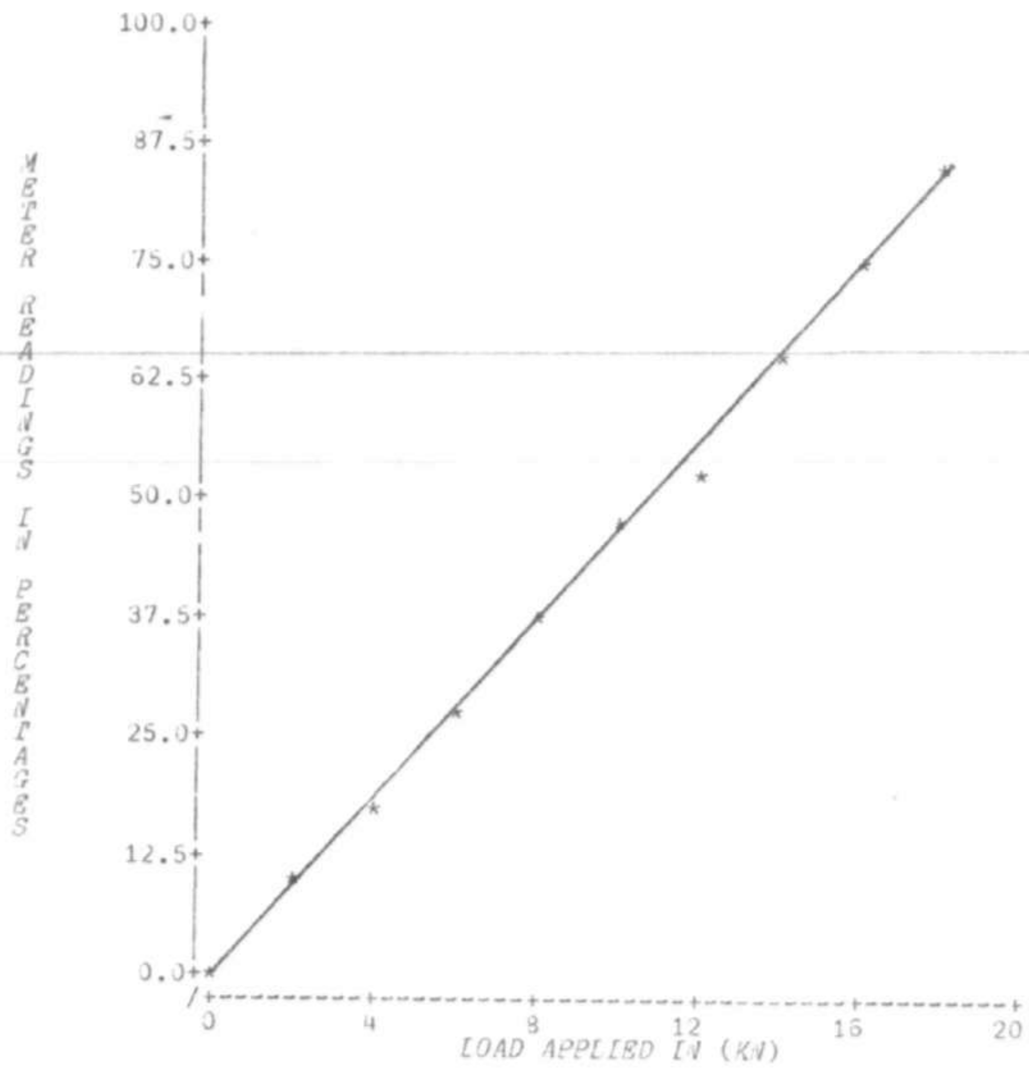
CALIBERATED GRAPH FOR LOAD CELL - 1 (10 TONS)

APPENDIX.2 CONTD.



