



**ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES**

**ANALYSIS AND DESIGN OF STRUTS TO NODE CONNECTION
OF
SPACE TRUSS IN BUILDINGS**

EMEBET WORKU

OCTOBER 2007

**ANALYSIS AND DESIGN OF STRUTS TO NODE CONNECTION
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**A THESIS
PRESENTED FOR THE
MASTER OF SCIENCE IN
CIVIL ENGINEERING (STRUCTURES)
DEGREE
THE ADDIS ABABA UNIVERSITY**

**PROF.NEGUSSIE TEBEDGE
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Approved by Board of Examiners

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DEDICATION

This work is dedicated to Memhir Tesfaye Shewaye, without his tireless encouragement I would have given up long ago.

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Abstract

Construction of special structures such as space frames and trusses, shells and domes is to provide large spaces with aesthetically pleasing appearance. Such structures are nowadays finding wide applications in our country. This demonstrated that number of clients interested in the space truss technology is increasing by the day. But considering the lack of standards and codes and training for such special structures design and analysis, currently most of such works are done by foreign engineering companies. These special structures are not yet locally manufactured.

This study has as the objective to design, analyze and manufacture space truss made of steel, which acts as a network of struts and nodes with emphasis on the nodes. This may serve as a reference for designers and also to identify how and in what way the nodes can be produced by locally available material and manufacturing means. It is hoped that this will make the construction cost for steel space frames lower and the supervision easier due to the use of readily available local manufacturing methods.

1. INTRODUCTION

1.1 Background

Nowadays in our country the construction of special structures such as space frames and trusses, shells and domes is starting to develop. For example, the aircraft hanger and airport terminal recently constructed at Bole Airport used special tube to node connections on the roof space trusses. Stadiums are being designed that require the use of such special connections. These show that number of clients interested in the space truss technology is increasing by the day. But considering the lack of standards and codes and direct training for such special structures design and analysis were mostly done by foreign engineering companies. And also, these special connections are not locally manufactured.

It is the aim of this study to design and analyze space truss made of steel, which acts as a network of struts and nodes with emphasis on the nodes. This may serve as a reference for designers and also to identify how and in what way the nodes can be produced by locally available steel. This will make the construction cost lower and the supervision easier due to available manpower that can be found in the country rather than the foreign one.

1.2 Objectives of the Thesis

The objective of this research is to analyze the struts to node connection in general and in special to study deeper and find out about

- What struts to node connections are
- It's advantage and usage in and out of the country
- Design and analysis of space truss and connections
- In what way we can develop it by using local resources that are available
- Design specification which will be useful for further improvement

1.3 Content of the Thesis

This thesis consists of six chapters. The first chapter deals with the general background of space frame construction in Ethiopia and the objectives of the thesis, and outcome of the study.

The second chapter is devoted to discuss the literature survey carried on types and classification of connections.

The third chapter focused on design consideration of space truss, with specific attention on loadings, material properties and design assumptions made. Moreover, it also addresses the design specification to be considered during analysis and design of space truss per the standards stated in the Design Manuals and general standards. The core and specific input of the researcher is presented under this chapter, indicating all the necessary steps and calculations.

Chapter four is devoted for how to manufacture space truss made of steel for local production.

Chapter five is demonstrating a space truss prototype testing result for local production.

The last chapter of this thesis is made to contain the conclusions drawn from the outputs and the recommendations on the basis of the findings.

1.4 Applications of the Study

The result is applicable to design nodes of struts in space truss structures with locally available steel, which is not currently practiced in Ethiopia.

From the study the client, the contractor and steel production company

would benefit. For the client the cost will be reduced, the contractor can use local manpower and steel production company will have a new variety added to its products.

2. SPACE FRAMES

2.1 General Introduction

The search for new structural forms to accommodate large unobstructed areas has always been the main objective of architects and engineers. With the advent of new building techniques and construction materials, space frames frequently provide the right answer and satisfy the requirements for lightness, economy, and speedy construction. Significant progress has been made in the process of the development of space frame.

In the past few decades, the proliferation of the space frame was mainly due to its great structural potential and visual beauty. New and imaginative applications of space frames are being demonstrated in the total range of building types, such as sports arenas, exhibition pavilions, assembly halls, transportation terminals, airplane hangars, workshops, and warehouses. They have been used not only on long-span roofs, but also on mid-and short-span enclosures as roofs, floors, exterior walls, and canopies. Many interesting projects have been designed and constructed all over the world using a variety of configurations. For example some figures of space frames built locally and abroad are shown below in Fig. 2.1, Fig. 2.2, Fig.2.3, and Fig.2.4.



Figure 2.1 Airport terminal in Addis Ababa



Figure 2.2 Assembly hall in Addis Ababa



Figure 2.3 Seahawks Stadium in Seattle



Figure 2.4 Stockholm Globe arenas in Sweden

Some important factors that influence the rapid development of the space frame can be cited as follows.

- The search for large indoor space has always been the focus of human activities. Consequently, sports tournaments, cultural performances, mass assemblies, and exhibitions can be held under one roof.
- The modern production and the needs of greater operational efficiency also created demand for large space with a minimum interference from internal supports. The space frame provides the benefit that the interior space can be used in a variety of ways and

thus is ideally suited for such requirements.

Space frames are highly statically indeterminate and their analysis leads to extremely tedious computation if performed manually. The difficulty of the complicated analysis of such systems contributed to their limited use. The introduction of electronic computers has radically changed the whole approach to the analysis of space frames. By using computer programs, it is possible to analyze very complex space structures with great accuracy and less time.

Space frames also have the problem of connecting a large number of members (sometimes up to 20) in space through different angles at a single point. The emergence of several connecting methods of proprietary systems has made great improvement in the construction of the space frame, which offered simple and efficient means for making connection of members.

2.2 Definition of the Space Frame

Sometimes structural engineers and architects seem to fail to convey with what they really want to communicate. Thus, it is appropriate to define here the term space frame as understood throughout this thesis. It is best to state a definition given by a Working Group on Spatial Steel Structures of the International Association.

A space frame is a structural system assembled of linear elements so arranged that forces are transferred in a three-dimensional manner.

According to the structural analysis approach, a space frame is analyzed by assuming rigid joints that cause internal torsions and moments in the members, whereas a space truss is assumed as hinged joints and therefore has no internal member moments. The choice between space

frame and space truss action is mainly determined by the joint connection detailing and the member geometry is no different for both. However in engineering practice, there is no absolutely rigid or hinged joint. For example, a double layer flat surface space frame is usually analyzed as hinged connections, while a single layer curved surface space frame may be analyzed either as hinged or rigid connection. The term space frame will be used to refer to both space frames and space trusses.

2.3 Systems of Space Frames

Double layer grids or flat surface frames acts as a network of struts and nodes. The joint module determines the position of every point of direct connection from the chosen system. Each node must be connected with at least three non-coplanar struts to maintain stability and to prevent translation

2.4 Advantages of Space Frames

Use of space frames has the following advantages:

1. The ability to create multipurpose column-free large architectural spaces;
2. Light weight reduces their susceptibility to seismic forces;
3. Use of small elements facilitates their mass production;
4. Transportation and handling;
5. Ease of assembly without highly skilled labor and with limited access;
6. Aesthetic appeal, visual elegance, and interesting geometric patterns;
7. An open form that allows easy installation of mechanical and electrical services.

2.5 Jointing Systems

2.5.1 General Description

The jointing system is an extremely important part of a space frame design. An effective solution of this problem may be said to be fundamental to successful design and construction. The type of jointing depends primarily on:-

- The connecting technique, whether it is bolting, welding, or applying special mechanical connectors.
- The shape of the members, This usually involves a different connecting technique depending on whether the members are circular or square hollow sections or rolled steel sections.

The joints for the space frame are more important than the ordinary framing systems because more members are connected to a single joint. Furthermore, the members are located in a three-dimensional space, and hence the force transfer mechanism is more complex. The role of the joints in a space frame is so significant that most of the successful commercial space frame systems utilize proprietary jointing systems. Thus, the joints in a space frame are usually more sophisticated than the joints in planar structures, where simple gusset plates will suffice.

In designing the jointing system, the following requirements should be considered:-

- The joints must be strong and stiff
- Simplicity in terms of structurally and mechanically
- Easy to fabricate without recourse to more advanced technology.
- The eccentricity at a joint should be kept to a minimum, yet the joints detailing should provide for the necessary tolerances that may be required during the construction.
- The joints of space frames must be designed to allow for easy and effective maintenance.

The cost of the production of joints is one of the most important factors affecting the final economy of the finished structure. Usually the steel consumption of the connectors will constitute 15 to 30% of the total. Therefore, a successful prefabricated system requires joints that must be repetitive, mass produced, simple to fabricate, and able to transmit all the forces in the members interconnected at the node.

2.5.2 Classification of Proprietary Jointing Systems

The vast majority of buildings with space frames are designed using one of many proprietary systems, such as:-

2.5.2.1 Mero System

The Mero connector, introduced in 1940 by Dr. Mengerhanusen, proved to be extremely popular and has been used for numerous temporary and permanent buildings. Its joint consists of a node that is spherical hot-pressed steel forging with flat facets and tapped holes. Members are circular hollow sections with cone-shaped steel forgings welded at the ends, which accommodate connecting bolts. Bolts are tightened by means of a hexagonal sleeve and dowel pin arrangement, resulting in a completed joint such as that shown in Fig.2.5. Up to 18 members can be connected at a joint with no eccentricity [1], [2].

The manufacturer can produce nodes of different size with diameter ranging from 46.5 mm to 350 mm, the corresponding bolts ranging from M12 to M64 with a maximum permissible force of 1413kN. A typical space-module of a Mero system is a square pyramid ($\frac{1}{2}$ Octahedron) with both chord and diagonal members of the same length "a", angles extended are 90° or 60° . Thus, the depth of the space-module is $(a/\sqrt{2})$ and the vertical angle between diagonal and chord member is 54.7° . The steel node is shown in Fig. 2.5.

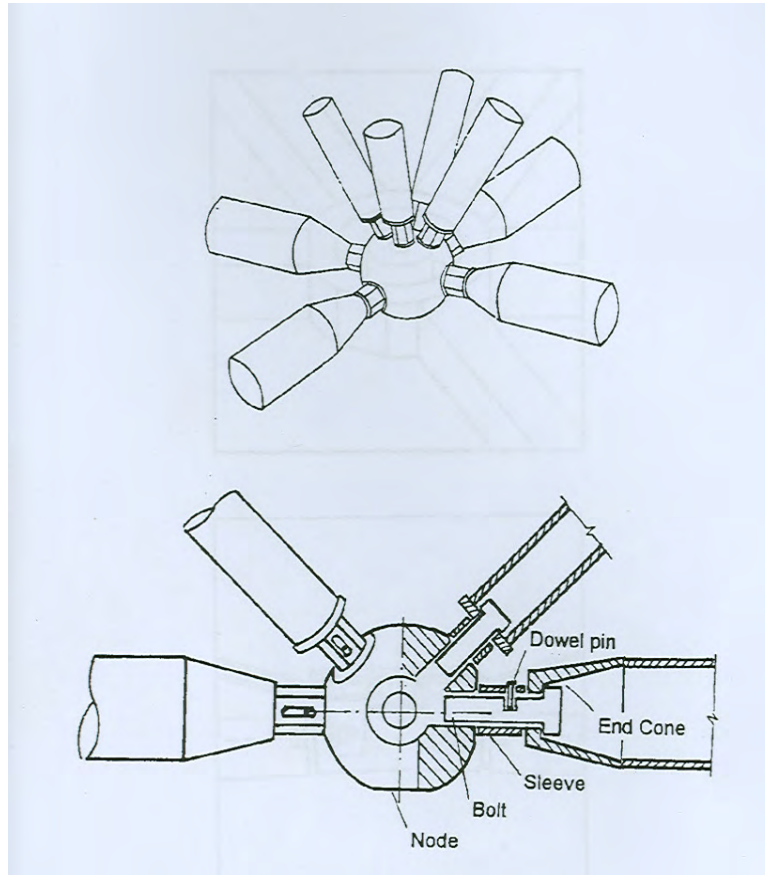


Figure 2.5 Mero system

The Mero connector was originally developed for double layer grids. Due to the increasing usage of non-planar roof forms, it is required to construct the load-bearing space frame integrated with cladding element. A new type of jointing system called Mero Plus System was developed so that a variety of curved and folded structures are possible. Square or rectangular hollow sections are used to match the particular requirements of the cladding can transmit shear force, resist torsion, and in special cases can resist bending moment. There are four groups in this system, which are described as follows [1].

- a. Disc Node (Type TK) (see Fig. 2.6)-This is a planar ring-shaped node connecting 5 to 10 member of square or rectangular sections. A single bolt is used to connect the node and member and depth of the node is equal to the member section depth. Such jointing systems can transmit shear force and resist rotation. In the following discussion, the U-angle is designated as the angle between two members connected to the same node. Also, the V-angle is the angle between the member axis and the normal in the plane of the node which is a measure of curvature. For a disc node, the U-angle varies from 30° to 80° and the V-angle varies from 0° to 10° . This type of jointing system is essentially pin-jointed connections and is suitable for latticed shells made of triangular meshes.

