A CASE STUDY OF SPACE FRAME MEMBER'S DEFLECTION AT THE ADDIS ABABA INTERNATIONAL AIRPORT

ABNET ABAYNEH

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DECLARATION

I, the undersigned, declare that this thesis is my work and that all sources of materials used for this thesis have been duly acknowledged.

NAME: ABNET ABAYNEH

SIGNATURE: ________________________

PLACE: FACULTY OF TECHNOLOGY, ADDIS ABABA UNIVERSITY

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DEPARTMENT OF CIVIL ENGINEERING

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ABNET ABAYNEH AFFA
SEPTEMBER 2010

Approved by Board of Examiners

Dr. Shifferaw Taye	____________________	____________________
Advisor	Signature	Date
Dr. Asnake Adamu	____________________	____________________
Internal Examiner	Signature	Date
Dr. Bedilu Taye	____________________	____________________
External Examiner	Signature	Date
Ato Abreham Gebre	____________________	____________________
Chairman	Signature	Date

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ABSTRACT

Space frame structures are nowadays widely used in large span and short span structures. The case of assembly and simplicity has made such structures popular particularly in public buildings, including airports. Addis Ababa International Airport Ethiopia Passengers Terminal is one of such building in Ethiopia. Despite their simplicity however, space frame structure have been noted to exhibit failure. Some of the failure of space frame traced back to inaccuracy in assembling, design and material property.

This thesis is concerned with the deflection analysis of Addis Ababa International Airport Ethiopia Passengers Terminal. The method of assembly or erection of AAIAEPT depends on space frame behavior of load transmission and construction details. Scale of the structure, method of jointing, strength and rigidity are the objective function of deflection problems. The module dimension, depth and jointing system have been taken as design variables. Laboratorial research is not performed on the material property of member.

Deflection analysis has been carried out on geometrical and material non-linearity. Factors influence load carrying capacity, data for analysis and confirmation of theoretical values have been calculated. Using this data redesigning has been made. The relationships between temperature and deflection are presented graphically to show the performance of the structure.

The analysis results are found to be in agreement with some of the approximate analysis models except joint rigidity and temperature. The carrying capacity of AAIAEPT space frame is on the safe or conservative side.

Finally, propose possible procedures that will help to stop further deformation and provide recommendation on certain precaution to be taken during space frame design and construction. Provide proposals as curative measures to be taken based on the findings.

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NOTATIONS

D - diameter of steel

θ - the smaller intersecting angle between two bolts

d - diameter of bolt

ξ - ratio between the inserted length of the bolt into the steel ball and the diameter of the bolt

η - ratio between the diameter of the circumscribed circle of the sleeve and the diameter of the bolt

λ – slenderness ratio

I - moment of inertia of member

r - radius of gyration

Ee - effective modulus of elasticity that coincides with Young's modulus in the elastic range

l - length of the member

α - is coefficient of deflection

t - equivalent member thickness

A - cross-sectional area of the member

E - modulus of elasticity

Ef - tangent modulus of elasticity

R - radius of curvature of the framework mid-surface

B - non-dimensional bending stiffness of the grid given in Table 2.5

ABBREVIATION

EAE : Ethiopia Airport Enterprise
AAIAPT : Addis Ababa International Airport Ethiopia Passengers Terminal
IASS : International Association of Steel Structures
CABR : Chinese Academy of Building Research

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1. Introduction

1.1 Background

The search for new structural forms to accommodate large unobstructed areas has always been one of the main objectives of 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. A space frame is a structure system assembled of linear elements so arranged that forces are transferred in a three-dimensional manner. In some cases, the constituent element may be two-dimensional. Macroscopically a space frame often takes the form of a flat or curved surface (Chen, 1999).

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.

The space frame construction system is composed of only two elements: the node and the tube. The node is derived from a cube surface. It has 26 faces, i.e. 8 equilateral triangles and 18 squares (Fig. 1.1). However, for practical and economical reasons the node is actually made from a ball-shaped hollow with rounded off corners, thus giving 18 circular cutting planes. The Tubes of the space frame construction system which have threaded ends on both sides introduce tensile or compression force into the node (Fig. 1.2) (http://www Meroform.org).

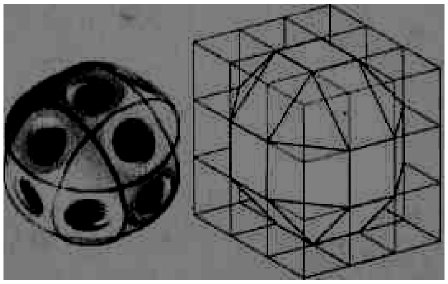


Figure 1.1. Node (http://www Meroform.org).

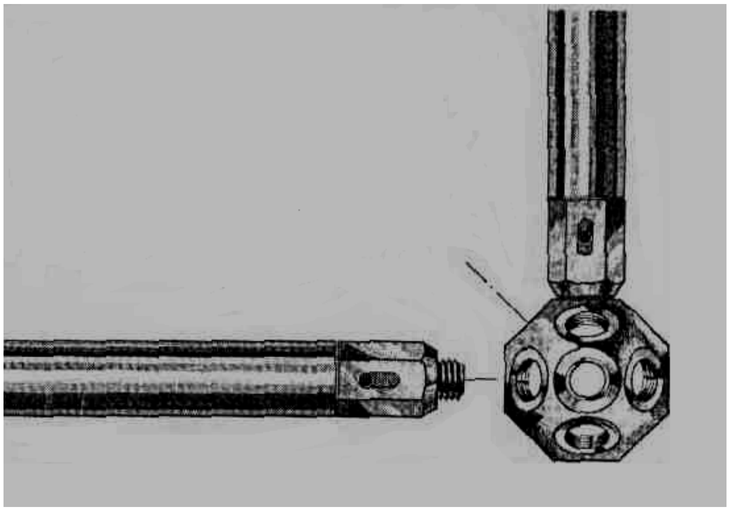


Figure 1.2. Node and Tube (http://www Meroform.org).

A space frame is usually arranged in an array of single, double, or multiple layers of intersecting members (Fig. 1.3 and Fig. 1.4). A single layer space frame that has the form of a curved surface is termed as braced vault, braced dome, or latticed shell (Chen, 1999).

Figure 1.3 Single layer space frame of Ethiopian assembly hall

(Photo by Abnet Abayneh, May 26, 2010, Addis Ababa)

Figure 1.4 Double layer space frame of AAIAEPT

(Photo by Abnet Abayneh, May 26, 2010, Addis Ababa)

Since space frames structures are used in the construction of long span structures like sports arenas, exhibition pavilions, assembly halls, transportation terminals, airplane hangars, workshops, and warehouses they require a significant amount of investments. Nowadays implementation of space frame in Ethiopia is growing rapidly. Addis Ababa International Airport Ethiopia Passengers Terminal, Intercontinental Hotel Building and Ethiopian Assembly hall are some of those. In spite of the increasing wide utilization of space frame members however, some problems have been observed. These include member deflection at Addis Ababa International Airport Ethiopia Passengers Terminal, which exhibit deflection on local and individual members of space frame structure.

This thesis presents the cause of space frame deformation, effects of this deflection on the overall structure and suggests remedial action to stop farther deformation on the Addis Ababa International Airport Ethiopian passenger terminal.

1.2 Objectives

The objective of this thesis is to study the causes of deflection on the roof of the AAIAEPT building, determine its effect on the rest of structural components and finally provides recommendation on certain precaution to be taken during space frame design and construction in general and solutions for the above mentioned problem in particular.

The specific objectives of this work are to: -

- Review available theories, method of analysis and design of space frame structures
- Redesign the space frame structure of the Ethiopian Airports Enterprise passenger's terminal.
- Prepare a graph demonstration of temperature effect on deflection
- Conduct cross-check of original detail drawings, as-built drawing and existing construction.
- Verify the structural performance, supporting system and erection of the frame work.

1.3 Scope of Thesis

Within this thesis literature review had been carried on theoretical backgrounds, advantage of space frame, type of space frame and jointing system. Further, structural analysis and design were performed. In the literature review, basic facts like selection of member component, method of support, method of erection and design parameter are discussed. The capacity of space frame for the applied load at each deflected member is compared with analytical output. The cause of deflection is investigated based on the analysis of deflection.

In the analysis program, redesigning was conducted by applying acting loads on the frame structure with different load cases and load combinations. The effect of temperature on individual members is calculated. The existing terminal structure is compared with the original details and conclusions and recommendations are drawn.

1.4 Thesis Content

The first section provides the introduction, which includes the background, the objectives, scope of the work and what is in this thesis work.

In second section, a comprehensive literature review on the state of the art is discussed. Basic facts are considered for construction of double layer grid frame structures. These are selection of member components, method of support and method of erection, design parameters and jointing system. All this facts are foot print to draw the case study.