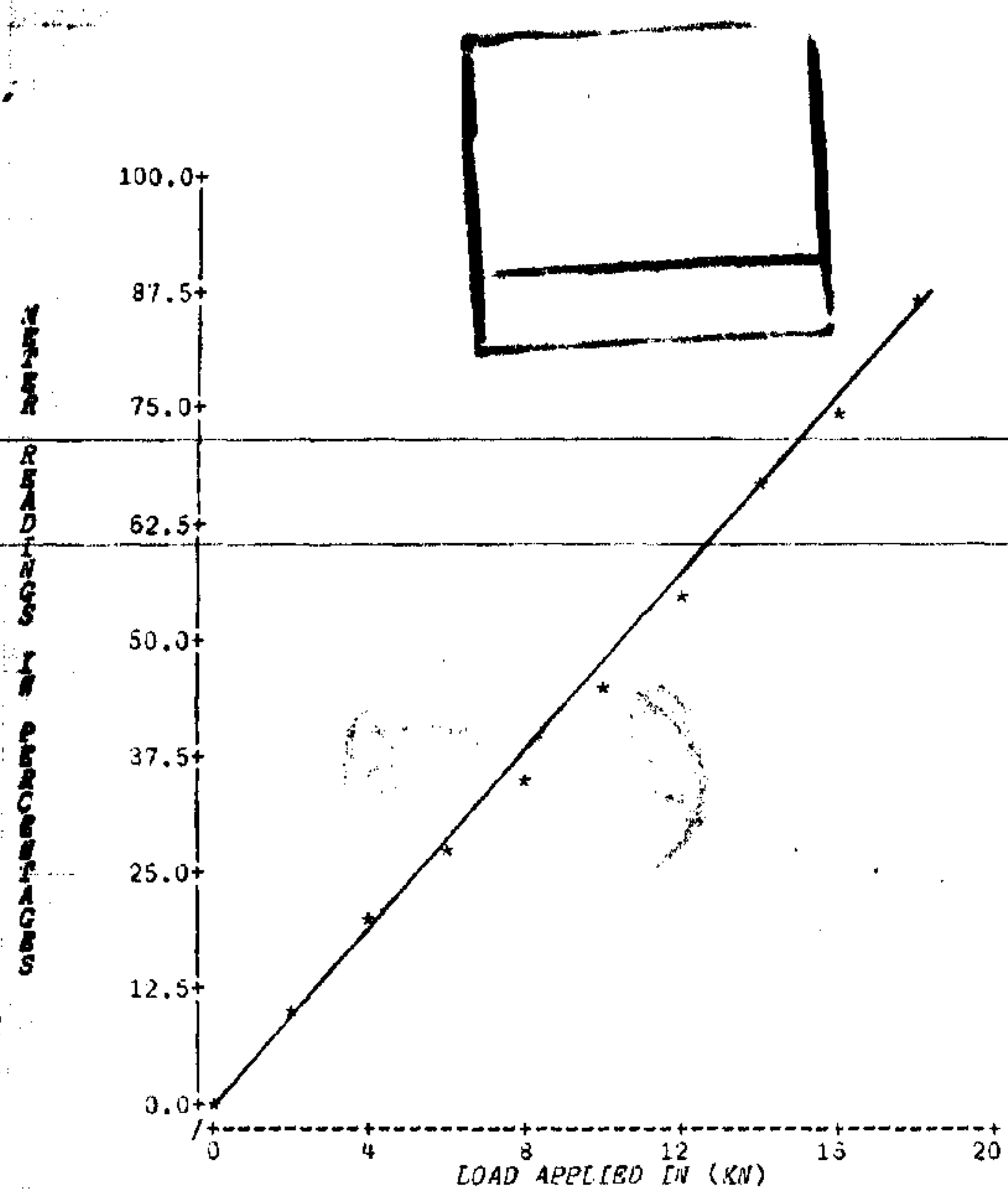
CALIBERATED GRAPH FOR LOAD CELL - 2 (10 TONS)

APPENDIX 2 CONTD.



CALIBERATED GRAPH FOR LOAD CELL - 3 (2 TONS)

APPENDIX 2 CONTD.



CALIBERATED GRAPH FOR LOAD CELL - 4 (2 TONS)