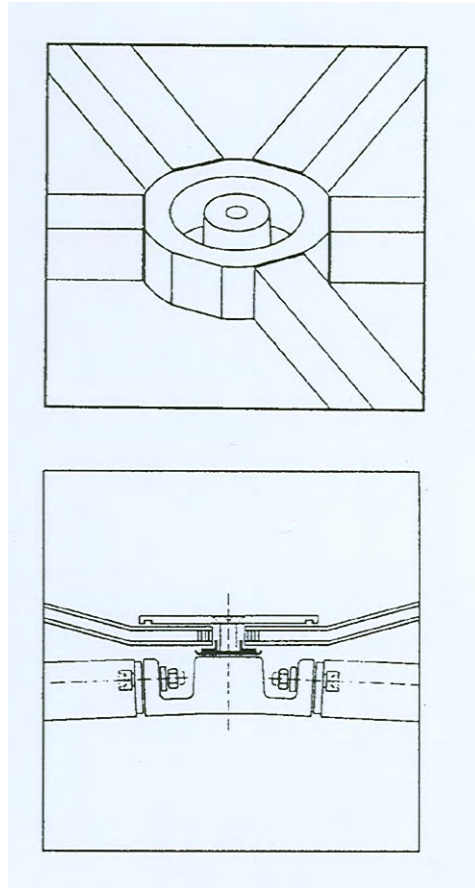


Figure 2.6 Disk node – (Type TK)

- b. Bowl Node (Type NK) (see Fig. 2.7) - This is a hemispherical node connecting top chord and diagonal members. Single bolted connection from node to member is used. The top chord members of square or rectangular sections can be loaded in shear and are fitted flush to the nodes. Bowl nodes are used for double layer planar and curved surfaces, in particular buildings irregular in plan or pyramid in shape. The diagonals and lower chords are constructed in an ordinary Mero system with circular tubes and spherical nodes.

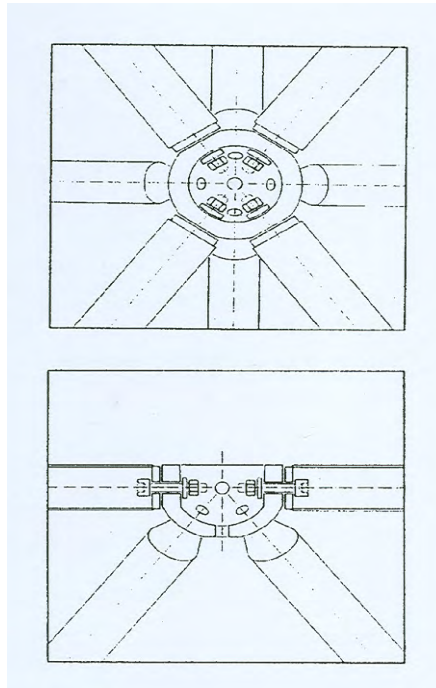


Figure 2.7 Bowl node (type NK)

- c. Cylinder Node (Type ZK) (see Fig. 2.8) – This is a cylindrical node with a multiple bolted connection that can transmit bending moment. Usually the node connection angle varies: 30° to 10° for V-angle, 0° to 10° for V-angle Cylinder nodes are used in singly or doubly curved surface of latticed shells with trapezoidal meshes where flexural rigid connections are required.

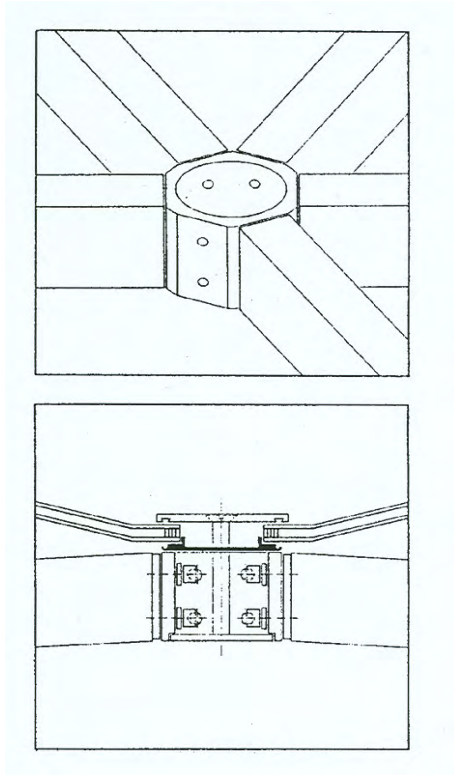


Figure 2.8 Cylinder node (type ZK)

- d. Block Node (Type BK) (see Fig. 2.9) - This is a block-or prism-shaped solid node connecting members of square or rectangular sections. The U-angle varies from 70° to 120° and V-angle varies from 0° to 10° . It can be used for single or doubly curved surfaces with pin-jointed or rigid connections where the number of members is small. The structure is of simple geometry and small dimensions.

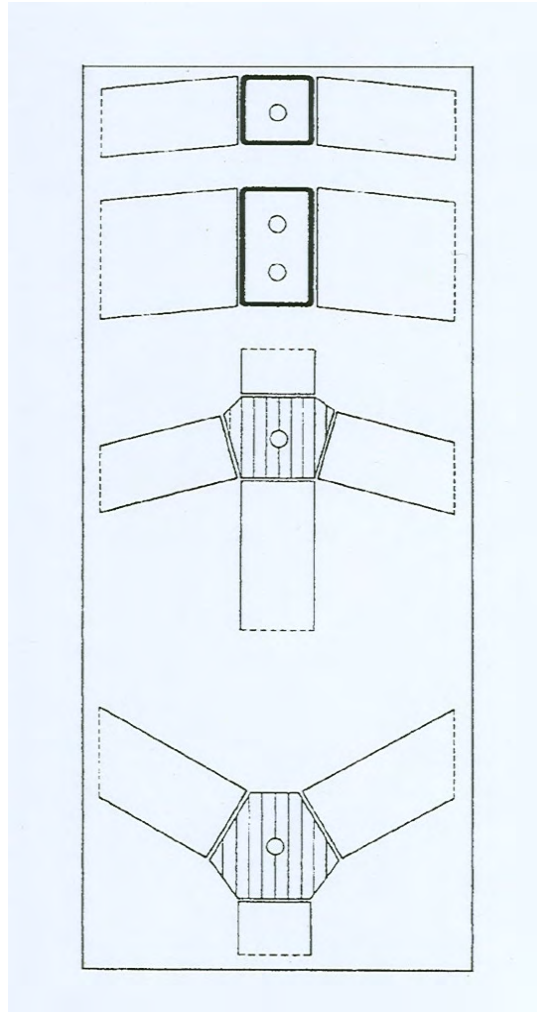


Figure 2.9 Block node (type BK)

2.5.2.2 Space Deck System

The Space Deck system, introduced in England in the early fifties, utilizes pyramidal units that are fabricated in the shop, as shown in Fig. 2.10. The four diagonals made of rods or bars are welded to the corners of the angle frame and joined to a fabricated boss at the apex. It is based on square pyramid units that form a configuration of square on square offset double layer space grids.

The units are field-bolted together through the angle frames. The apexes of the units are connected in the field by using tie bars made from high-

tensile steel bars. Camber can be achieved by adjusting the tie bar lengths, since right-hand and left-hand threading is provided in the boss. The space Deck system is usually used for buildings of span less than 40 m with a standard module and depth of 1.2 m. A minimum structural depth of 0.75 m is also provided. For higher design loading and larger spans, alternative production modules 1.5 m and 2 m with the same depth as the module are also available.

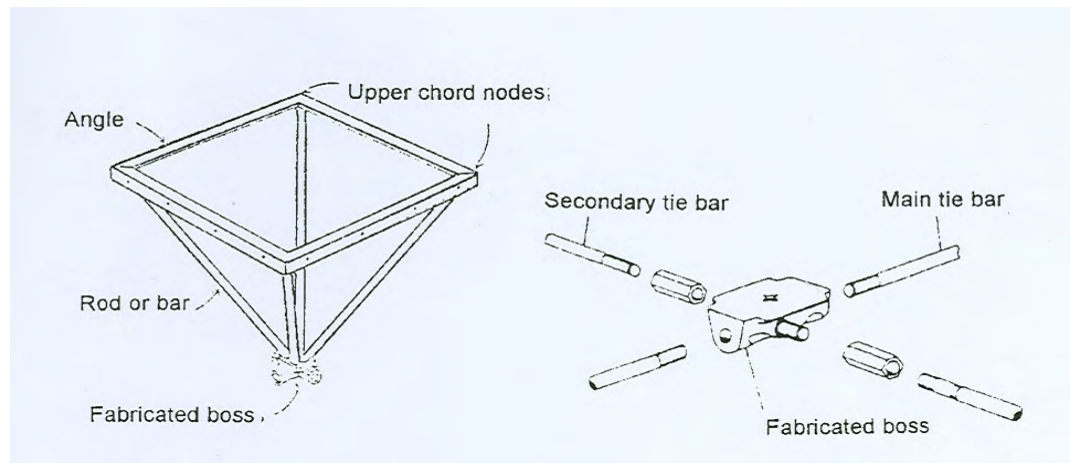


Figure 2.10 Space deck system

2.5.2.3 Triodetic System

The joint for the Triodetic system, developed in Canada, Consists of an extruded aluminum connector hub with serrated keyways. Each member end is pressed in order to form a coined edge that fits into the hub keyway. The joints is completed when the members are inserted into the hub, washers are placed at each end of the hub, and a screw bolt is passed through the center of hub, as shown in Fig. 2.11. The Triodetic connector can be used for any type of three-dimensional space frame. Originally only aluminum structures were built in this system, but later space frames were erected using galvanized steel tubes and aluminum hubs. Triodetic double layer grids have been used up to 33m clear span.

The basic module can be almost any size up to approximately 2.7m in square. The depth is usually 70% of the module size [1].

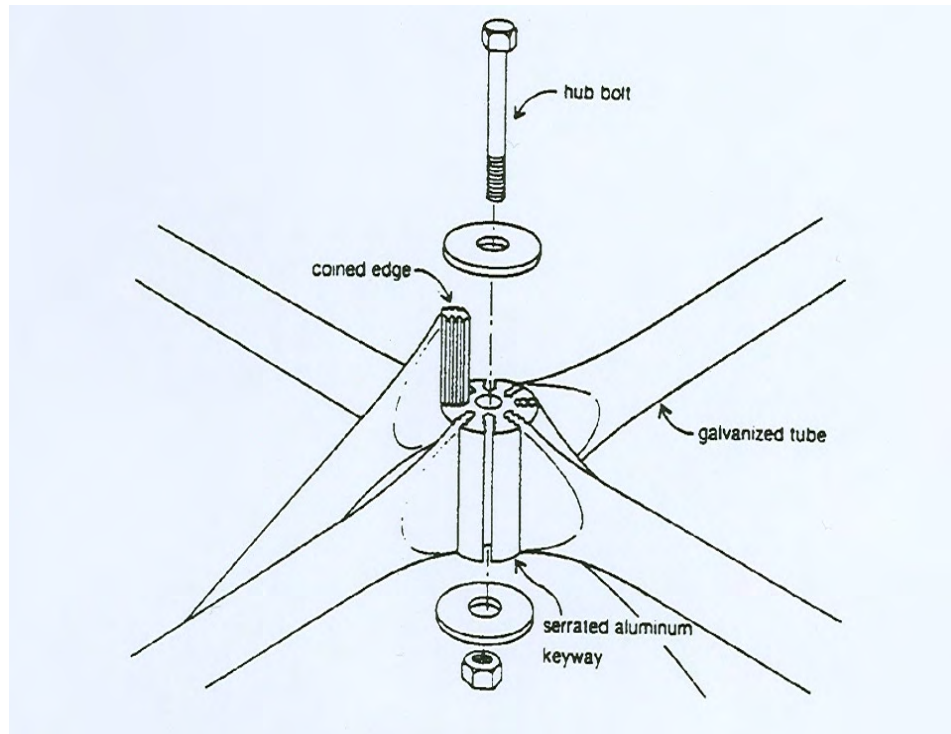


Figure 2.11 Triodetic system.

2.5.2.4 Unistrut System

The Unistrut system was developed in the U.S. in the early fifties. Its joint consists of a connector plate that is press-formed from steel plate. The members are channel-shape cold-formed sections and are fastened to the connector plate by using bolt at each end. The connectors for the top and bottom layers are identical and therefore the Unistrut double layer grids consist of four components only, i.e., the connector plate, the strut, the bolt, and the nut (see Fig. 2.12). The maximum span for this system is approximately 40 m with standard modules of 1.2 m and 1.5m. The name of Moduspan has also been used for this system [1].

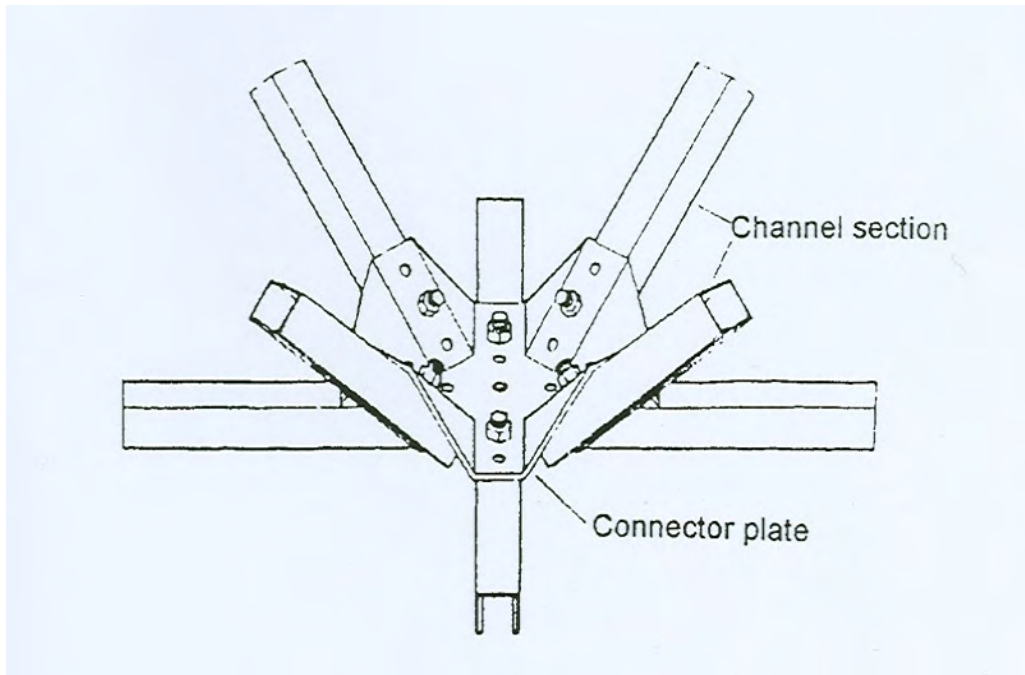


Figure 2.12 Unistrut system

2.5.2.5 Oktaplatte System

The Oktaplatte system utilizes hollow steel spheres and circular tube members that are connected by welding. The node is formed by welding two hemispherical shells together which are made from steel plates either by hot or cold pressing. The hollow sphere may be reinforced with an annular diaphragm. This type of node was popular at the early stage of development of space frames. It is also useful for the long span structures where other proprietary systems are limited by their bearing capacity. Hollow spheres with diameter up to 500 mm have been used. It can be applied to single layer latticed shells as the joint can be considered as semi-or fully rigid. The whole jointing system and the hollow sphere with its parts are shown in Fig. 2.13 [1].