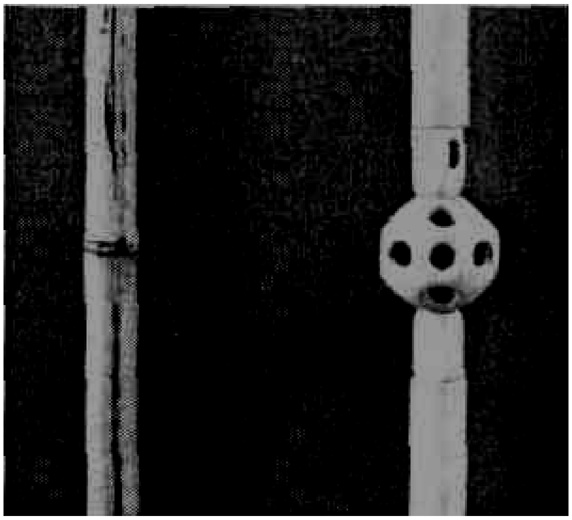
The third section is devoted to the structural and design analysis program of the study. It shows the general geometry analytical output of AAIAEPT space frame and the acting loads on the frame structure are calculated.

The discussion is presented in the fourth section, which is followed by elements that should be considered in the deflection analysis of AAIAEPT. Finally, recommendations and conclusions drawn are presented in the fifth section of this thesis work.

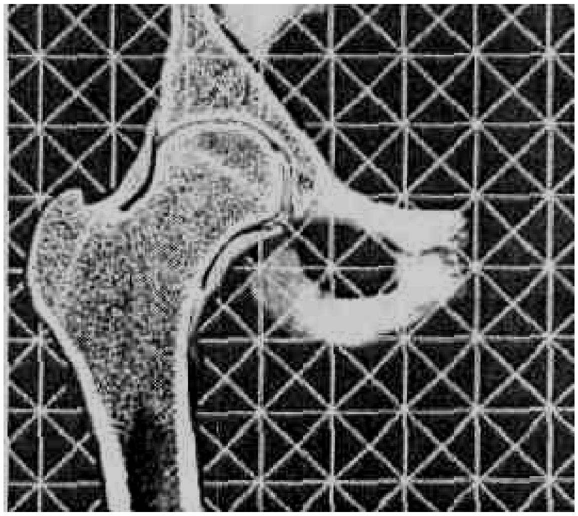
2. Review on the State of the Art

2.1 Space Frame

Nature has demonstrated to us the essential elements of all constructions. Mankind only imitates what has already been created. Nature hardly ever wastes resources and efforts, except where the survival of a species and creation of new is at stake. Economy is one of the basic laws of nature. What matters is the relation between construction material used, the space thus created, and load bearing capacity. This light-construction system, imitating the laws of nature, is the basic principle on which the space frame structures are founded. The stems of many plants, in the shape of tubes with "nodes" to shorten the static length, are typical constructional elements designed to cope with bending stress with alternating direction of force. The internal architecture of a human bone shows, apart from its tubular form, a network of intersecting supports (Fig. 2.1). This leads to the origin of frame works (http://www Meroform.org).



(a) The stems of plant



(b) Human bone format

Figure 2.1 A natural space frame structure (http://www Meroform.org).

In a very broad sense, the definition of the space frame is literally a three-dimensional structure. However, in a more restricted sense, space frame means some type of special structure action in three dimensions. Sometimes structural engineers seem to fail to convey with it what they really want to communicate. It is best to quote a definition given by a Working Group on Spatial Steel Structures of the International Association (IASS, 1984).

> "*A space frame is a structure system assembled of linear elements so arranged that forces are transferred in a three-dimensional manner. In some cases, the constituent element may be two-dimensional. Macroscopically a space frame often takes the form of a flat or curved surface.*"

Also space frame is a structure system in the form of a network of elements (as opposed to a continuous surface). Rolled, extruded or fabricated sections comprise the member elements. Another characteristic of its structural system is that their load-carrying mechanism is three dimensional in nature (ASCE, 1976).

2.2 Advantage of Space Frame

1. One of the most important advantages of a space frame structure is its light weight. It is mainly due to fact that material is distributed spatially in such a way that the load transfer mechanism is primarily axial tension or compression. Consequently, all material in any given element is utilized to its full extent. Furthermore, most space frames are now constructed with steel or aluminum, which decreases considerably their self-weight. This is especially important in the case of long span roofs that led to a number of notable examples of applications.

2. The units of space frames are usually mass produced in the factory so that they can take full advantage of an industrialized system of construction. Space frames can be built from simple prefabricated units, which are often of standard size and shape. Such units can be easily transported and rapidly assembled on site by semi-skilled labor. Consequently, space frames can be built at a lower cost.

3. A space frame is usually sufficiently stiff in spite of its lightness. This is due to its three-dimensional character and to the full participation of its constituent elements. Engineers appreciate the inherent rigidity and great stiffness of space frames and their exceptional ability to resist unsymmetrical or heavy concentrated load. Possessing greater rigidity, the space frames also allow greater flexibility in layout and positioning of columns.

4. Space frames possess a versatility of shape and form and can utilize a standard module to generate various flat space grids, latticed shell, or even free-form shapes. Architects appreciate the visual beauty and the impressive simplicity of lines in space frames. A trend is very noticeable in which the structural members are left exposed as a part of the architectural expression. Desire for openness for both visual impact as well as the ability to accommodate variable space requirements always calls for space frames as the most favorable solution (Makowski, 1992).

2.3 Type of Space Frame

2.3.1 Single Layer Grids

The earliest form of space frame structures is a single layer grid. By adding intermediate grids and including rigid connecting to the joist and girder framing system, the single layer grid is formed. The major characteristic of grid construction is the Omni-directional spreading of the load as opposed to the linear transfer of the load in an ordinary framing system. Since such load transfer is mainly by bending, for larger spans, the bending stiffness is increased most efficiently by going to a double layer system. The load transfer mechanism of curved surface space frame is essentially different from the grid system that is primarily membrane-like action.

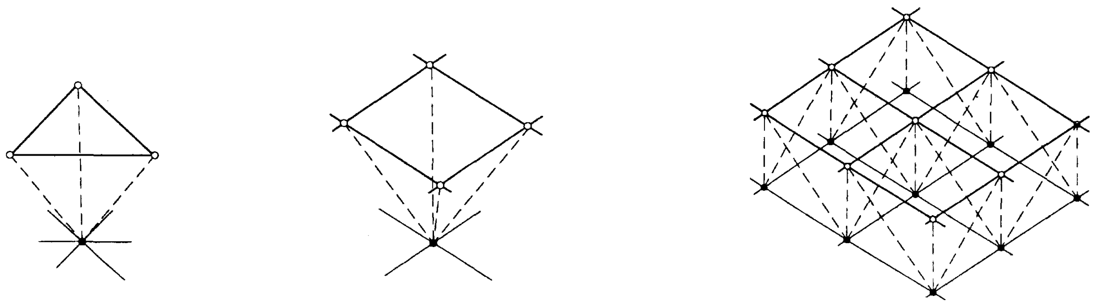
2.3.2 Double Layer Grids

Double layer grids, or flat surface space frames, 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 characterized by the hinged joints with no moment or tensional resistance; therefore, all members can only resist tension or compression. Even in the case of connection by comparatively rigid joints, the influence of bending or tensional moment is insignificant.

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

a) a pyramid with a triangular base (tetrahedron) (Fig. 2.2(a))
b) a pyramid with a square base that is essentially a part of an octahedron (Fig. 2.2(b))
c) a planar latticed truss (Fig. 2.2(c))

These basic elements used for various types of double-layer grids are shown in Figure. 2.2



(a) (b) (c)

Figure 2.2 Basic elements of double layer grids.

Different types of double layer grids are grouped and named according to their composition and the names is indicate in the parenthesis.

Group 1. Composed of latticed trusses

Type 1. Two-way orthogonal latticed grids (square on square) (Fig. 2.3(a))

Type 2. Two-way diagonal latticed grids (Fig. 2.3(b))

Type 3. Three-way latticed (Fig. 2.3(c))

Type 4. One-way latticed grids (Fig. 2.3(d))

Group 2A. Composed of square pyramids

Type 5. Orthogonal square pyramid space grids (square on square offset) (Fig. 2.3(e))

Type 6. Orthogonal square pyramid space grids with openings (square on square offset with internal openings, square on larger square) (Fig. 2.3(f))