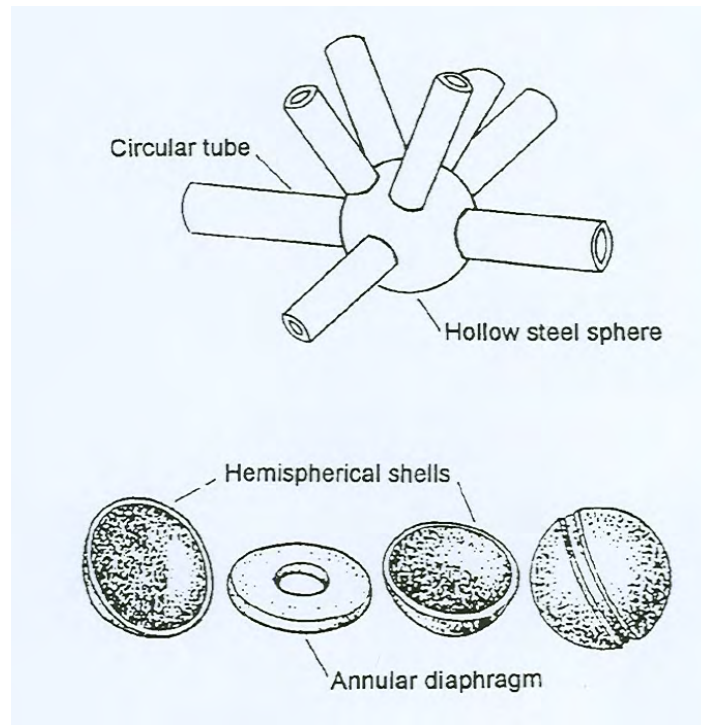


Figure 2.13 Oktaplatts system

2.5.2.6 Unibat System

The Unibat system, developed in France consists of pyramidal units by arranging the top layer set on a diagonal grid relative to the bottom layer. The short length of the top chord members results in less material being required in these members to resist applied compressive and bending stresses. The standard units are connected to the adjacent units by means of a single high-tensile bolt at each upper corner. The apex corners of the pyramidal unit may be forgings, to which the top chord and web members are welded.

The units may employ any combination of rolled steel or structural sections. As shown in Fig. 2.14, the top chords are rolled I sections and members are square hollow sections. The bottom layer is formed by a

two-way grid of circular hollow sections which are interconnected with the apex by a single vertical bolt. Numerous multi-story buildings, as well as large span roofs over sports buildings have been built using the Unibat system since 1970 [1].

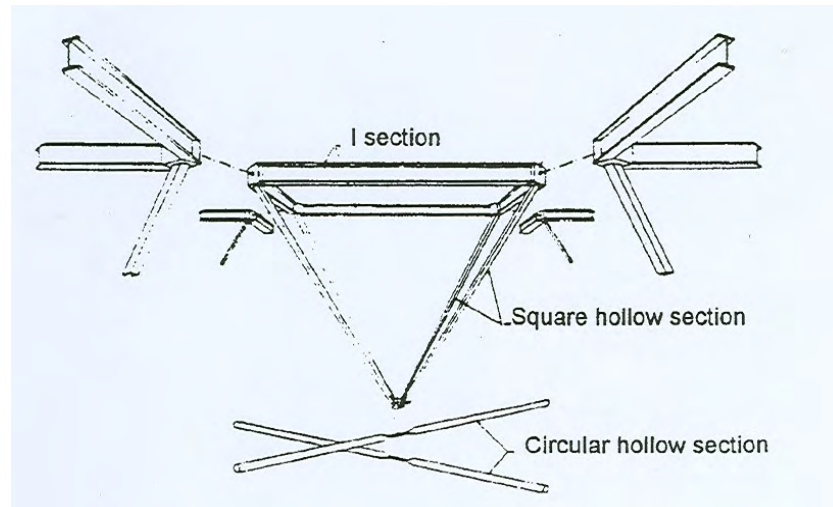


Figure 2.14 Unibat system

2.5.2.7 Nodus System

The Nodus system was developed in England in the early seventies. Its joint consists of half-casings which are made of cast steel and have machined grooves and drilled holes, as shown in Fig. 2.15. The chord connections are made of forged steel and have machined teeth, and are full-strength welded to the member ends. The teeth and grooves have an irregular pitch in order to ensure proper engagement. The forked and grooves have an irregular pitch in order to ensure proper engagement. The forked connectors are made of cast steel and are welded to the diagonal members. For the completed joint, the corresponding intersecting points of the chord members. This eccentricity produces amount of local bending in the chord members and the joint

components. Destructive load tests performed on typical joints usually result in failures due to bending of the teeth in the main half-casting. The main feature of the Nodus jointing system is that all fabrication is carried out in the workshop so that only the simplest erection techniques are necessary for the assembly of the structure on-site [1].

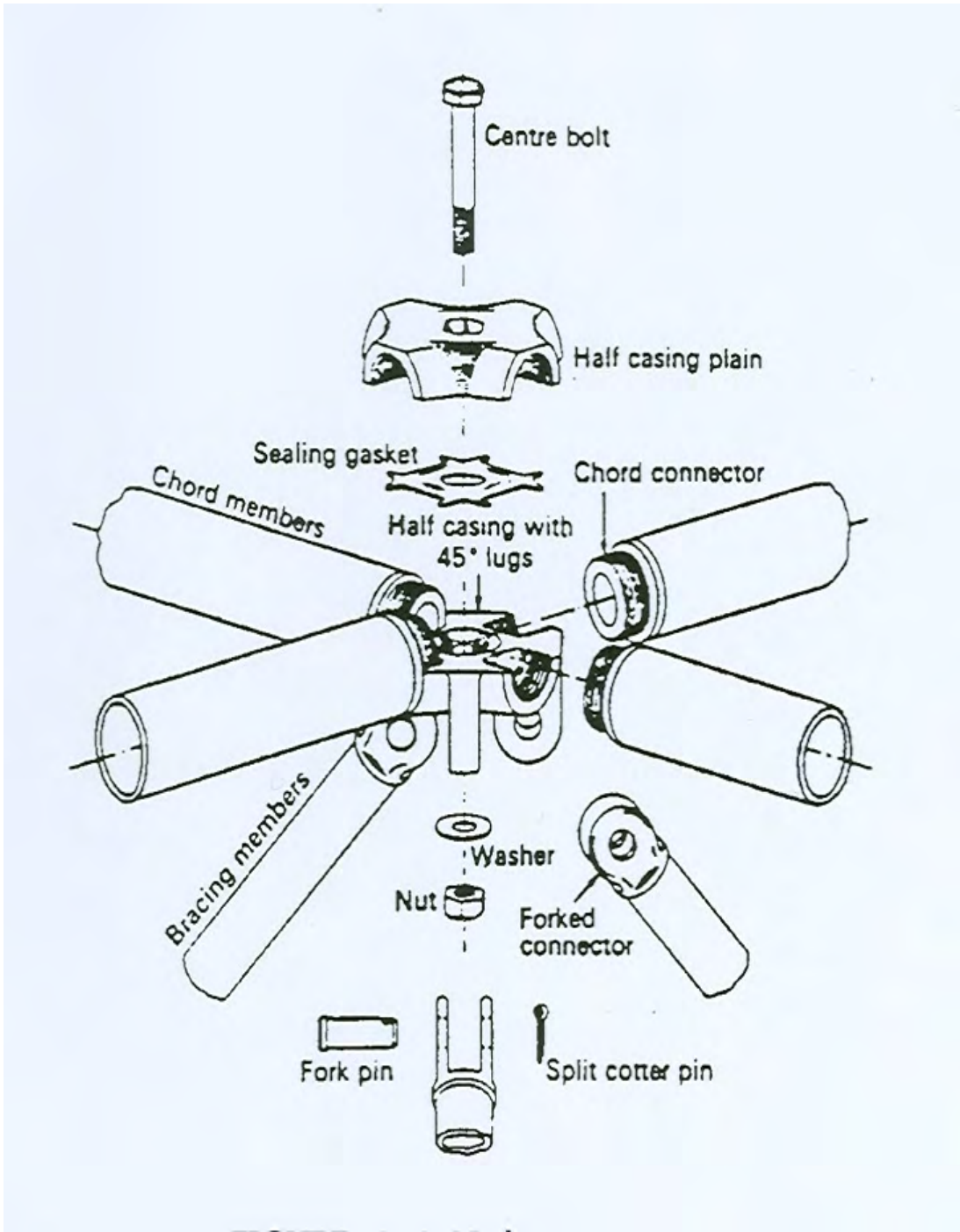


Figure 2.15 Nodus system

2.5.2.8 NS Space Truss

The NS Space Truss system was introduced around 1970 by the Nippon Steel Corporation. It originates from the space truss technology developed for the construction of the huge roof at the symbol zone for Expo '70 in Japan. The NS space Truss system has a joint consisting of thick spherical steel shell connectors open at the bottom for bolt insertion. The structural members are steel hollow sections having specially shaped end cones welded to both ends of the tube. End cones have threaded bolt holes. Special high strength bolts are used to join the tubular members to the spherical shell connector. The NS nodes enable several members to be connected to one node from any direction without any eccentricity of internal forces. The NS Space Truss system has been used successfully for many large span double and triple layer grids, domes, and other space structures. The connection detail of the NS node is shown in Fig. 2.16 [1]

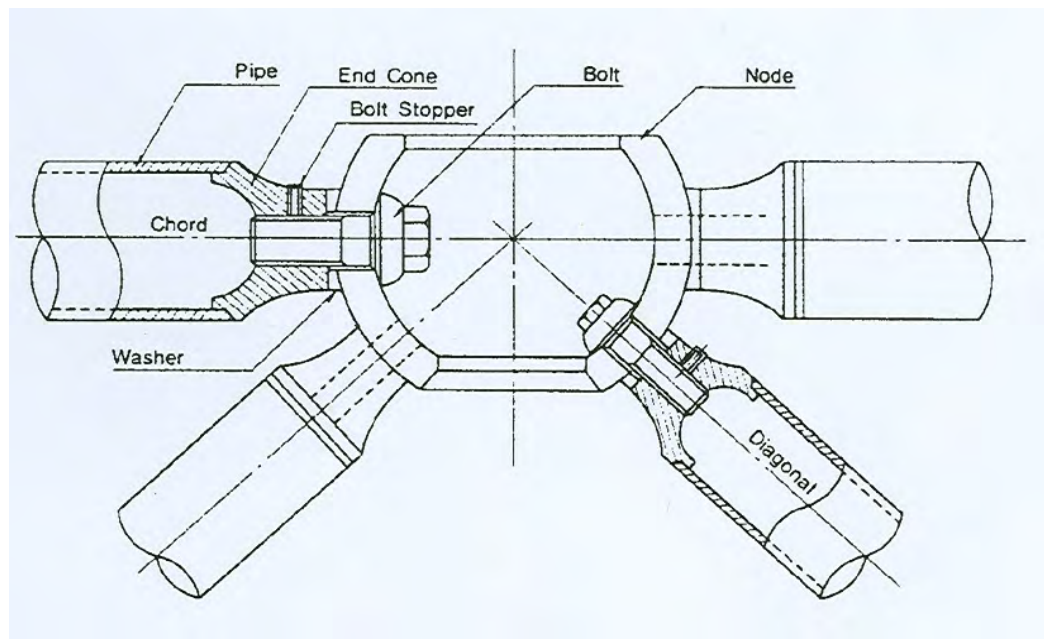


Figure 2.16 NS space truss

What makes each system unique is

- the geometry of the node

- how the struts are connected to the nodes
- the method of manufacturing the nodes and struts
- the polyhedral units possible with each system.

2.6 Usage within the Country

Space frames are recently finding many applications with in Ethiopia. For example, the aircraft hanger and airport terminal recently constructed at Bole Airport used special tube to node connections on the roof space trusses. Stadiums are being designed that require the use of such special connections. A new building near Mexico square in front of Philips Building uses double grid space truss for the entrance into the building. These show that number of clients interested in the space truss technology is increasing by the day.

2.7 Materials used for Space Frame

Most buildings with space frames are made of high strength or mild steel tubes with circular or square shapes as well as channels and special forms, either hot rolled or cold formed. Aluminum, wood, and composites have also been used in different cross sections. The nodes for steel and aluminum space frame have been designed in several shapes and forms based on their strength and aesthetics.

3. Analysis and Design

3.1 General

Space frames are currently analyzed using linear elastic theory. The load-carrying capacity of a space frame is usually limited by the first member or set of members to fail. Connections are either made of readily available standard shapes or proprietary prefabricated pieces. Connection pieces are designed for structural efficiency or appearance. It is assumed that the connection will be strong enough so that any failure will take place in the struts or ties.

The struts and ties are treated as straight, axially loaded pin-ended members, for which the load-deformation relationship is linear up to buckling in compression or yielding in tension. Tension members ideally would yield, but may rupture in a brittle manner at the net section or at the connection.

For the research, a connection that is simple to use and manufacture is chosen from the proprietary connection types. Of the types described in the classification Mero system is chosen, a connection similar to Mero system will be manufactured from locally available steel. This mero system is used for connection steel tubes with circular shapes and so, locally available circular tubes can be applied.

The advantage of mero connection is that the axes of all members pass through the center of the node eliminating eccentricity loading at the joint. Thus, the joint is only under the axial forces. Then tensile forces are carried along the longitudinal axis of the bolts and resisted by the tube members through the end cones. The compressive forces produce negligible stresses in the bolt; they are distributed to the node through the hexagonal sleeve.

3.2 Preliminary Planning

In the preliminary stage of planning a space frame to cover a specific building, a number of factors should be studied and evaluated before proceeding to structural analysis and design.

- Choosing the general form of the building and the type of space frame appropriate. (Surface shape, number of layers, grid pattern)
- The geometry of the space frame
- Connecting joints
- Construction technology

Space frame are classified as single-, double-, or multi-layered structures which may be flat, resulting in grid structures, or may be curved in one or two directions, forming barrel vaults and dome structures.

Double layer grids consist of two planar networks of members forming the top and bottom layers parallel to each other and interconnected by vertical and inclined web members.

Double layer grids are usually composed of basic elements such as:

- A planar latticed truss
- A pyramid with a square base that is essentially a part of an octahedron
- A pyramid with a triangular base (tetrahedron)

A large number of types of double layer grids can be formed by these basic elements. They are developed by varying the direction of the top and bottom layers with respect to each other and also by the positioning of the top layer nodal points with respect to the bottom layer nodal points.

According to the form of basic elements, double layer grids can be divided in two groups

- Latticed grids

- Space grids

The latticed grids consist of intersecting vertical latticed trusses and form a regular grid. Two parallel grids are similar in design, with one layer directly over the top of another. Both top and bottom grids are directionally the same.

The space grids consist of a combination of square or triangular pyramids. This group covers the so-called offset grids, which consist of parallel grids having an identical layout with one grid offset from the other in plane but remaining directionally the same, as well as the so-called differential grids in which two parallel top and bottom grids are of a different layout but are chosen to coordinate and form a regular pattern [see Fig. 3.1].

3.3 Choosing type of double layer grid

In the preliminary state of design, it is most important to choose an appropriate type of double layer grid that will have direct influence on the overall cost and speed of construction. It should be determined comprehensively by considering

- Shape of the building plan
- The size of the span
- Support conditions
- Magnitude of loading
- Roof construction
- Architectural requirements

In general, the system should be chosen so that the space grid is built of relatively long tension members and short compression members. In choosing the type, the ratio of a longer span to a shorter span has more

influence than the span of the double layer grids.

The supports of double layer grids may directly rest on the columns or on ring beams connecting the columns or exterior walls. The columns for double layer grids must support gravity loads and possible lateral forces.

The main types of double-layer grids in common use are shown in Fig. 3.1. The behavior of both single-layer and double-layer space trusses is influenced to a great extent by the support positions of the structure. The effects of joint and overall frame rigidity also have a commanding influence and affect the buckling behavior of the compression members within the structure.

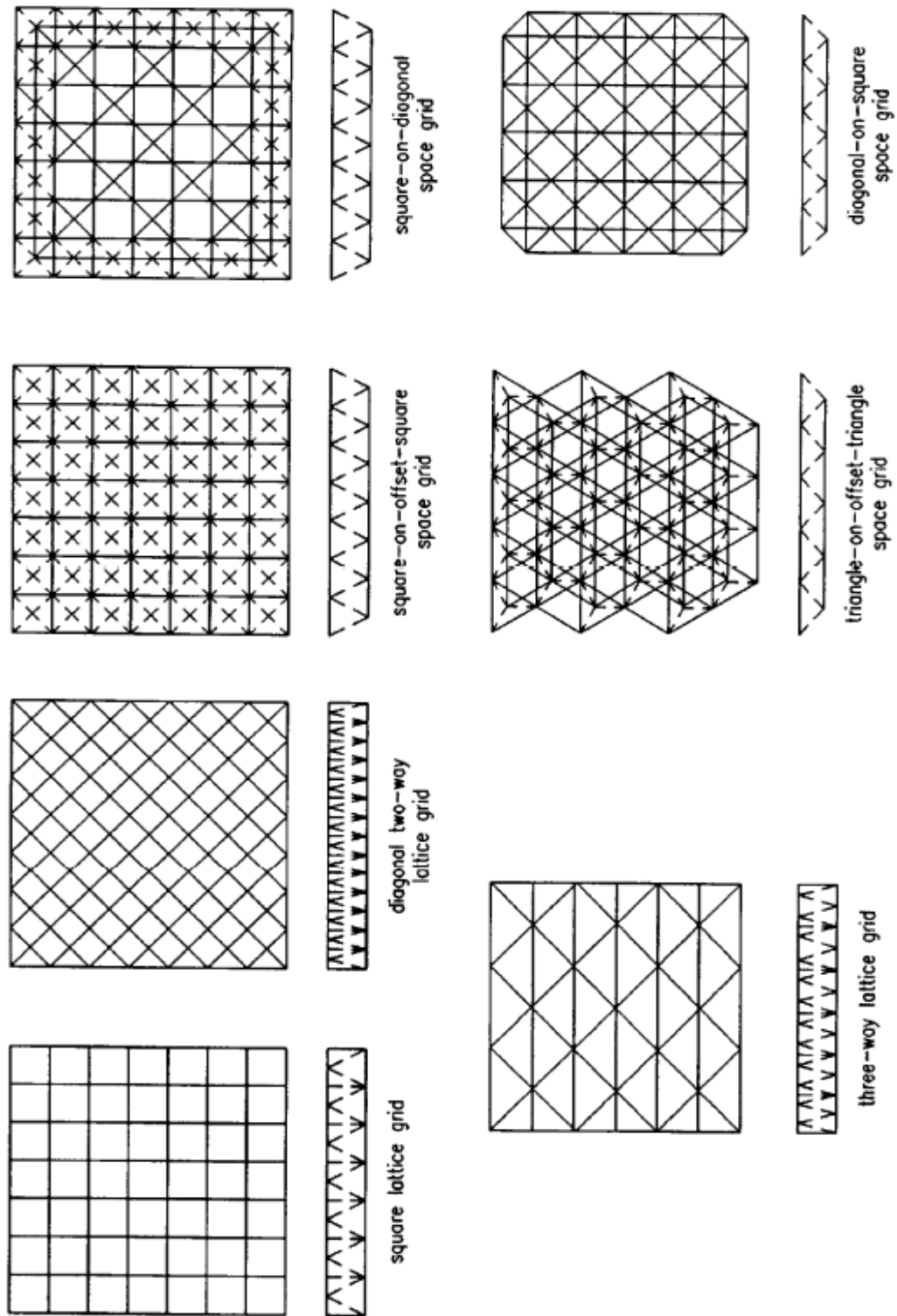


Figure 3.1 Lattice and space grid

3.4 Design Parameters

Before any work can proceed on the analysis of a double layer grid, it is necessary to determine the depth and the module size. The depth is the distance between the top and bottom layers and the module is the distance between two joints in the layer of the grid. There are many factors influencing these parameters, such as

- The type of double layer grid
- The span between the supports
- The roof cladding
- The proprietary system used.

The depth and module size of double layer grids are usually determined by practical experience. It is best to determine these parameters through structural optimization.

3.5 Structural Analysis and Design

The planning and proportioning of a structure to satisfy functional, economical and aesthetic requirements is known as structural design.

Some of the factors which will govern the design are

1. **Dead loads** – The design load is established on the basis of the actual loads which may be expected to act on the structure and is of constant magnitude. The weight of various accessories – cladding, supported lighting, heat ventilation equipment- and the weight of the space frame comprise the total dead load.
2. **Live load** – Live load is specified by the local building code [EBCS 1]. Rain load may be important in a tropical climate especially if the drainage provisions are insufficient. Pounding results when the water on a double layer grid flat roof accumulates faster than it runs off, thus causing excessive load on the roof.
3. **Wind load** - Wind load is specified by the local building code [EBCS 1].
4. **Temperature effect** – Most space frames are subject to thermal expansion and contraction due to changes in temperature, and thus may

be subject to axial loads if restrained.

Factor which has contribution for minimizing the effect of thermal expansion

- The choice of support locations
- Intermediate columns and type of support, i.e. fixed, free rotation and translation.
- The geometry of members adjacent to the support.

For a double layer grid, if it satisfies one of the following requirements, the calculation for temperature effect may be exempted.

- a) The joints on supports allow the double layer grid to move horizontally.
- b) Double layer grids of less than 40m span are supported along perimeters by independent reinforced concrete columns or brick pilaster.
- c) The displacement at the top of the column due to a unit force is greater or equal to the value calculated according to the following formula [1].

$$\delta = (L/2\xi EA)(Ea\Delta_t/0.05(s)-1)$$

Where

L = span of double layer grid in the direction of checking temperature effect

E = modulus of elasticity

A = arithmetic mean value of the cross-sectional area of members in the supporting plane (top or bottom layer)

a = coefficient of thermal expansion

Δ = temperature difference

s = allowable stress of steel

ξ = coefficient, when the chords in the supporting plane are arranged in orthogonal grids $\xi = 1$, in diagonal grids $\xi = 2$, and in three-way grids $\xi = 2$

5. **Construction loads**- During construction, structures may be subjected to loads different from the design loads after completion, depending on the sequence of construction and method of scaffoldings. For example, a space frame may be lifted up at points different from the final supports, or it may be constructed in blocks or strips. Therefore, the whole structure, or a portion of it, should be checked various stages of construction.

3.5.1 Static analysis

There are generally two different approaches in use for statical analysis. The first approach is to apply directly as a general assembly of discrete member, i.e., discrete method. In the second approach, the structure is represented by an equivalent continuum like a plate or shell, i.e., continuum analogy method.

The advent of computers has radically changed the whole approach to the analysis and design of space frames. It has also been realized that matrix methods of analysis provide an extremely efficient means for rapid and accurate treatment of many types of space structures. In the matrix analysis, a structure is represented as a discrete system and all the usual equations of structural mechanics are written conveniently in matrix form. Thus, matrix analysis is particularly suitable to computer formulation, with an automatic sequence of operations.

The two common formulations of the matrix analysis are

- The stiffness method
- Flexibility method

The stiffness method is also referred to as the displacement method because the displacements of the redundant members are treated as unknowns [1].

The flexibility method (or force method) treats the forces in the members as unknowns. Of these two methods, the displacement method is widely used in most computer programs [1].

In the displacement method, the stiffness matrix of the whole structure is obtained by adding appropriately the stiffness matrixes of the individual elements. Supports are then introduced because the displacements at these points are known. A set of simultaneous equations are solved for displacements. From the joint displacements the member elongation can be found and hence the member forces and reaction at supports.

The matrix displacement method is by far the most accurate method for the analysis of space frames. It can be used without any limit on the type and shape of the structure, the loadings, the supporting conditions, or the variation of stiffness. The effect of temperature or uneven settlement of supports also can be analyzed conveniently by this method.

For design work, a special purpose computer program for space frames is preferred; otherwise the input of generating nodal coordinates and member connectivity plus loading information will be a tremendous amount of work. Sophisticated computer programs provide the functions of automatic design, optimization, and drafting.

Double layer grids can be analyzed as pin connected and rigidity of the joints does not change the stress by more than 10 to 15%. In the displacement method, bar elements are used with three unknown displacements in x, y, and z directions at each end.

When using a computer, the engineer must know the assumptions on which the program is based, the particular conditions for its use (boundary conditions for example), and the manner of introducing the input data. In the static analysis of the space frames, care should be taken on the following issues:

- **Support conditions**

A fixed support (bolted or welded) in construction should not be treated literally as a completely fixed node in analysis. As a matter of fact, most space frames are supported on columns or walls that have a lateral flexibility. Upon the action of the external loads, there will be lateral displacements on the top of columns. Therefore, it is more reasonable to assume the support as horizontally movable rather than fixed, or as an elastic support by considering the stiffness of the supporting column.

- **Criterion for the number of reanalysis**

It is necessary to limit the number of reanalysis. In practice, certain criterion is specified such that the reanalysis will terminate automatically. One of the criterions is suggested as the number of the modified members less than 5% of the total number of members. Usually three or four runs will produce a satisfactory result.

- **Checking of computer output**

It is dangerous for an engineer to rely on the computer output as being infallible. Always it is advisable to try to estimate and anticipate results. A simple manual calculation by approximate method and comparing it with computer output will be beneficial. By doing so, an order of magnitude for the results can be obtained. In this operation intuition plays an important role. At the same time, simple checks should be done to test the reliability of the computer program such as

- The equilibrium of forces at nodes and the equilibrium of total loading with the summation of reactions.
- A check on the deflections along certain axis of the structure. The size and location of any large deflection should be noted.
- All deflections should be scanned to look for possible bad solutions caused by improper modeling of the structure.