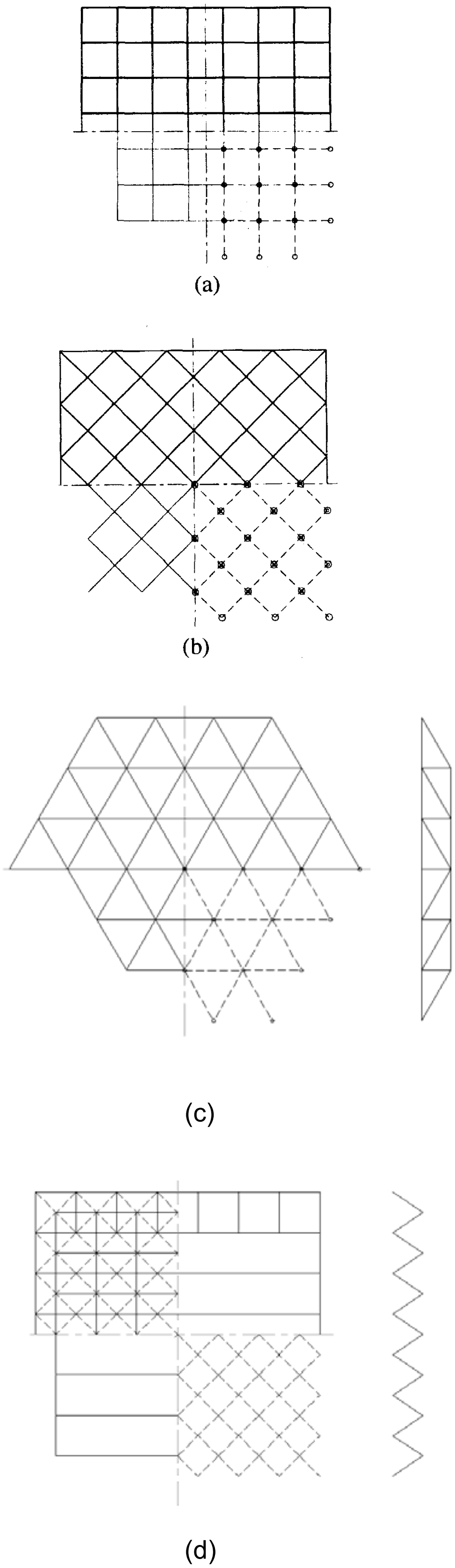
Type 7. Differential square pyramid space grids (square on diagonal) (Fig. 2.3(g))

Type 8. Diagonal square pyramid space grids (diagonal square on square with internal openings, diagonal on square) (Fig. 2.3(h))

Group 2B. Composed of triangular pyramids

Type 9. Triangular pyramid space grids (triangle on triangle offset) (Fig. 2.3(i))

Type 10. Triangular pyramid space grids with openings (triangle on triangle offset with integral openings) (Fig. 2.3(j))



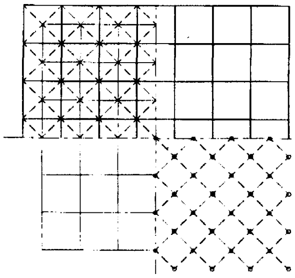
(a)

(b)

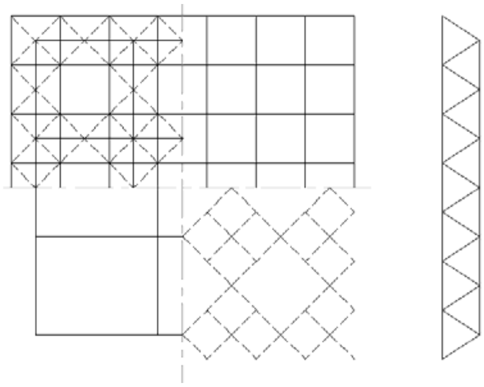
(c)

(d)

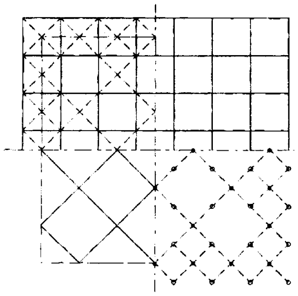
Figure 2.3 Framing systems of double layer grids (Chen, 1999).



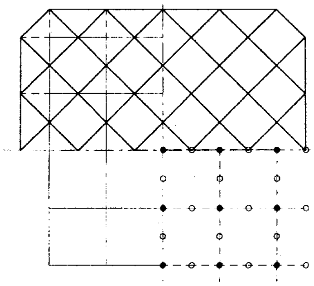
(e)



(f)

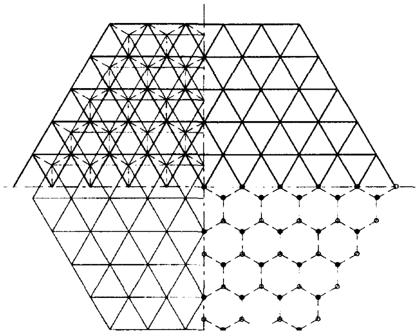


(g)

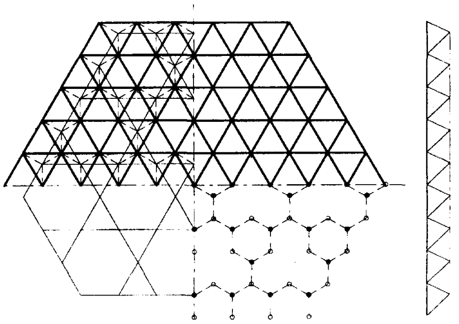


(h)

Figure 2.3 *(Continued)* Framing systems of double layer grids (Chen, 1999) .



(i)



(j)

Figure 2.3 *(Continued)* Framing systems of double layer grids (Chen, 1999).

2.3.2.1 Member Components selection

In the preliminary stage 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 the shape of the building plan, the size of the span, supporting conditions, magnitude of loading, roof construction, and architectural requirements. In general, the system should be chosen so that the space frame is built of relatively long tension members and short compression members (Chen, 1999).

In choosing the type, the steel weight is one of the important factors for comparison. If possible, the cost of the structure should also be taken into account, which is complicated by the different costs of joints and members. By comparing the steel consumption of various types of double layer grids with rectangular plans and supported along perimeters, it was found that the aspect ratio of the plan, defined here as the ratio of a longer span to a shorter span, has more influence than the span of the double layer grids. The recommended types of double layer grids are summarized in Table 2.1 according to the shape of the plan and their supporting conditions (http://www Meroform.org).

Table 2.1 Types Selection for Double Layer Grids

Shape of the plan	Supporting condition	Recommended types
Square, rectangular (aspect ratio = 1.0 to 1.5)	Along perimeters	1, 2, 5,6, 7, 8
Rectangular (aspect ratio = 1.5 to 2.0)	Along perimeters	1,5,6,7
Long strip (aspect ratio > 2.0)	Along perimeters	4
Square, rectangular	Intermediate support	1,5,6
Square, rectangular	Intermediate support combined with support along perimeters	1,2.5.6,8
Circular, triangular, hexagonal, and other odd shapes	Along perimeters	3,9,10

2.3.2.2 Method of Support

Ideal double layer grids would be square, circular, or other polygonal shapes with overhanging and continuous supports along the perimeters. This will approach more of a plate type of design which minimizes the maximum bending moment. However, the configuration of the building yields a great number of varieties and the support of the double layer grids can take the following locations:

Category 1. Support along perimeters - this is the most commonly used support location. The supports of double layer grids may directly rest on the columns or on ring beams connecting the columns or exterior walls. Care should be taken that the module size of grids matches the column spacing.

Category 2. Multi-column supports - for single-span buildings, such as a sports hall, double layer grids can be supported on four intermediate columns as shown in (Fig. 2.4a). For buildings such as workshops, usually multi-span columns in the form of grids as shown in (Fig. 2.4b) are used. Sometimes the column grids are used in combination with supports along perimeters as shown in (Fig 2.4c). Overhangs should be employed where possible in order to provide some amount of stress reversal to reduce the interior chord forces and deflections. For those double layer grids supported on intermediate columns, it is best to design with overhangs, which are taken as 1/4 to 1/3 of the mid-span. Corner supports should be avoided if possible because they cause large forces in the edge chords. If only four supports are to be provided, then it is more desirable to locate them in the middle of the sides rather than at the corners of the building.

Category 3. Support along perimeters on three sides and free on the other side. For buildings of a rectangular shape, it is necessary to have one side open, such as in the case of an airplane hanger or for future extension. Instead of establishing the supporting girder or truss on the free side, triple layer grids can be formed by simply adding another layer of several module widths (Fig. 2.5). For shorter spans, it can also be solved by increasing the depth of the double layer grids. The sectional area of the members along the free side will increase accordingly.

The columns for double layer grids must support gravity loads and possible lateral forces. Typical types of support on multi-columns are shown in Figure 2.6. Usually the member forces around the support will be excessively large, and some means of transferring the loads to columns are necessary. It may carry the space grids down to the column top by an inverted pyramid as shown in Figure 2.6a or by triple layer grids as shown in Figure 2.6b, which can be employed to carry skylights. If necessary, the inverted pyramids may be extended down to the ground level as shown in Figure 2.6c. The spreading out of the concentrated column reaction on the space grids reduces the maximum chord and web member forces adjacent to the column supports and reduces the effective spans. The use of a vertical strut on column tops as shown in Figure 2.6d enables the space grids to be supported on top chords, but the vertical strut and the connecting joint have to be very strong. The use of crosshead beams on column tops as shown in Figure 2.6e produces the same effect as the inverted pyramid, but usually costs more in material and special fabrication (Chen, 1999).

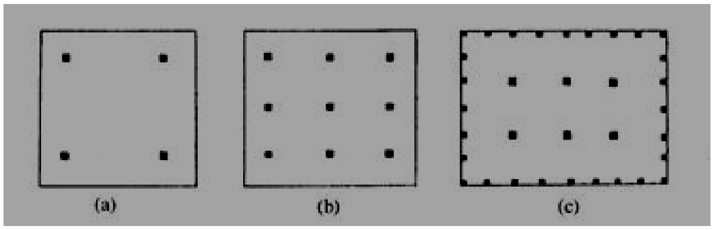


Figure 2.4 Multi-column supports.