- **Earthquake Resistance**

- One of the important issues that must be taken into consideration in the analysis and design of space frames is the earthquake excitation in case the structure is located in seismic area.
- Double layer grids can be treated as a pin-connected space truss system and their free vibration is formulated as an equation of motion for a freely vibrating undamped multi-degree-of-freedom system. By solving the generalized eigenvalue problems, the frequencies and vibration modes are obtained.
- In the region where the maximum vertical acceleration is $0.05g$, usually the earthquake effect is not a governing factor in design and it is not necessary to check the forces induced by vertical or horizontal earthquake [1].

3.5.2 Components of Space Frames

3.5.2.1 Nodes

The node is a solid steel sphere. To suit different force and connection directions, the node possesses the most neutral form of a sphere with correspondingly directed axial/radial threaded bores to accommodate the member bolts. To improve seating of the spanner-sleeve and for reasons of tolerance and series production, the sphere is machined-flat around the threaded holes.

Nodes in a space frame serve the purpose of establishing a connection between the three-dimensionally arranged members, which implies that nodes hold both themselves and the members in a fixed position and guarantee the balance of forces. The longitudinal forces within the members act radially on the nodes, the forces having differing magnitudes and sometimes even reversing direction. The force is transmitted between the member and the node via a single-bolt axial connection with an intermediate spanner-sleeve.

The size of the spherical nodes is optimized on the basis of the following three parameters:

1. The magnitudes of the forces to be transmitted;
2. Avoiding mutual interference between neighboring members and between bolts within the sphere;
3. Minimization of types of components to ensure maximum efficiency and economy of production. A standard range of spherical nodes is available which is based on these principles.

MERO normal node 1 is derived from the cubic-hexagonal grid. The 18 bores correspond to the directions of the grid module edges and to the directions of the face diagonals.

MERO normal node 2 is derived from the cubic-hexagonal space-centered grid. The 14 bores correspond to the directions of the grid module edges and to the directions of the three –dimensional space diagonals.

These two normal joints are generally employed for buildings and used repeatedly in various configurations, primarily for pedestrian bridges, towers, masts and all special structures derived there from.

The MERO type-node only has those bores necessary for a specific space frame structure at the corresponding position, e.g. 10 bores for a type-node $\frac{1}{2} O + T$ (4 for chords + 4 for diagonals +2 for upright support and suspension). The table provides a list of the possible type-node ranges and their diameters.

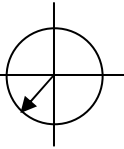























































The MERO special-node is entitled to freely selectable angular geometry and a freely selectable number of bores. However, the angle between axes of two bore holes should not be less than 35°. This node permits the construction of non-plane space frames and other special buildings [3].

The range of nodes listed in the table follows the criteria of logical sequences of diameters, geometric dependence of the connecting bores and of the residual material necessary for transmitting the forces. Per node size there are various bore diameters and connection bolts; and per bolt there are various material qualities from which result the critical connection forces "Permissible tension". The characteristic data of the

range of nodes designate;

1. the diameter ' $2r$ ' of the blank node;
2. The face dimension ' f ' as a rated node dimension (which is added with the rated member length to the axis length);
3. The possible connection bores;
4. The material qualities of the connection bolts.

Table 3.1 Components of Space Frames Nodes [3].

	Nodes: Interdependence of node diameter, connection tread diameter and bearing Capacity											
	M12	M12	M20	M20	M20	M27	M33	M42	M48	M56	M64	Connection Thread
	8,8	10,9	5,6	8,8	10,9	10,9	10,9	10,9	10,9	10,9	10,9	Bolt quality
2r/2f	27,3	35,5	43,4	92,5	120,5	217,5	343,5	560,0	735,0	1015,0	1413,0	Permissible force, kN
49,5/46												Symbol of: Type node  Special node 
60/54												
85/78												
110/98												
132/118												
155/138												
180/158												
200/178												
220/198												
240/218												
260/238												
285/258												
310/288												
350/328												

3.5.3 Design Example

3.5.3.1 General Description

Space frame Geometry is shown in Figure 3.2.

- i. Site – Addis Ababa
- ii. Approach – Limit state Design
- iii. Material Used
 - Structural steel
 - Characteristics yield strength $F_{yk} = 250 \text{ Mpa}$
 - Particle safety factor 1.1
 - Design strength $F_{yd} = 227.27 \text{ Mpa}$
- iv. Design Aids
 - EBCS-1 Basis of Design and Actions on Structures, (1995)
 - EBCS-3 Design of Steel Structures, (1995)

Space frame grid, consisting of semi-octahedron and tetrahedron in parallel edge arrangement is assumed as the roof of workshop for the design example (see Fig. 3.3) Workshop is 10m in height and covers an area of 144m².

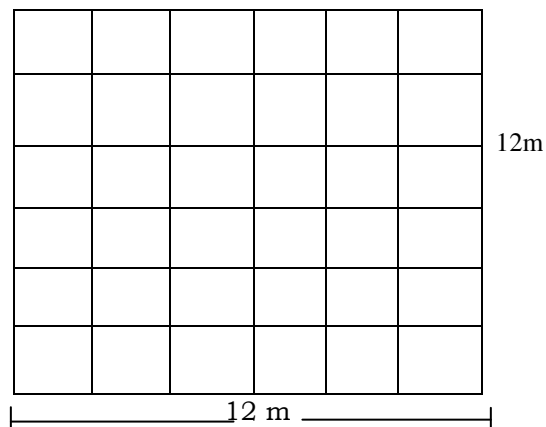


Figure 3.2 Space frame Geometry

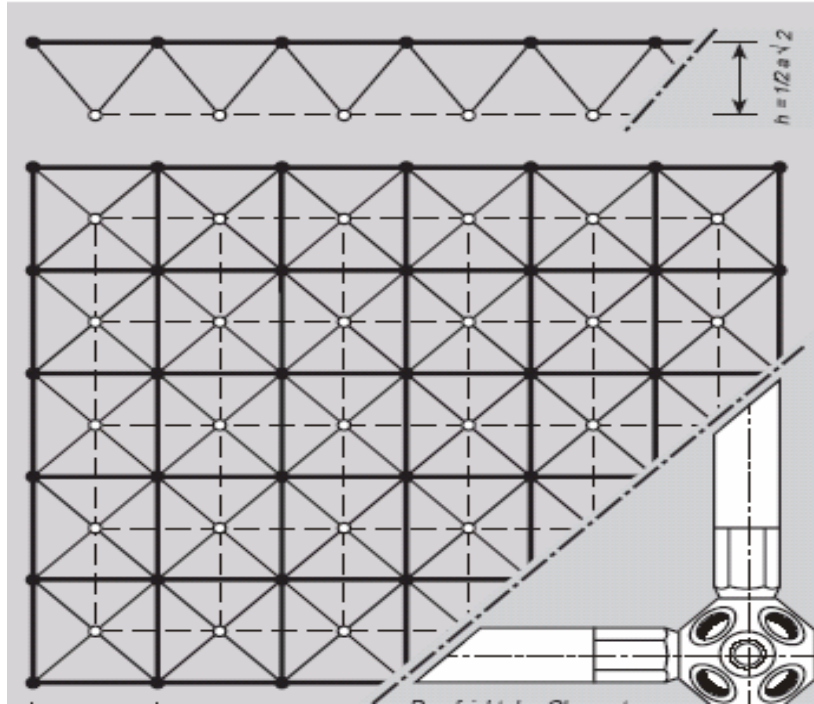


Figure 3.3 Top view of top chord nodes

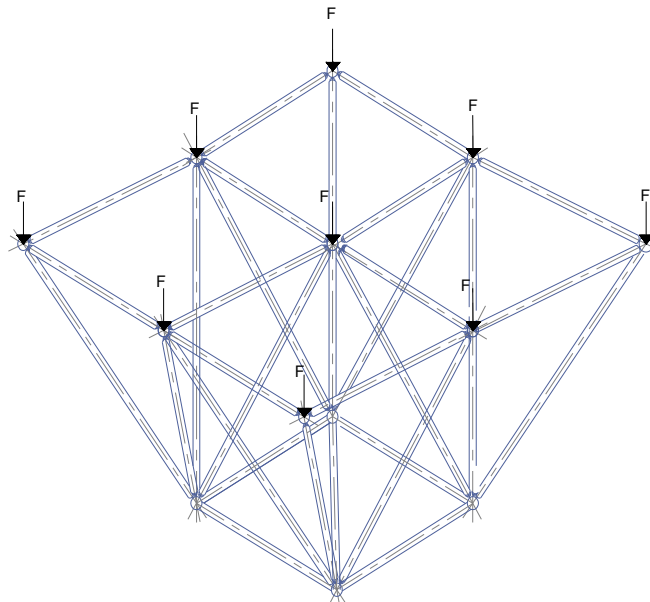


Figure 3.4 Loading of space truss

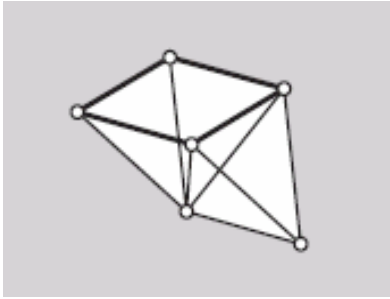


Figure 3.5 Semi-octahedron and tetrahedron arrangement

Number of Top node = 49

Number of Bottom node = 36

Module, a , = $2m$

System Height, $h = a\sqrt{2}/2 = 2m\sqrt{2}/2 = 1.414m$

3.5.3.2 Supports

The space frame sets on columns supported at bottom chord nodes. Supports nodes and the restraints are as follows.

<u>NODES</u>	<u>RESTRAINTS</u>
1 - - - - -	x, y, z
6 - - - - -	z
31 - - - - -	z
36 - - - - -	x, y, z

3.5.3.3 Loadings

3.5.3.3.1 Primary loadings

3.5.3.3.1.1 Dead Loads

- The weight of space frame, purlin, roof covering and various accessories comprise the total dead load.

3.5.3.3.1.2 Roof live loads

- Roofs not accessible except for normal maintenance, repair, painting and minor repairs.

$$\text{Live load} = q_k = 0.5 \text{ kN/m}^2 = 50\text{Kg/m}^2$$

$$Q_k = 1\text{kN} = 100\text{kg}$$

3.5.3.3.2 Secondary Loading

3.5.3.3.2.1 Wind Loading

- Wind pressure on surface

- External pressure, W_e , is the wind pressure acting on the external surface of a structure. W_e shall be obtained from

$$W_e = q_{ref} C_e(Z_e) C_{pe}$$

- Internal pressure, W_i , is the wind pressure acting on the internal surface of a structure. W_i shall be obtain from

$$W_i = q_{ref} C_e(Z_e) C_{pi}$$

Where:- C_{pe} = external pressure coefficient
 C_{pi} = internal pressure coefficient
 q_{ref} = reference mean wind velocity

Pressure q_{ref} shall be determined as:-

$$q_{ref} = \rho / 2 V_{ref}^2$$

V_{ref} is the reference wind velocity and ρ is the air density.

Roof angle (θ) = 0°

Height of the building = 10m

$H = 10\text{m}$, $B = 12\text{m}$

Therefore, consider the building as one part as specified in EBCS 1, 1995 Appendix A22.

$$Z_e = h = 10\text{m}$$

Basic value of the reference wind velocity as specified in EBCS 1, 1995 clause 3-7-2

$$V_{ref} = C_{DIR} C_{TEM} C_{ALT} V_{ref, 0}$$

Where:- $V_{ref, 0}$ - is the basic value of the reference wind velocity to be taken as 22m/sec

C_{DIR} - is the direction factor to be taken as 1.0

C_{TEM} - is the temporary (seasonal) factor to be taken as 1.0

C_{ALT} - is the altitude factor to be taken as 1.0.

Therefore, $V_{ref} = 1 \times 1 \times 22 \text{ m/s}$

$$V_{ref} = 22 \text{ m/s}$$

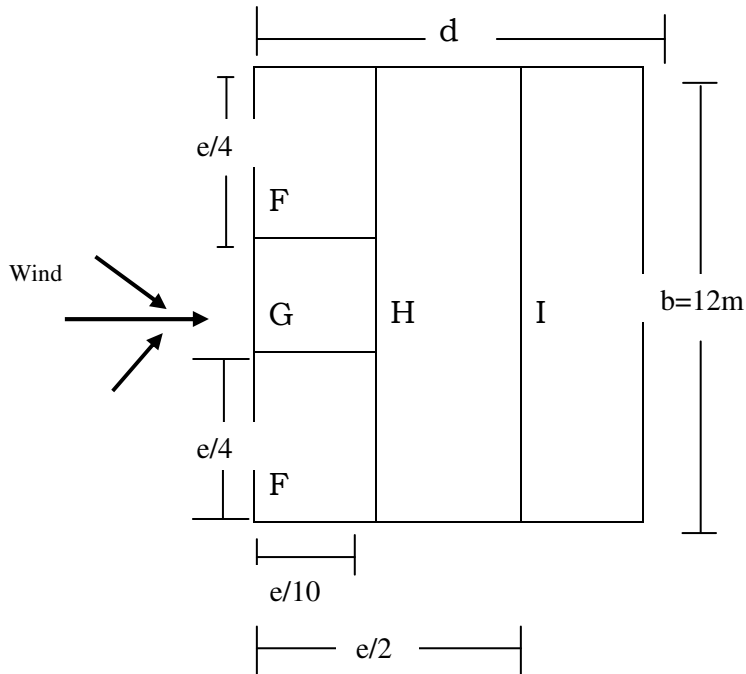
Take $\rho = 0.94$ for Addis Ababa

$$\text{Therefore, } q_{ref} = \frac{0.94 \text{ kg/m}^3}{2} \times 22^2 \text{ m}^2/\text{s}^2 = 227.48 \frac{\text{Kg}}{\text{ms}^2} = \frac{227.48 \text{N}}{\text{m}^2}$$