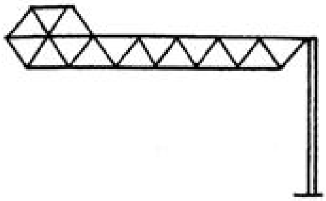


Figure 2.5 Triple layer grids on the free side.

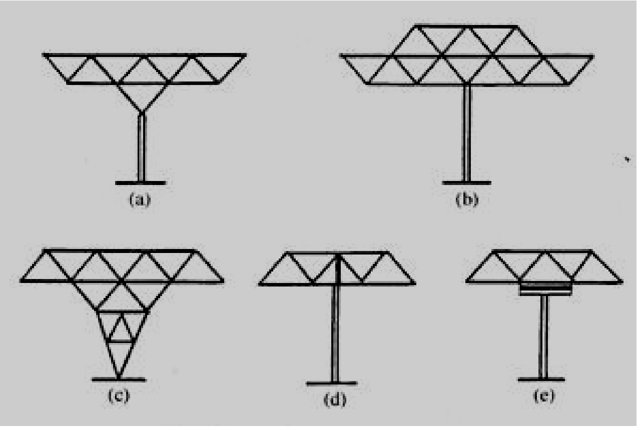


Figure 2.6 Supporting columns.

2.3.2.3 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 (Fig. 2.7). Although these two parameters seem simple enough to determine, they will play an important role on the economy of the roof design. There are many factors influencing these parameters, such as the type of double layer grid, the span between the supports, the roof cladding, and also the proprietary system used. In fact, the depth and module size are mutually dependent which is related by the permissible angle between the center line of web members and the plane of the top and bottom chord members. This is normally taken as less than 30° or the forces in the web members and the length will be relatively excessive, but not greater than 60° or the density of the web members in the grid will become too high. For some of the proprietary systems, the depth and/or module are all standardized.

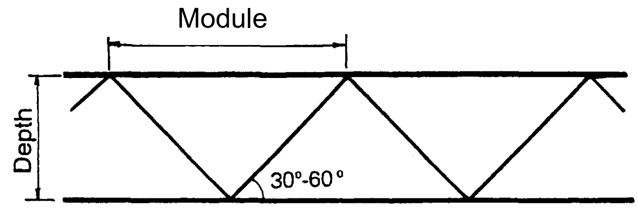


Figure 2.7 Depth and module.

The depth and module size of double layer grids are usually determined from practical experience. For example, the span-depth ratio varies from 12.5 to 25.0, or even more. It is usually considered that the depth of the space frame can be relatively small when compared with more conventional structures. This is generally true because double layer grids produce smaller deflections under load. However, depths that are small in relation to span will tend to use smaller modules and hence a heavier structure will result. In the design, almost unlimited possibilities exist in practice for the choice of geometry. It is best to determine these parameters through structural optimization (Lan, 1994).

2.3.2.4 Methods of Erection

The method chosen for erection of a space frame depends on its behavior of load transmission and constructional details; so that it will meet the overall requirements of quality, safety, speed of construction, and economy. The scale of the structure being built, the method of jointing the individual elements, and the strength and rigidity of the space frame until its form is closed must all be considered. The general methods of erecting double layer grids are as follows. Most of them can also be applied to the construction of latticed shells.

1. Assembly of space frame elements in the air—Members and joints or prefabricated sub-assembly elements are assembled directly on their final position. Full scaffoldings are usually required for such types of erection. Sometimes only partial scaffoldings are used if cantilever erection of a space frame can be executed. The elements are fabricated at the shop and transported to the construction site and no heavy lifting equipment is required. It is suitable for all types of space frame with bolted connections.

2. Erection of space frames by strips or blocks—The space frame is divided on its plane into individual strips or blocks. These units are assembled on the ground level, then hoisted up into the final position and assembled on the temporary supports. With more work being done on the ground, the amount of assembling work at high elevation is reduced. This method is suitable for those double layer grids where the stiffness and load-resisting behavior will not change considerably after dividing into strips or blocks, such as two-way orthogonal latticed grids, orthogonal square pyramid space grids, and those with openings. The size of each unit will depend on the hoisting capacity available.

3. Assembly of space frames by sliding element in the air—Separate strips of space frame are assembled on the roof level by sliding along the rails established on each side of the building. The sliding units may either slide one after another to the final position and then assembled together or assembled successively during the process of sliding. Thus, the erection of a space frame can be carried out simultaneously with the construction work underneath, which leads to savings of construction time and cost of scaffoldings. The sliding technique is relatively simple, requiring no special lifting equipment. It is suitable for orthogonal grid systems where each sliding unit will remain geometrically non-deferrable.

4. Hoisting of whole space frames by derrick masts or cranes—The whole space frame is assembled on the ground level so that most of the assembling work can be done before hoisting. This will result in an increased efficiency and better quality. For short and medium spans, the space frame can be hoisted up by several cranes. For long-span space frames, derrick masts are used as the support and electric winches as the lifting power. The whole space frame can be

translated or rotated in the air and then seated on its final position. This method can be employed to all types of double layer grids.

5. Lifting-up the whole space frame—This method also has the benefit or assembling space frames on the ground level, but the structure cannot move horizontally during lifting. Conventional equipment used is hydraulic jacks or lifting machines for lift-slab construction. An innovative method has been developed by using the center hole hydraulic jacks for slip forming. The space frame is lifted up simultaneously with the slip forms for reinforced concrete columns or walls. This lifting method is suitable for double layer grids supported along perimeters or on multi-point supports.

6. Jacking-up the whole space frame—Heavy hydraulic jacks are established on the position of columns that are used as supports for jacking-up. Occasionally roof claddings, ceilings, and mechanical installations are also completed with the space frame on the ground level. It is appropriate for use in space frames with multi-point supports, the number of which is usually limited (CABR, 1981).

2.4 Jointing System

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. It is also affected by 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.

In designing the jointing system, the following requirements should be considered. The joints must be strong and stiff, simple structurally and mechanically, and yet easy to fabricate without recourse to more advanced technology. The eccentricity at a joint should be kept to a minimum, yet the joint detailing should provide for the necessary tolerances that may be required during the construction. Finally, 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.

A survey around the world reveal that there are over 250 different types of jointing systems suggested or used in practice, and there are some 50 commercial firms trying to specialize in the manufacture of proprietary jointing systems for space frames (Lan, 1994). All the connection techniques can be divided into three main groups: (1) with a node, (2) without a node, and (3) with prefabricated units. Some of the most successful prefabricated jointing systems are summarized as Mero, Space Deck, Triodetic, Unistrut, Oktaplatte, Unibat, Nodus and Space Truss. In Addis Ababa passenger's terminal Mero jointing system has been used.

2.4.1 Mero Connector

The Mero connector, introduced some 50 years ago by Dr. Mengeringhausen, 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 Figure 2.8. Up to 18 members can be connected at a joint with no eccentricity. The manufacturer can produce modes of different size with diameter ranging from 46.5mm to 350.0mm, the corresponding bolts ranging from Ml2 to M64 with a maximum permissible force of 1413kN. A typical space-module of a Mero system is a square pyramid (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 Mero connector has the advantage 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 do not produce any stresses in the bolts; they are distributed to the node through the hexagonal sleeves. The size of the connecting bolt of compression members based on the diameter calculated from its internal forces may be reduced by 6.0 to 9.0 mm.

The diameter of a steel node may be determined by the following equations (Fig. 2.9).

$$D \geq \sqrt{\left(\frac{d_2}{\sin\theta}+\left(d_1 ctg\theta+2\xi d_1\right)^2+\eta^2 d_1{}^2\right)} \tag{2.1}$$

However, in order to satisfy the requirements of the connecting face of the sleeve, the diameter should be checked by the following equation:

$$D \geq \sqrt{\left(\frac{\eta d_2}{\sin\theta}+\eta d_1 ctg\theta\right)^2+\eta^2 d_1{}^2} \tag{2.2}$$

D = diameter of steel ball (mm)

θ = the smaller intersecting angle between two bolts (rad)

d_1 and d_2 = diameter of bolts (mm)

ξ = ratio between the inserted length of the bolt into the steel ball and the diameter of the bolt

η = ratio between the diameter of the circumscribed circle of the sleeve and the diameter of the bolt

ξ and η may be determined, respectively, by the design tension values or compression strength of bolt. Normally ξ = 1.1 and η = 1.8.

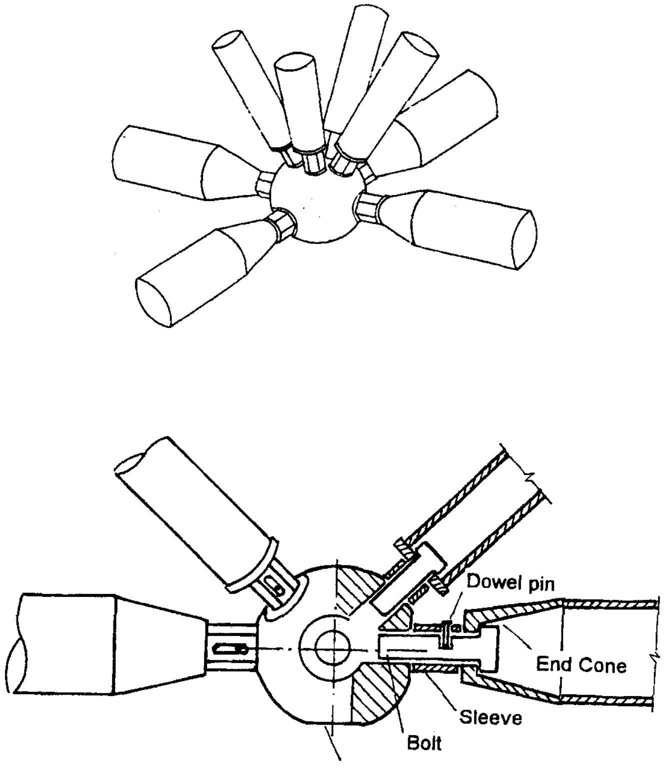


Fig 2.8 Mero system.

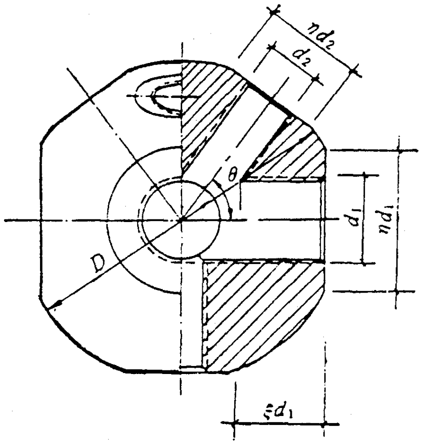


Fig 2.9 Dimensions of spherical node (Lan, 1994).

3. Structural Analysis and Design

3.1 Space Frame Geometry

Fig 3.1 shows the space frame geometry of Addis Ababa International Airport Ethiopia Passenger Terminal. The member and nodes are shown in the subsequent member incidence plots.

3.2 Supports

The space frames are supported on columns that have a lateral flexibility. Upon the acting of external loads, there is a lateral displacement on the top of columns. Under this circumstance the support should be taken as horizontally movable rather than fixed, or as an elastic support by considering the stiffness of the supporting column. AAIAEPT space frame rests on reinforced concrete columns Ø700 supported at top chord nodes. Support nodes and the restraints in the X, Y and Z- direction are shown in Table 3.1.

Table 3.1 Support nodes and restraint

NODES	RESTRAINTS		NODES	RESTRAINTS		
87		Z	1081		Y	Z
90		Z	1084		Y	Z
93		Z	1087		Y	Z
96	X	Z	1090	X	Y	Z
99		Z	1093		Y	Z
102		Z	1096		Y	Z
105		Z	1099		Y	Z
108		Z	1102		Y	Z
498		Z	1253			Z
501		Z	1256			Z
504		Z	1259			Z
507	X	Z	1262	X		Z
510		Z	1265			Z
513		Z	1268			Z
516		Z	1271			Z
519		Z	1274			Z
670		Z	1664			Z
673		Z	1667			Z
676		Z	1670			Z
679	X	Z	1673	X		Z
682		Z	1676			Z
685		Z	1679			Z
688		Z	1682			Z
691		Z	1685			Z

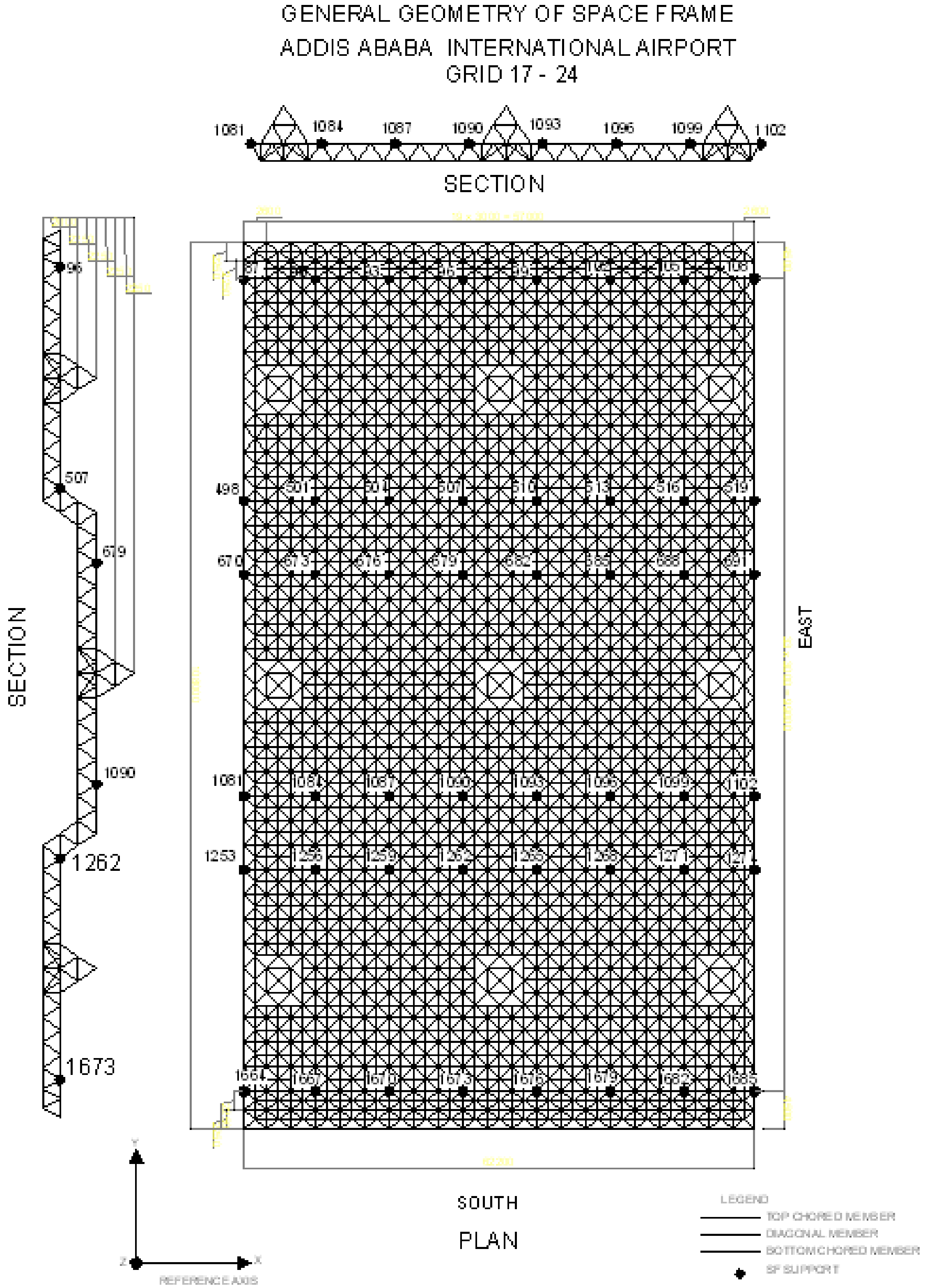


Fig 3.1 General geometry of AAIAEPT space frame

3.3 Loads Acting on the Space Frame Structure

3.3.1 Primary Loadings

1. Dead Loads

The design dead load is established on the basis of the actual loads which may be expected to act on the structure of constant magnitude. The weight of various accessories cladding, supported lighting, heat and ventilation equipment and the weight of the space frame comprise the total dead load. An empirical formula is suggested to estimate the dead weight g of space frame (Chen, 1999).

$$g = 1/200\left(\xi\sqrt{q_w}L\right) \quad KN/m^2 \qquad (4.1)$$

where

q_w = all dead and live loads acting on a double layer grid except its self-weight in kN/m^2

L = shorter span in m

ξ = coefficient, 1.0 for steel tubes, 1.2 for mill sections

- Calculation of applied dead weight of space frame

Space frame & purline: 38kg/m^2

Roof covering: 15kg/m^2

Service load: 20kg/m^2

Live load: 102kg/m^2

Skylights: 175kg/m^2

Supported fans: 250kg/m^2

Weight of curtain wall: 50kg/m^2

Fans: 500kg/unit

hence q_w = 38 + 15 + 20 + 102 + 175 + 250 + 50 + 500= 1150 kg/m^2

= 1150x10 = 11.5 KN/m^2

L = 9.00 m

ξ = 1.00 (1.00 for steel tubes)

g = 0.15 kN/m^2

- Total loads applied in AAIAEPT space frame on the design document is:

Space frame & purline	38kg/m^2
Roof covering	15kg/m^2 (but 30kg/m^2 in skylight regions only)
Service loads	20 kg/m^2
TOTAL	73kg/m^2 = 0.73kN/m^2

Hear g = < 0.73kN/m^2 ok!