External pressure coefficients for flat roofs:-

Zone			
F	G	H	I
$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$	$C_{pe,10}$
-1.8	-1.2	-0.7	± 0.2

Table 3.2 :- Sharp eaves



Reference height, $Z_e = h$
 $e = b$ or $2h$, whichever is smaller

= Since all areas $A \geq 10 \text{ m}^2$ use $C_{pe, 10}$



Figure 3.6 Key for flat roofs

Internal pressure coefficients for flat roofs:-

- For closed buildings with internal partitions and opening windows the extreme values.

$$C_{pi} = 0.8 \text{ or } C_{pi} = -0.5$$

Case I

$$C_{pi} = 0.8$$

$$\text{Max } (C_{pe} - C_{pi}) = (-1.8 - 0.8) = -2.6$$

$$C_{pi} = -0.5$$

$$\text{Max } (C_{pe} - C_{pi}) = (0.2 - (-0.5)) = 0.7$$

$$W_e (\text{max}) = q_{ref} C_e (Z_e) C_{pe} = 227.1 \text{ N/m}^2$$

Case II

$$C_{pi} = 0.8$$

$$\text{Max } (C_{pe} - C_{pi}) = (-0.7 - 0.8) = -1.5$$

$$C_{pi} = -0.5$$

$$\text{Max } (C_{pe} - C_{pi}) = (0.2 - (-0.5)) = 0.7$$

Wind load on roof sheeting

For design of sheeting the maximum pressure coefficient is taken from zone F.

$$W = (C_{pe} - C_{pi}) (q_{ref}) (C_e (Z_e))$$

$$W = (-1.8 - 0.8) (227.48) (C_e (Z_e))$$

$$W = (-591.448) (C_e (Z_e))$$

Exposure coefficient, $C_e (Z_e)$, as specified in EBCS 1, 1995 Art 3.8.5

$$C_e (Z_e) = C_r^2 (Z) C_t^2 (Z) \left[1 + \frac{7 K_T}{C_r (Z) C_t (Z)} \right]$$

Where:- K_T is the terrain factor

$C_r (Z)$ = roughness coefficient

$C_t (Z)$ = topography coefficient

Terrain category Iv

$$K_T = 0.24$$

$$Z_o (\text{m}) = 1$$

$$Z_{min} (\text{m}) = 16$$

$C_r (Z) = 0.67$ as specified in EBCS 1, 1995, Table 3.3

$C_t = 1$ as specified in EBCS 1, 1995 Art 3.8.4

$$C_e (Z) = 0.67^2 \cdot 1^2 \left[1 + \frac{7 (0.24)}{0.67 \times 1} \right]$$

$$C_e (Z) = 1.5745$$

$$W = (-591.448) (1.5745) = 931.23 \text{ N/m}^2$$

$$W = \underline{-0.931 \text{ kN/m}^2}$$

Wind load (inward)

$$(C_{pe} - C_{pi}) = 0.7$$

$$W = (0.7) (227.48) (1.5745) = 250.717 \text{ kN/m}^2$$

$$W = 0.251 \text{ kN/m}^2$$

Roof Design

Design of EGA

Truss inclination is equal to $\cong 0^\circ$

Taking EGA 500 thickness = 0.4mm, yield strength $f_{yd} = 227.27 \text{ N/mm}^2$ from KASI standard.

EGA load = 3.14 kg/m^2 , Span = 0.782m

Dead load = $(3.14 \text{ kg/m}^2) (0.782\text{m}) = 2.455 \text{ kg/m} = 0.025 \text{ kN/m}$

Assuming purlin spacing = 2m

Category of roofs as specified in EBCS 1, table 2.31

H= Roofs not accessible except for normal maintenance, repair, painting and minor repairs.

Values of Action as specified in EBCS 1, article 2.6.3.4.2

1. The characteristic value Q_k and q_k given in table 2.14 of EBCS 1 are taken to the projected under consideration.

Roof Category	q_k (kN/m ²)	Q_k (kN)
Flat roof	0.5	1.0

Table 3.3 Imposed loads on roofs

$$\text{Live load concentrated (LL}_c\text{)} = 1 \text{ kN}$$

$$\text{Live load uniformly distributed (LL}_u\text{)} = 0.5 \text{ kN/m}^2$$

Load combination

Assume purlin spacing of 2m.

Case 1: The design moment, M_d , by considering the concentrated live load of 1kN,

$$\begin{aligned}
 M_d &= 1.3 DL + 1.6 LLc \\
 &= 1.3 \frac{WL^2}{8} + 1.6 \frac{Pl}{4} \\
 &= \frac{(1.3 \cdot 0.025 \text{ kN/m}) (2^2 \text{ m}^2)}{8} + \frac{(1.6) (1 \text{ kN}) (2)}{4} \\
 M_d &= 816250 \text{ N mm}
 \end{aligned}$$

Case 2: The design moment by considering the uniform of 0.5 kN/m²

$$\begin{aligned}
 M_d &= 1.3 DL + 1.6 LL \\
 &= 1.3 \frac{WL^2}{8} + 1.6 \frac{WL^2}{8} \\
 &= \frac{(1.3 \cdot 0.025)(2^2)}{8} + \frac{(1.6) (0.5) (2^2)}{8} \\
 M_d &= 416250 \text{ N mm}
 \end{aligned}$$

Therefore the concentrated load governs

$$M_d = 816250 \text{ N mm}$$

For 1 m width, section modules

$$S = \frac{M_{max}}{f_{yd}} = \frac{816250}{227.27}$$

$$S = 3592 \text{ mm}^3$$

Section modules of Ega 500 thickness 0.4 is 3756 mm³ which is greater, the calculated $S = 3592 \text{ mm}^3$

Therefore, $3592 < 3756 \Rightarrow \text{ok!}$

Design of purlin

a) Dead load

The dead load considered in design is a combination of a self weight of EGA and self wt of purlin.

b) Live load

Characteristics live loads on roofs where no access is provided to a roof. For roof design a live load of 0.5 kN/m² or 1 kN considered for the design of purlin.

c) Wind load

As calculated above

$$P = -0.931 \text{ kN/m}^2 \text{ or } P = -1.86 \text{ kN/m for 2m purlin spacing}$$

Load combination

Case 1: Dead load + live load (LL_u)

$$DL = \text{Self wt (purlin + EGA)}$$

Assume 50 x50 x 4 mm RHS is used as a purlin

$$DL \text{ of purlin} = 0.0545 \text{ kN/m}$$

$$\text{Section modules, } S_{all}, = 9490 \text{ mm}^3$$

$$DL \text{ of EGA} = 0.0314 \text{ kN/m}$$

$$LL_u = 0.5 \text{ kN/m}^2$$

$$\text{Total DL} = 0.0545 + 0.0314 = 0.0859 \text{ kN/m}$$

$$P_d = 1.3DL + 1.6LL$$

$$= (1.3) (0.0859) + (1.6) (0.5)$$

$$P_d = 0.9117 \text{ kN/m}$$

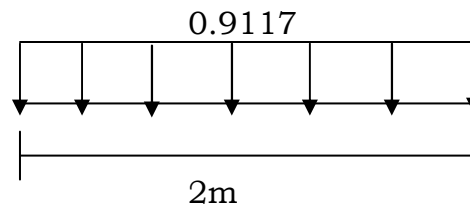


Figure 3.7 Design load on purlin with uniform live load

$$M = \frac{WL^2}{8} = \frac{(0.9117) (2^2)}{8} = 0.4558 \text{ kNm}$$

$$R = \frac{WL}{2} = \frac{(0.9117) (2)}{2} = 0.9117 \text{ kN}$$

Case 2: Dead load + live load (LL_c)

$$DL = \text{Self wt (purlin + EGA)}$$

$$DL = 0.0859 \text{ kN/m}$$

$$P_d = 1.3 DL + 1.6 LL$$

$$P_d = (1.3) (0.0859 \text{ kN/m}) + (1.6) (1 \text{ kN})$$

$$P_d = 0.1117 \text{ kN/m} + 1.6 \text{ kN}$$

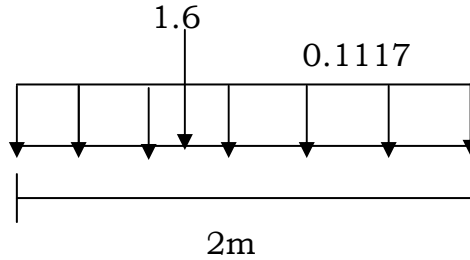


Figure 3.8 Design load on purlin with concentrated live load

$$M = \frac{Wl^3}{8} + \frac{Pl}{4} = \frac{(0.1117)(2^3)}{8} + \frac{(1.6)(2)}{4} = 0.8558 \text{ kNm}$$

$$R = \frac{Wl}{2} + \frac{P}{2} = \frac{(0.1117)(2)}{2} + \frac{1.6}{2} = 0.9117 \text{ kN}$$

Case 3: Dead load + wind load (in ward)

in ward wind load = 0.751 kN/m^2

$$= (0.751)(2) = 1.502 \text{ kN/m}$$

$$P_d = 1.3 DL + 1.6 WL$$

$$P_d = (1.3)(0.0859 \text{ kN/m}) + (1.6)(1.502 \text{ kN/m})$$

$$P_d = 2.5149 \text{ kN/m}$$

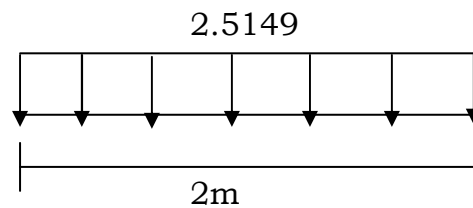


Figure 3.9 Design load on purlin with in ward wind load

$$M = \frac{Wl^2}{8} = 1.2574 \text{ kNm}$$

$$R = \frac{Wl}{2} = 2.5149 \text{ kN}$$

Case 4: Dead load + Wind load (outward)

Out ward wind load = -1.86 kN/m

$$P_d = 1.3 DL + 1.6 WL$$

$$P_d = (1.3)(0.0859) + (1.6)(-1.86)$$

$$P_d = -2.8643$$

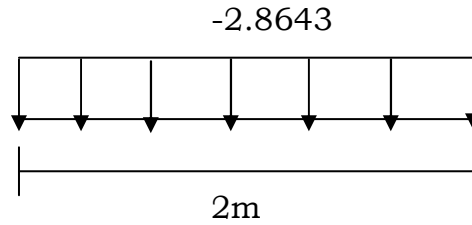


Figure 3.10 Design load on purlin with out ward wind load

$$M = \frac{WL^2}{8} = \frac{(-2.8643)(2^2)}{8} = -1.4321 \text{ kN/m}$$

$$R = \frac{WL}{2} = \frac{(-2.8643)(2)}{2} = -2.8643 \text{ kN}$$

Case 5: Dead load + Σ live load

$$P_d = 1.3 DL + 0.9 (1.6 \text{ distributed live load} + 1.6 \text{ outward wind load})$$

$$P_d = 1.3 (0.0859) + 0.9 (((1.6) (0.5) (2)) - ((1.6) (1.86)))$$

$$P_d = -1.1267 \text{ kN/m}$$

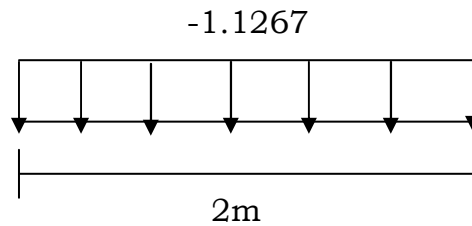


Figure 3.11 Design load on purlin with live load and outward wind load

$$M = \frac{WL^2}{8} = \frac{(-1.1267) (2^2)}{8} = -0.5634 \text{ kN/m}$$

$$R = \frac{WL}{2} = \frac{(-1.1267) (2)}{2} = -1.1267 \text{ kN/m}$$

Case 6: Dead load + Σ live load

$$P_d = 1.3 DL + 0.9 (1.6 \text{ distributed live load} + 1.6 \text{ inward wind load})$$

$$P_d = (1.3) (0.0859) + 0.9 ((1.6) (0.5) (2) + (1.6) (1.502))$$

$$P_d = 3.7146 \text{ kN/m}$$

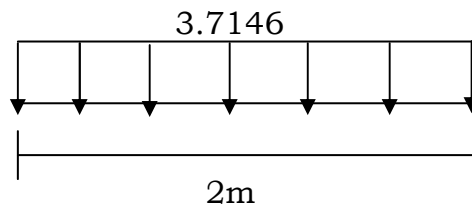


Figure 3.12 Design load on purlin with live load and inward wind load

$$M = \frac{Wl^2}{8} = 1.8573 \text{ kNm}$$

$$R = \frac{Wl}{2} = 3.7146 \text{ kN}$$

Case 7: Dead load + Σ live load

$P_d = 1.3 \text{ DL} + 0.9 (1.6 \text{ outward wind load}) + 0.9 \times 1.6 \text{ concentrate live load}$

$$P_d = (1.3)(0.0859) + (0.9)(1.6)(-1.86) + (0.9)(1.6)(1)$$

$$P_d = -2.5667 \text{ kN/m} + 1.44 \text{ kN}$$

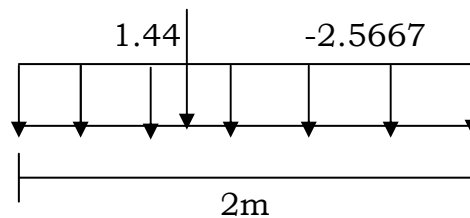


Figure 3.13 Design load on purlin with concentrated live load and outward wind load

$$M = \frac{Wl^2}{8} + \frac{PL}{4} = \frac{(-2.5667)(2^2)}{8} + \frac{(1.44)(2)}{4} = -0.5634 \text{ kNm}$$

$$R = \frac{Wl}{2} + \frac{P}{2} = \frac{(-2.5667)(2)}{2} + \frac{1.44}{2} = -1.8467 \text{ kN}$$