The design document shows a dead load of 0.73kN/m^2 and the standard guideline calculated value dead load is 0.15kN/m^2. The load specified in the design documents is a lesser amount than the calculated values. This suggests a conservative design. For further analysis of space frame the recorded value of design dead load 0.73kN/m^2 has been used.

2. Live load

Live load is specified by the local building code and compared with the possible snow or rain load. The larger one should be used as the design load. But rain load is important in a tropical climate especially if the drainage provisions are insufficient. Pounding results when water on a double layer grid flat roof accumulates faster than it runs off, thus causing excessive load on the roof. In view of this the design live load applied on the AAIAEPT structure will be:

Roof Live loads shown in the design document is 102kg/m^2 which is 1.02kN/m^2 where as EBCS-1, 1995 recommend a live load of roof is 0.25kN/m^2.

The live load specified in the design documents is grater amount than the load specified in EBCS-1, 1995. This suggests a conservative design. For further analysis live load 1.02kN/m^2 has been used.

3.3.2 Secondary Loadings

1. Wind Load

The wind load is applied at 1.5kN/m^2 (i.e. q_h) perpendicular towards the exposed surface of whole structure based on its tributary area and design wind pressure, $p = q_h [(G_{cpf}) - (G_{cpi})]$ from table 6-1, page 16 of ANSI/ASCE 7-95. The structure being are enclosed type with mean roof height 'h' less than 60ft. (18m), the external pressure coefficients (G_{cpf}) for loads on main wind – forced resisting system can be taken from figure 6-4, page 22 of ANSI/ASCE 7-95 considering both Case-A and Case-B. Whereas the Internal pressure coefficient (G_{cpi}) can be taken from table 6-4, page 31, which is equal to ± 0.18.

Roof angle (θ) = 0°
$G_{cpi} = \pm 0.18$
Design wind pressure,
$p = q_h[(G_{cpf}) - (\pm G_{cpi})]$ = Net pressure coefficient * q_h

Table 3.2 Wind normal to ridge (i.e. Case-A)

	Net Wall Pressure coefficient		Net Roof Pressure coefficient	
	with - G_{cpi}	with + G_{cpi}	with - G_{cpi}	with + G_{cpi}
Wind Ward side	0.4 - (-0.18) = 0.58	0.4 - (0.18) = 0.22	(-0.69) - (-0.18) = (-0.51)	(-0.69) - (0.18) =(- 0.87)
Leeward side	(-0.29) - (-0.18) = (-0.11)	(-0.29) - (0.18) = (-0.47)	(-0.69) - (-0.18) = (-0.51)	(-0.69) - (0.18) =(- 0.87)

Table 3.3 Wind along the ridge (i.e. Case-A)

	Net Wall Pressure coefficient		Net Roof Pressure coefficient	
	with - G_{cpi}	with + G_{cpi}	with - G_{cpi}	with + G_{cpi}
Wind Ward side	0.4 - (-0.18) = 0.58	0.4 - (0.18) = 0.22	(-0.69) - (-0.18) = (-0.51)	(-0.69) - (0.18) =(- 0.87)
Leeward side	(-0.29) - (-0.18) = (-0.11)	(-0.29) - (0.18) = (-0.47)	(-0.69) - (-0.18) = (-0.51)	(-0.69) - (0.18) =(- 0.87)
Orthogonal sides	(-0.45) - (0.18) = (-0.27)	(-0.45) - (0.18) = (-063)		

Note: Both roof windward and leeward external pressure coefficients have been considered to be same as (-0.69) to give the most unfavorable effect since the roof angle is 0°. Net wind pressure coefficient evaluation in the above tables will be incorporated in the Load Combinations. The wind load calculation of the design document is similar to EBCS-1, 1995 recommendation.

2. Earthquake Loading

Base shear = 0.125 W *(where W = Total Dead Load per m^2 = 73kg/m^2)*

Area of space frame = 62.2x108 = 6717.6 m^2

No. of nodes = 1771

Load / node = 0.125x73x6717.6/1771 = 34.61 kg/node (say 35 kg/node)

Earthquake loading in both directions have been considered in load combinations.

3. Temperature Loading

A temperature difference of ± 30ºC is considered in the design document which is conservative because the National Metrological Agency report indicates with in the past five years the temperature ranges from minimum temperature of 9.0ºC and maximum temperature of 26.6º C. So for the further analysis we can take a temperature of ± 30ºC.

A. Load Case 1 (LC1): Vertical Loads

Dead Load	73 kg/m^2
Live Load	102 kg/m^2
TOTAL	175 kg/m^2

Vertical load is applied on top nodes. The nodal forces are calculated as the product of the this loads with a tributary area.

B. Load Case 2 (LC2):

- Distributed Dead Load on nodes supporting fans (500 kg/unit) has been applied. In addition to this 350 kg for framing and cage has been applied at the same nodes.
- Dead Load of curtain wall assumed to be 50 kg/m^2 and hence the load has been applied to all the nodes supporting the curtain wall as 450 kg per bottom chord node and 225 kg per top chord node.
- Extra Dead Load of skylights, i.e. @ 20 kg/m^2 as the whole roof (including skylights) is already loaded @ 175 kg/m^2

C. Load Case 3 (LC3):

Wind along + X direction.

D. Load Case 4 (LC4):

Wind along - X direction.

E. Load Case 5 (LC5):

Wind along + Y direction.

F. Load Case 6 (LC6):

Wind along - Y direction.

G. Load Case 7 (LC7):

Earthquake load in + X direction.

H. Load Case 8 (LC8):

Earthquake load in + Y direction.

I. Load Case 7 (LC7):

Earthquake load in - X direction.

J. Load Case 8 (LC8):

Earthquake load in - Y direction.

K. Load Case 9 (LC9):

A temperature difference of ± 30º C is considered in the analysis.

3.4 Load Combinations

According to requirement of ANSI/ASCE 7-95, 42 load combinations have been selected as the most unfavorable ones. The combination are based on 8 principal load combinations as

1. D
2. D ± T
3. D + L
4. D + L ± T
5. D + W
6. D + W ± T
7. D + E
8. D + E ± T

where

D	Dead load
L	Live Load
W	Wind Load
T	Temperature

E Earthquake load

Consideration of uplift, i.e. roof pressure on space frame:

Design wind load for roof = Net roof pressure coefficient * q_h

Since the uplift is a vertical loading, the best way to consider it in the analysis is to apply the wind load as a factor of the uniformly distributed gravity vertical loading (i.e. DL + LL) on roof (i.e. LC1). Therefore, for Wind + Dead Load only, coefficient to multiply with LC1,

1. Wind normal to ridge (i.e. Case-A) :
   With – G_{cpi} : ((53) + (-0.51 x 150))/175 = -0.13
   With = G_{cpi} : ((53) + (-0.87 x 150))/175 = -0.44
2. Wind along the ridge (i.e. Case-B) :
   With – G_{cpi} : ((53) + (-0.5 x 150))/175 = -0.13
   With = G_{cpi} : ((53) + (-0.87 x 150))/175 = -0.44

The resulting load cases are therefore enumerated as

Table 3.4 Load Combination

Comb.	Load Combination									Description
No.	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8	LC9	
1	0.42	1.00								D
2	0.42	1.00							1.00	D + T
3	0.42	1.00							-1.00	D - T
4	1.00	1.00								D + L
5	1.00	1.00							1.00	D + L + T
6	1.00	1.00							-1.00	D + L - T
7	-0.13	1.00			0.58	-0.11				D + Wind Normal to Ridge
8	-0.13	1.00			-0.11	0.58				
9	-0.44	1.00			0.22	-0.47				
10	-0.44	1.00			-0.47	0.22				
11	-0.13	1.00	0.58	-0.11	-0.27	-0.27				D + Wind along Ridge
12	-0.13	1.00	-0.11	0.58	-0.27	-0.27				
13	-0.44	1.00	0.22	-0.47	-0.63	-0.63				
14	-0.44	1.00	-0.47	0.22	-0.63	-0.63				
15	0.42	1.00					1.00			D + E
16	0.42	1.00					-1.00			
17	0.42	1.00						1.00		
18	0.42	1.00						-1.00		
19	-0.13	1.00			0.58	-0.11			1.00	D + Wind Normal to Ridge + T
20	-0.13	1.00			-0.11	0.58			1.00	
21	-0.44	1.00			0.22	-0.47			1.00	
22	-0.44	1.00			-0.47	0.22			1.00	
23	-0.13	1.00	0.58	-0.11	-0.27	-0.27			1.00	D + Wind along Ridge + T
24	-0.13	1.00	-0.11	0.58	-0.27	-0.27			1.00	
25	-0.44	1.00	0.22	-0.47	-0.63	-0.63			1.00	
26	-0.44	1.00	-0.47	0.22	-0.63	-0.63			1.00	
27	0.42	1.00					1.00		1.00	D + E + T
28	0.42	1.00					-1.00		1.00	
29	0.42	1.00						1.00	1.00	
30	0.42	1.00						-1.00	1.00	
31	-0.13	1.00			0.58	-0.11			-1.00	D + Wind Normal to Ridge - T
32	-0.13	1.00			-0.11	0.58			-1.00	
33	-0.44	1.00			0.22	-0.47			-1.00	
34	-0.44	1.00			-0.47	0.22			-1.00	
35	-0.13	1.00	0.58	-0.11	-0.27	-0.27			-1.00	D + Wind along Ridge - T
36	-0.13	1.00	-0.11	0.58	-0.27	-0.27			-1.00	
37	-0.44	1.00	0.22	-0.47	-0.63	-0.63			-1.00	
38	-0.44	1.00	-0.47	0.22	-0.63	-0.63			-1.00	
39	0.42	1.00					1.00		-1.00	D + E - T
40	0.42	1.00					-1.00		-1.00	
41	0.42	1.00						1.00	-1.00	
42	0.42	1.00						-1.00	-1.00	

The following table gives the result of the analysis. Resulting include member axial forces, nodal displacements, support reactions and a list of member types. The material used is shown for each member type selected material 20 indicates the yield stress of 205Mpa.

Table 3.5 Result of analysis for maximum forces.

Member Type	Material	Member [mm]	Length [m]	Primery Loading		Secondary Loading	
				Tension [kN]	Compression [kN]	Tension [kN]	Compression [kN]
18	20	73.00X5.16	3	49.672	46.052	60.691	56.631
19	20	73.00X5.16	3	0	74.025	24.348	135.321
20	20	73.00X5.16	3	105.446	0	47.468	52.334
21	20	88.90X5.49	3	0	35.762	20.665	177.019
22	20	88.90X5.49	3	0	92.543	52.779	42.369
23	20	88.90X5.49	3	148.95	0	75.313	83.834
24	20	141.30X6.02	3	0	141.791	73.2	67.149
25	20	141.30X6.02	3	142.611	0	60.923	75.015
26	20	141.30X6.02	3	187.249	0	97.24	102.606
27	20	141.30X6.02	3	213.153	0	105.961	111.911
28	20	141.30X6.65	3	0	212.753	113.823	96.159
29	20	141.30X6.65	3	237.765	261.144	134.19	122.725
30	20	141.30X6.65	3	272.2	0	121.026	130.084

Table 3.6 Result of analysis for maximum and minimum deflection.

Joint to minimum	X- direction	Y- direction	Z-direction
83	-1.0624		
8		-1.94966	
219			-4.6422
Maximum			
100	0.71173		
390		0.054	
8			1.4289

4. Discussion

4.1 General

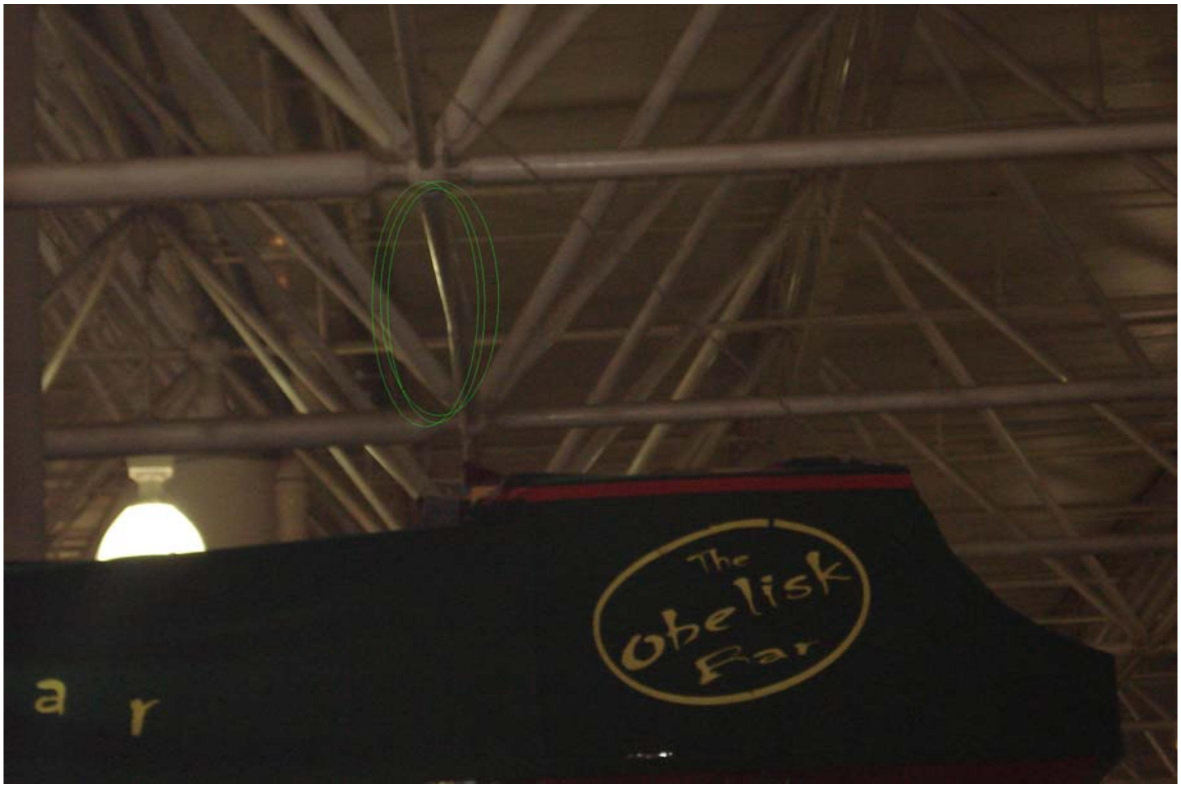
Addis Ababa International Airport Ethiopia Passengers Terminal is approximately 330 m by 221 m with a space frame covering area of approximately 26,000 m^2. The construction took five years and it has been in service for about six and half years. The terminal is the one story building with a mezzanine floor and it has shopping center, restaurants, praying rooms, rest rooms, office and others.

The terminal roof was designed by Rafid Steel Industry. Accordingly fabrication, supply and installation of space frame system were carried out by this company. The roof consisted of orthogonal square pyramid space grids (square on square offset) (Fig. 4.1). This is one of the most commonly used framing patterns with top layer square grids offset over bottom layer grids. In addition to the equal length of both top and bottom chord members, the angle between the diagonal and chord members is 30°. All members in the space grids have the same length composed of horizontal steel bars 2 m apart. The basic element is a square pyramid that is used in some proprietary systems as prefabricated units to form this type of space grid.

Fig 4.1 Orthogonal square pyramid space grids of AAIAEPT.

(Photo by Abnet Abayneh, June 10, 2009, Addis Ababa)

Addis Ababa International Airport Ethiopian Passengers Terminal has a space frame structure composed of a tube and a node, Mero connection. During a site visit we were able to notice deflected members, which are local deflection and member deflection (Fig 4.2).



(a) Member Deflection

(b) Local deflection

Fig 4.2 Deflected members of AAIAEPT

(Photo by Abnet Abayneh, June 10, 2009, Addis Ababa)

4.2 Deflection analysis

Mainly deflection of space frame structure can be caused by design error, construction error and usage. Elements considered in the analysis of AAIAEPT deflection are as follows:

1. Method of Erection

Method of erection is one of the most important aspects that should be taken in to consideration in deflection analysis. The erection of a space frame depends on its behavior of load transmission and constructional details. The scale of the structure being built, the method of jointing the individual elements, and the strength and rigidity of the space frame until its form is closed must all be painstaking. The Terminal space frame structure was erected based on the general methods of erection which is members and joints or prefabricated sub-assembly elements are assembled directly on their final position. Partial scaffoldings are used for cantilever erection of a space. The elements are fabricated at the shop and transported to the construction site and no heavy lifting equipment was required. The erection of this terminal has no contribution in our problem.

2. Load Resisting Capacity

The design dead load of the terminal is numerically 0.73KN/m^2. Considering roof cover, purline, cladding, supported lighting, logo, overhang, heat and ventilation equipment and the weight of the space frame. Live load of the terminal is numerically 1.02KN/m^2. Performing all the above earthquake excitation and wind load is taken in to account. According to the result of redesigning analysis the AAIAEPT has adequate carrying capacity for the specified loads.

3. Geometry

In the investigating of the geometry of the terminal:

I. demonstrating the original detailed drawings to the actual construction no variation has been exhibited
II. There was no misplacement of diagonal members.
III. The tube configuration and member intersection have much more stable and less vulnerable to bending and twisting.

4. Joint Rigidity

Structurally as well mechanically joints must be simple, strong, stiff and easy to fabricate. The eccentricity at a joint should be kept to a minimum, yet the joint detailing should provide for the necessary tolerances that may be required during the construction. The problem is complicated by the effect of geometric nonlinearity of the structure and also the influence of the joint system according to which the members can be considered as pin-connected or partially or completely restrained at the nodes. In general, the behavior of flexible connection is expressed by its moment-rotation curves. Because of complexity, the moment-rotation relation is usually determined by experiments. All frame type space structure should be semi-rigid or flexible. The experimental study has shown that the rigid connections have an effect of local distortions, yielding and deflection, etc. The flexible connection affects significantly the performance of the structures such as deformations, stress distributions and dynamic responses. In order to account for the effects of flexible connections on the behavior of the structures the modeling of the flexible connections is an essential step. But this experiment can not be performed due to the lack of material and laboratory. So it is hard to determine the exact number numerically. In view of the local deflection of the terminal, members are affected neither by the stiffness nor by the loads on the adjacent members but the rigidity of the joints is our first reason for the deflection (**http://www.** jasmirt.org).