Case 8: Dead load + Σ live load

$P_d = 1.3 \text{ DL} + 0.9 (1.6 \text{ inward wind load}) + 0.9 \times 1.6 \text{ concentrate live load}$

$$P_d = (1.3)(0.0859) + 0.9(1.6)(1.502) + 0.9(1.6)(1)$$

$$P_d = 2.2746 \text{ kN/m} + 1.44 \text{ kN}$$

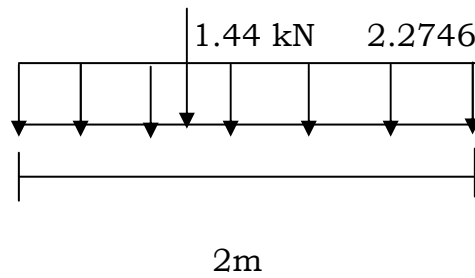


Figure 3.14 Design load on purlin with concentrated live load and inward wind load

$$M = \frac{WL^2}{8} + \frac{PL}{4} = \frac{(2.2746)(2^2)}{8} + \frac{(1.44)(2)}{4} = 1.8573 \text{ kNm}$$

$$R = \frac{WL}{2} + \frac{P}{2} = \frac{(2.2746)(2)}{2} + \frac{1.44}{2} = 2.9946 \text{ kN}$$

To check the capacity of the purlin, the maximum moment is

Case 8: 1.8573 kNm

Check the required section modules

$$S = \frac{M_{\max}}{f_{yd}} = \frac{1857300}{227.27}$$

$$S = 8172.22 \text{ mm}^3 < S_{all} = 9490 \text{ mm}^3 \Rightarrow \text{ok!}$$

Choosing from eight reactions for design, the maximum transferred downward load is

Case 6: $R = 3.7146 \text{ kN}$

For top node \Rightarrow for exterior nodes transferred load is equal to the load transferred from purlin and self weight of the node which is equal to,

$$R = 3.7146 \text{ kN} + 0.01 \text{ kN} = 3.7246 \text{ kN} = \underline{3.72 \text{ kN}}$$

\Rightarrow For interior nodes transferred load is equal to the load twice transferred from purlin and self weight of the node which is equal to,

$$R = 2(3.7146 \text{ kN}) + 0.01 \text{ kN} = 7.4392 \text{ kN} = \underline{7.44 \text{ kN}}$$

For bottom node \Rightarrow for bottom node transferred load is equal to the self weight of node and combined lighting, ventilator and other accessories weight which is equal to,

$$R = 0.007\text{kN} + 0.01\text{kN} = \underline{0.017\text{kN}}$$

Choosing from eight reactions for design, the maximum transferred upward load is

Case 4: $R = -2.8643 \text{ kN}$

$$2R = -5.7286 \text{ kN}$$

For top node \Rightarrow for exterior nodes transferred load is equal to the load transferred from purlin and self weight of the node which is equal to,

$$R = -2.8643 \text{ kN} + 0.01\text{kN} = -2.8543 \text{ kN} = \underline{-2.85\text{kN}}$$

\Rightarrow For interior nodes

$$R = 2(-2.8643\text{kN}) + 0.01\text{kN} = -5.7186\text{kN} = \underline{-5.72\text{kN}}$$

For bottom node \Rightarrow for bottom node transferred load is equal to

$$R = 0.007\text{kN} + 0.01\text{kN} = \underline{0.017\text{kN}}$$

These loads are applied to the space frame and analyzed using SAP 2000 version 9. All design loadings shall be applied to the space frame nodal position no direct load shall be applied to any chord or diagonal member.

4. Preparing a node prototype for local production

Spherical node size assumed sufficient for the majority of the nodes is taken from Table 3.1.

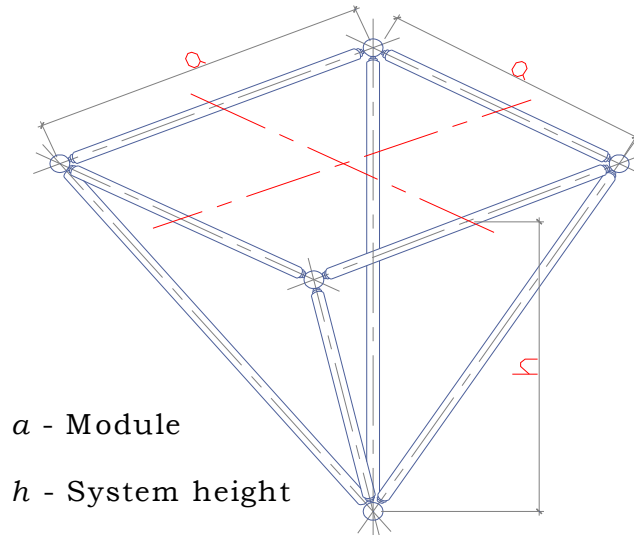


Figure 4.1 Space frame modulation



Figure 4.2 Struts to node connections

4.1 System components of prototype

4.1.1 Truss joints (Nodes)

Design criteria: - Nodes size is determined to allow enough spacing for the pipes without any interference and to carry the forces applied by the connected members.

Size of the node (Table 3.1)

$$2r = 60\text{mm}$$

$$2f = 54\text{mm}$$

Manufacturing techniques: - By hot forging and then machining.

Number of threaded bolt holes: -Eight

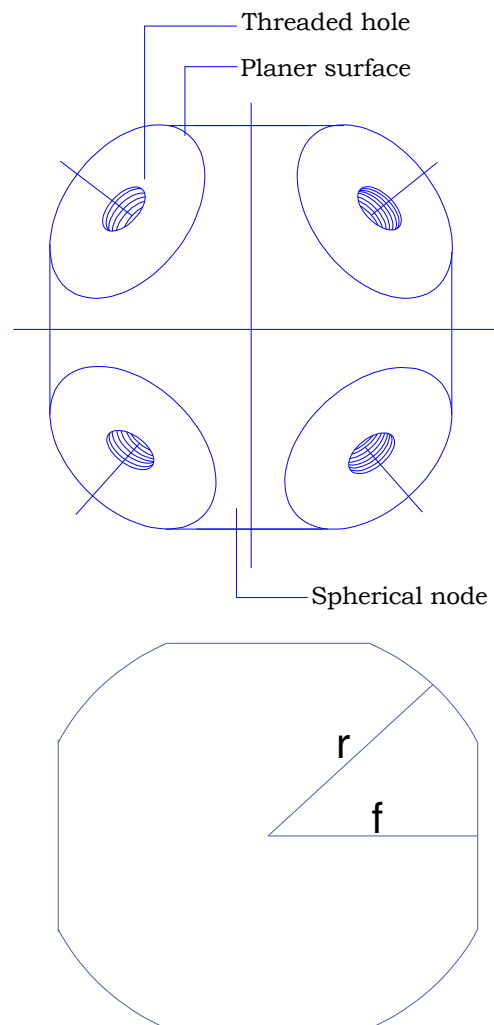


Figure 4.3 Detail of a node



Figure 4.4 Node prototype

4.1.2 Members

- Members consist of circular pipes. These pipes are welded at the ends to conical tips, which have the end bolts. The cones must transmit both compressive and tensile forces as specified by design.

- Dimension of struts

$$D_o = 50\text{mm}$$

$$D_i = 46.5\text{mm}$$

$$t = 3.5\text{mm}$$

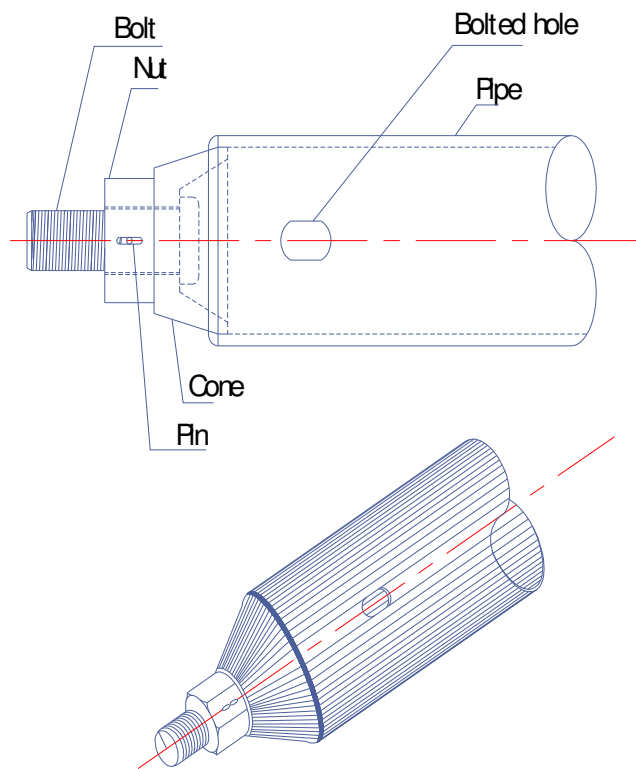


Figure 4.5 Plan and isometric view of a member

4.1.2.1 Conical Tips

Design criteria: - Conical tip is determined to fit inside the pipe to transmit safely the member force.

Manufacturing techniques: - machining forging

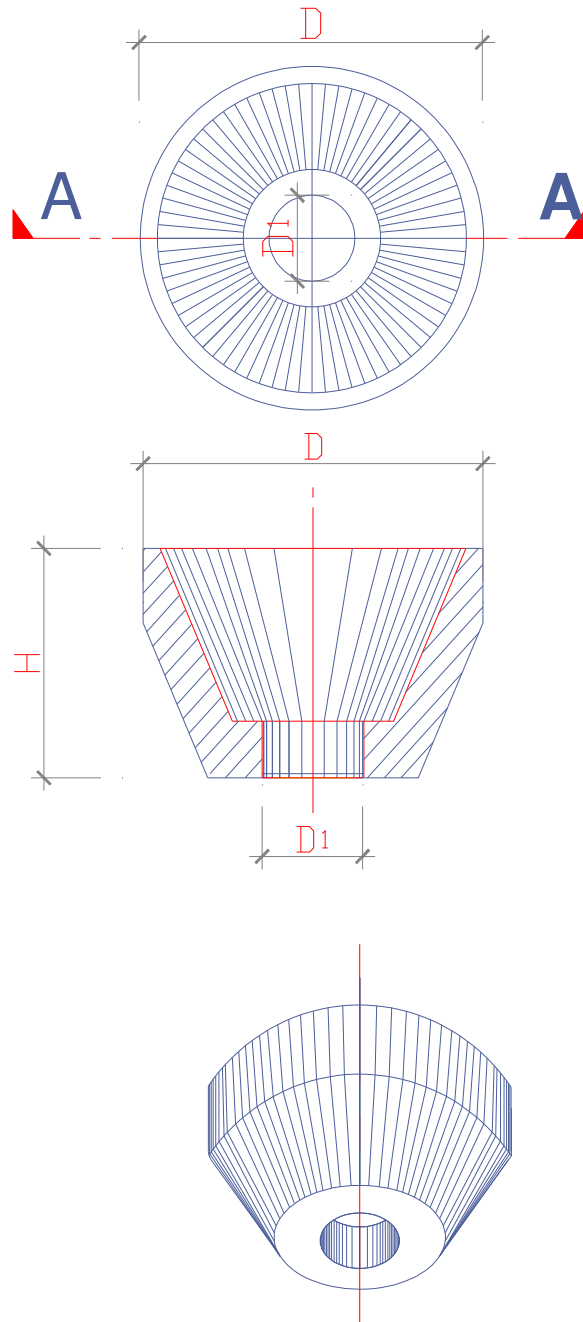


Figure 4.6 Plan, section & isometric view of a cone

4.1.2.2 Nuts

Design criteria: - Nuts are designed mainly for compressive forces. Effective net area of the nut is compared with its bearing area and nodal connection and is designed according to the most unfavorable one of these two.

Manufacturing techniques: - By machining.

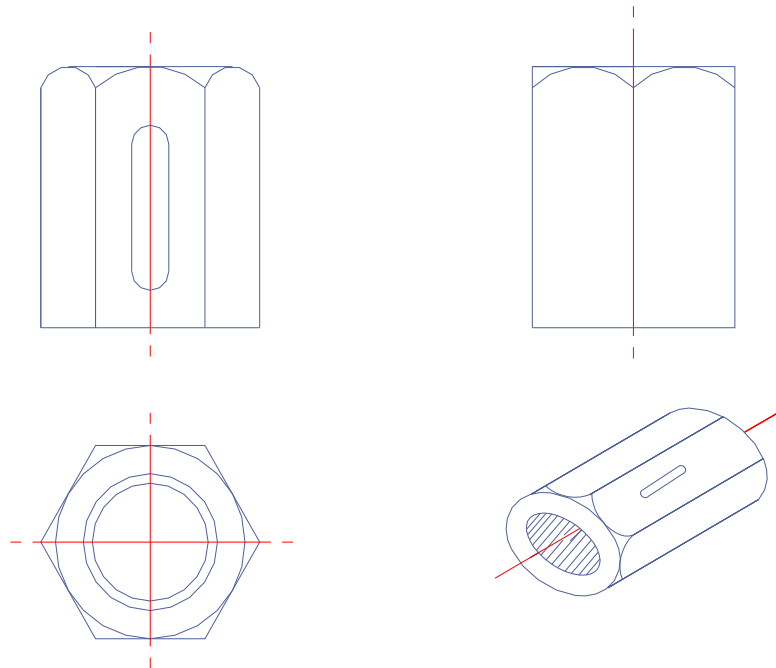


Figure 4.7 Detail of Nuts

4.1.2.3 Bolts

Design criteria: - Bolts are designed mainly for tension forces for its design load carrying capacity; effective net area after reduction of pin hole is considered.