5. 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. The choice of support locations perimeter, intermediate columns and types of support fixed, slid or free rotation and translation as well as the geometry of members adjacent to the support, all contribute to minimizing the effect of thermal expansion. A temperature difference of ± 30º C is considered in the analysis this will not result any deflection. The cafeteria which is found on the departure side of the terminal has a steam exhaustion this situation rise the temperature above 30ºC. The rise of temperature above its design value is the second problem that exists (**http://www.** Care.eng.uci.edu). Deflection at the support is proportional to the temperature to illustrate this statement the effect of temperature versus deflection curve is shown below. The behavior of structural steels members subjected to short-time loadings at elevated temperatures is usually determined from short time experimental testes.

Table 4.1 Deflection as a function of temperature

Temperature [^{o}C] Δt	Deflection [m] δ
10	1.28E-03
15	1.92E-03
20	2.56E-03
25	3.20E-03
30	3.85E-03
35	4.49E-03
40	5.13E-03
45	5.78E-03
50	6.42E-03
55	7.06E-03
60	7.70E-03
65	8.34E-03
70	8.98E-03
75	9.63E-03
80	1.01E-02
85	1.09E-02
90	1.16E-02
95	1.22E-02
100	1.28E-02

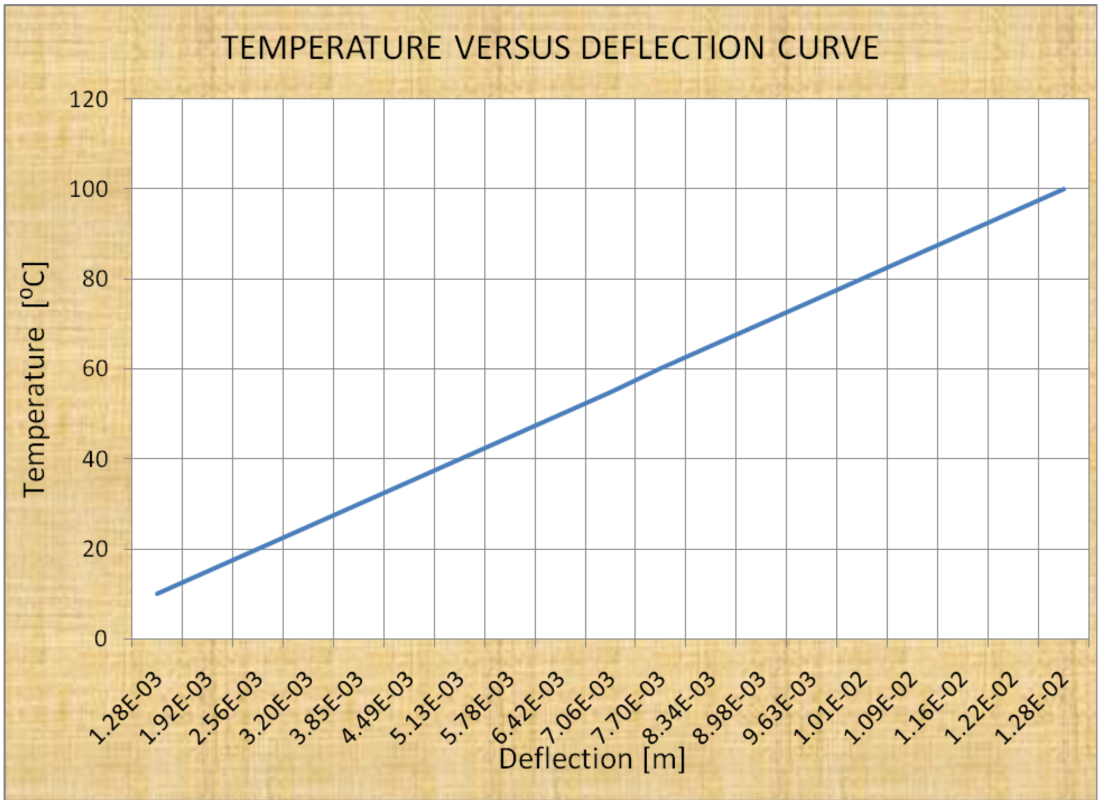


Fig 4.3 Deflection as a function of Temperature

4.3 Stability

1. *Member deflection* occurs when an individual member becomes unstable, while the rest of the space frame (members and nodes) remain unaffected. The design code for steel structures (EBCS-3, 1995) provides methods for estimating member deflection, usually by introducing the slenderness ratio *λ= l/r,* where r is the radius of gyration of the member's section. The deflection load P_{cr} of a straight prismatic bar under axial compression is given by

$$P_{cr} = \alpha \frac{\pi^2 E_e I}{l^2} \qquad \alpha = \alpha\left(c_i, c_j, w_o, e, m\right) \tag{3.1}$$

where

Ee = effective modulus of elasticity that coincides with Young's modulus in the elastic range

I = moment of inertia of member

l = length of the member

α = is coefficient *which* takes different values depending on the parameter in the parentheses

The calculation of P_{cr} :

E_e = 205 Gpa

I = 116 cm^2

l = 3 m

α = 1 (pin ended members) (EBCS-3, 1995)

hence using equation (Eqn. 3.1)

P_{cr} = 26.05KN

The axial compression load from the analysis is 20kN which is less than the calculated critical load so the acting load has no influence for the member deflection.

2. The *local deflection* of a space frame consists of a snap-through deflection which takes place at one joint. Snap-through deflection is characterized by a strong geometrical non-linearity. Local deflection is apt to occur when the ratio of *t/R* (where *t* is the equivalent shell thickness and *R* is the radius of curvature) is small. Local deflection is greatly affected by the stiffness of

and the loads on the adjacent members. Consider the pin-connected structure. Deflection load *c_jc_r* in terms of uniform normal load per unit area can be expressed as

$$\frac{AEI}{12R^3} \leq q_{cr} \leq \frac{AEI}{6R^3} \tag{3.2}$$

Where

A = cross-sectional area of the member

E = modulus of elasticity

R = radius of an equivalent spherical shell

$$Q_{cr} = \frac{E_t}{1+\alpha^2(8\pi^2)}\left(0.47\frac{Al^3}{R^3}+3\frac{BI}{lR}\right) \tag{3.3}$$

$$\alpha = \frac{l^2}{rR}$$

where

E_t = tangent modulus of elasticity

R = radius of curvature of the framework mid-surface

r = radius of gyration

B = non-dimensional bending stiffness of the grid given in Table 4.2 (Lan, 1994)

TABLE 4.2 Equivalent Bending Stiffness *B* (Lan, 1994)

α	1/32	1/16	1/8	1/4	1/2	*1*	*2*	*4*	*8*	*16*	*32*	*64*
B	0.868	0.873	0.886	0.95	1.176	1.85	3.15	4.83	6.48	7.35	7.8	7.9

The calculation of Q_{cr} :

A = 11.2 cm^2

R = 8000 cm

l = 300cm

I = 48.8cm^2

E_t = 205Gpa

r = 2.52 cm

B = 4.83

α = 4.46

hence using equation (Eqn. 3.3)

$$Q_{cr} = 9.36KN$$

The axial compression load from the analysis is 8kN which is less than the calculated critical load so the acting load has no influence for the local deflection.

5. Conclusion and Recommendation

5.1 Conclusion

The thesis has shown that the surrounding temperature increment affect the exposed frame structure. Air Handling Unit should be built within the system. The other option is the exposed frame structure should be either made of temperature resisting material or insulated to resist the steam exhaustion from the surrounding. What's more non-flexibility of the joint has put the terminal suitable for local deflection.

Desire for openness for both visual impact as well as the ability to accommodate variable space requirements, quality, safety, speed of construction, and economy always calls for space frames as the most favorable solution. And also the other important advantages of a space frame structure are its light weight, easy transportation and rapid assembling system on site by semi-skilled labor. Because of this entire reasons space frame in our country should not be limited to what is set up now.

5.2 Recommendation

1. Replace the buckled member with out influencing stability of the neighboring joint and tube of space type frame structure by erecting scaffold to support the gravity load and provide restraints which have equal tension and compression as of effected member.

2. Local deflection often plays the role of trigger for overall deflection through time. Plenty local deflection member existence also could be the reason for overall deflection. These two situations facilitate the collapse of space frame structure.

3. Investigating the deflection its origin is temperature which is resulted from exhausted steam from the restaurant. To avoid this temperature increment the restaurant should be removed if not Air Handling Unit or kitchen hood with ventilation system should be provided.

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Annex A

National Meteorological Agency Report