Round head bolts positioned inside the conical tips shall be free to rotate. A hexagonal sleeve (similar appearance to a nut) is attached to each bolt by a connecting pin.

The thickness of bolts used is 12 mm.

Manufacturing techniques: - Threaded by cold rolling then heat-treated.

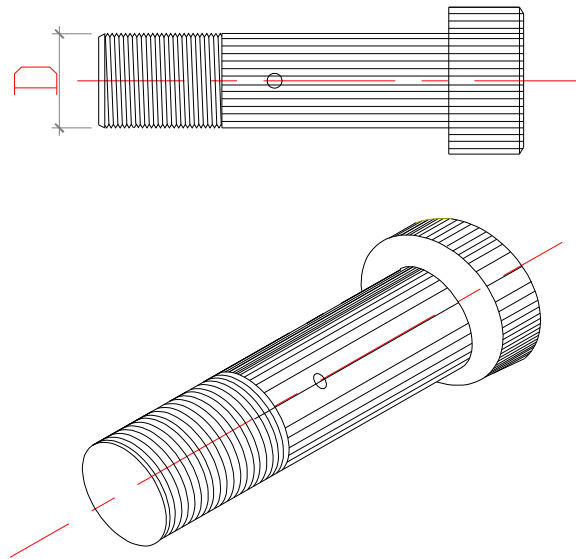


Figure 4.8 Plan and isometric view of a bolt



Figure 4.9 Struts prototype

5 Testing result

Before testing the prototype, mechanical behavior of materials during deformation must be known. Therefore, by taking sample of each System components (Nodes, Members, Conical Tips, Nuts, and Bolts) different tests are performed on the samples by different mechanism.

The degree of deformation to which the material is subjected is defined as strain. For tension or compression, the engineering strain or nominal strain is defined as;

- Strain, e ,
$$e = \frac{l - l_0}{l_0}$$

- Stress, σ ,
$$\sigma = \frac{P}{A}$$

- Tension test is used for members and bolts.
- Hardness test is used for nodes and nuts.

5.1 The tension test

The tension test, because of its relative simplicity, is the most common test for determining the strength–deformation characteristics of materials. It involves the preparation of a test specimen and testing it under tension on any of a variety of available testing machines.

The specimen has an original length l_0 and an original cross-sectional area A_0 . Although most of the specimens are solid and round, flat sheet or tubular specimens are also tested under tension. The original length is the distance between gage marks on the specimen and is generally 2 in (50 mm). Longer lengths may be used for larger specimens such as structural members.

5.2 The hardness test

One of the most common tests for assessment of the mechanical properties of materials is the hardness test. Hardness of a material is generally defined as its resistance to permanent indentation.

Various techniques have been developed to measure the hardness of materials using different indenter materials and geometrics. Among the most common standardized hardness tests is the Brinell test [4].

5.2.1 Brinell Test

In this test, introduced by the Swedish metallurgist J.A Brinell in 1900 [4], a steel or tungsten carbide ball 10 mm diameter is pressed against a surface with a load, p , of 500, 1500 or 3000 Kg[4].

The Brinell hardness number (HB) is defined as the ratio of the local P to the carried area of indentation.

$$HB = \frac{2P}{(\pi D)(D - \sqrt{D^2 - d^2})}$$

Where D is the diameter of the ball and d is the diameter of the impression in millimeters.

Because the indenter (with a finite elastic modulus) also undergoes elastic deformation under the applied load P , hardness measurements may not be as correct as expected. One method of minimizing this effect is to use tungsten carbide balls, which, because of their high modulus of elasticity, deform less than steel balls. Also, since harder work piece materials produce very small impression, a 1500 Kg or 3000 kg load is recommended in order to obtain impression that are sufficiently large for Brinell hardness numbers higher than 500 . In reporting the test results for these high hardnesses the type of ball used should be cited.

The test results of components are as follows.

Table 5.1 Test result of components

<p>Hardness Test</p> <p>1. For Nut</p> <p>$P = 3000 \text{ Kg}$</p> <p>$D = 10 \text{ mm}$</p> <p>$d_1 = 4.23$</p> <p>$d_2 = 4.205$</p> <p>$d_3 = 4.2175$</p>	<p>Hardness Test</p> <p>2. For Nodes</p> <p>$P = 3000 \text{ Kg}$</p> <p>$D = 10 \text{ mm}$</p> <p>$d_1 = 4.69$</p> <p>$d_2 = 4.66$</p> <p>$d_3 = 4.75$</p>
$HB = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d^2})$	$HB = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d^2})$
$HB_1 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_1^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.23^2})$ $= \underline{203.457}$	$HB_1 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_1^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.69^2})$ $= \underline{163.5}$
$HB_2 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_2^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.205^2})$ $= \underline{206.009}$	$HB_2 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_2^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.66^2})$ $= \underline{165.76}$
$HB_3 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_3^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.2175^2})$ $= \underline{204.728}$	$HB_3 = \frac{2P}{(\pi D)} (D - \sqrt{D^2 - d_3^2})$ $= \frac{2 \times 3000 \text{ Kg}}{(\pi 10)} (10 - \sqrt{10^2 - 4.75^2})$ $= \underline{159.13}$
<p>Average of HB</p> $HB = \frac{HB_1 + HB_2 + HB_3}{3}$ $= \frac{203.457 + 206.009 + 204.728}{3}$ $= \underline{204.73} \rightarrow \text{UTS} = \underline{688} \text{ MPa (from Appendix B)}$	<p>Average of HB</p> $HB = \frac{HB_1 + HB_2 + HB_3}{3}$ $= \frac{163.5 + 165.76 + 159.13}{3}$ $= \underline{163.00} \rightarrow \text{UTS} = \underline{541} \text{ MPa (from Appendix B)}$
<p>Tension Test</p> <p>3 For Stuts</p> <p>$F_{ult \text{ max}} = 17.91 \text{ kN}$</p> <p>$F_y \text{ upper} = 17.91 \text{ kN}$</p> <p>$F_y \text{ lower} = 13.58 \text{ kN}$</p> <p>$l_o = 50 \text{ mm}$</p> <p>$l_f = 68.28 \text{ mm}$</p>	<p>Tension Test</p> <p>4 For Bolt</p> <p>$F_{ult \text{ max}} = 17.24 \text{ kN}$</p> <p>$F_y \text{ lower} = 14.66 \text{ kN}$</p> <p>$F_y \text{ upper} = 17.24 \text{ kN}$</p> <p>$d_o = 5.12 \text{ mm}$</p> <p>$l_o = 14.03 \text{ mm}$</p> <p>$l_f = 16.84 \text{ mm}$</p>
$A = W \times t; \quad W = 12.87 \text{ mm}, \quad t = 3.5 \text{ mm}$ $A = \underline{45.045} \text{ mm}^2$	$A = \pi d_o^2 / 4$ $A = \underline{20.589} \text{ mm}^2$
$\sigma_y = F_y / A$ $= 13.74 \text{ kN} / 45.045 \text{ mm}^2$ $= \underline{305.02} \text{ MPa}$	$\sigma_y = F_y / A$ $= 14.66 \text{ kN} / 20.589 \text{ mm}^2$ $= \underline{712.03} \text{ Mpa}$

$\sigma_{ult} = F_{ult}/A$ $= 17.91\text{kN}/45.045 \text{ mm}^2$ $= \underline{397.60 \text{ MPa}}$	$\sigma_{ult} = F_{ult}/A$ $= 17.24 \text{ kN}/20.589 \text{ mm}^2$ $= \underline{837.34 \text{ Mpa}}$
Relative Elongation	Relative Elongation
$d = \frac{l_f - l_o}{l_o} \times 100\%$ $= [68.28 - 50 / 50] \times 100\%$ $= \underline{36.5\%}$	$d = \frac{l_f - l_o}{l_o} \times 100\%$ $= [16.84 - 14.03 / 14.03] \times 100\%$ $= \underline{20.03\%}$

Pictures of the material samples after testing are shown in Figs. 5.1, 5.2, 5.3 and 5.4.



Figure 5.1 Bolt sample after testing



Figure 5.2 Node sample after testing



Figure 5.3 Nut sample being tested and after testing



Figure 5.4 Struts sample being tested and after testing

After knowing the capacities of the system components, the strut, the nut, the bolt and the node are connected to form space frame system prototype as shown in fig.5.5. The system is tested to determine its capacity as a unit (see Fig 5.7). The system is connected in such a way that the tension force is carried by longitudinal struts and transferred to the bolt and then to the node, therefore when the struts is in tension there are three possibilities of failure.

1. Shear failure, (SF), (by bolt/node thread)
2. Tension failure, (TF), (of bolt or strut or node)
3. Weld failure, (WF), (at a connection of nut to struts)

The bolt is M8.8 (ultimate strength 800 MPa and yield strength 640 MPa) and 12mm in diameter. It has on average 4mm hole in it to accommodate

the positioning pin. The node is 60mm in diameter. It has eight threaded bolt holes. The bolt and the node have seven threads each. The calculations for the different possibilities of failure are shown in Table 5.2.

The compression force is carried by the strut, transferred to the nut and to the node. Therefore the possibility failure is two, the nut itself or the node.

The ultimate tensile strength of the node and nut is approximately equal to their ultimate compressive strength, UCS, because of the nature of steel. Approximate compression stress values of the nut and node is taken from the sample hardening test.

Therefore, UCS node = 541 MPa

UCS nut = 688 MPa



Figure 5.5 Connected bolt, nut, node and strut



Figure 5.6 Connected bolt, nut and strut

Table 5.2 Failure calculations

If failure by bolt thread (SF)	If failure by node thread (SF)
$F_{ult} = \sigma_{ult} \times n (t \Pi d)$ $= 837.34 \text{ N/mm}^2 \times 7 \times 1.05 \text{ mm} \times \Pi \times 11 \text{ mm}$ $F_{ult} = \underline{212.68 \text{ kN}}$	$F_{ult} = \sigma_{ult} \times n (t \Pi d)$ $= 541 \text{ N/mm}^2 \times 7 (1.05 \times \Pi \times 12 \text{ mm})$ $F_{ult} = \underline{149.9 \text{ kN}}$
If failure by welding (WF)	If failure by bolt (TF)
$F_{ult} = \sigma_{ult} \times A_{net}$ $A = \Pi d^2 / 4 = \Pi / 4 (d_1^2 - d_o^2)$ $= \Pi / 4 (50^2 - 46.5^2)$ $A = 337.75 \text{ mm}^2$ $E60xx = \text{tensile strength (MPa)} = 427$ $F = 427 \text{ N/mm}^2 \times 337.75 \text{ mm}^2$ $F = 144.22 \text{ kN}$	$F_{ult} = \sigma_{ult} \times A_{net}$ $A_{net} = \Pi d^2 / 4 = \Pi / 4 (d_1^2 - d_o^2) = \Pi / 4 (12^2 - 4^2)$ $= 837.34 \times 100.48$ $F_{ult} = \underline{84.13 \text{ kN}}$
If failure by struts (TF)	
$F_{ult} = \sigma_{ult} \times A_{net}$ $A_{net} = \Pi d^2 / 4 = \Pi / 4 (d_1^2 - d_o^2) = \Pi / 4 (50^2 - 46.5^2)$ $= 397.6 \times 337.75$ $F_{ult} = \underline{134.29 \text{ kN}}$	

Pictures showing samples being tested are shown in Fig. 5.7 and Fig. 5.8 shows samples after test.



Figure 5.7 Samples being tested



Figure 5.8 Samples after testing

Tension test on the space frame prototype as a unit was carried out three times on three prototypes (see Fig.5.7). The tests were used to determine ultimate tensile capacity of the unit. The results of the test are as follows.

1st prototype

- $F_{ult} = 59.7 \text{ kN}$

2nd prototype

- $F_{ult} = 40.1 \text{ kN}$

3rd prototype

- $F_{ult} = 45.3 \text{ kN}$

The average $F_{ult} = 48.37 \text{ kN}$. The average yield strength is about 33.86 KN and F_{yd} is equal to 30.78 KN . When compared to the permissible strength given in table 3.1, this is a better result. The failure occurred as expected on the bolt at the pinhole position for all prototypes which has the lowest tensile strength as was shown in table 5.2.

6. Conclusion and recommendation

Conclusion

Space truss have so many advantages, it's ease of assembly without highly skilled labor, interesting geometric patterns, column-free large architectural spaces and also an open form which allows easy installation of mechanical and electrical services. But this technology is not adapted yet in our country and also such structures are nowadays finding wide applications, therefore manufacturing it locally is more advantageous. In this study design, analysis and manufacturing of space truss prototype made of steel including testing has been carried out using universal testing machine to determine ultimate tensile capacity of the unit.

From the study the following conclusion can be drawn;

- Manufacturing companies in Ethiopia can produce space truss.
- It is also possible to manufacture space truss in Ethiopia using imported bolts or specially manufactured bolts.
- It is possible to manufacture struts and also the nodes in Ethiopia.
- Local expertise can be used to erect the space truss.
- When the demand becomes high and mass production is used the cost will greatly be reduced.

Recommendation

- Local production for standardized sizes of struts and also the nodes should be encouraged.
- More accurate means should be employed by manufacturers to make equal sizes of pin holes on all bolts as it greatly affects the system capacity.
- Care should be taken to make the threaded node holes on the accurate positions to match the angle of the struts joining in the node.

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DECLARATION

This Thesis is my original work and has not been presented for a degree in any university and that all sources of material used in the Thesis have been dually acknowledged

Candidate

Name.....

Signature.....

