STUDY ON SECONDARY STRESSES IN TRUSSES DUE TO CONNECTION ECCENTRICITIES

A PROJECT REPORT

Submitted by

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BONAFIDE CERTIFICATE

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Last but not the least; we thank our **PARENTS** for their moral support..

ABSTRACT

Secondary stresses occur in steel truss members due to fixity at the nodes, which are considered, pinned in primary analysis. Secondary stresses may also occur in the members due to member imperfections or due to temperature variations.

When a truss is fabricated with web members directly welded to chord members, secondary stresses may be induced due to eccentricity of member at the node and weld length imbalance at a joint.

If loads are applied over a member, flexural stresses are superimposed over axial stresses, thus the member will be a beam – column. The secondary stresses associated with a beam column need specific study if the members are rigidly connected at the nodes.

The present study aims at studying the secondary stresses occurring in the following problems.

1. Pitched roof truss with directly welded connections with loads at nodes.
2. Pitched roof truss with concentric connections (with gusset plates)
3. Parallel chord lattice trusses with directly welded connections with loads at nodes.
4. Parallel chord lattice trusses with directly welded connections with loads on chords.

The above cases have been analyzed using Finite Element Method using ANSYS. The following inferences have been made.

1. Elimination of gusset plates will result a considerable economy in the total cost of the roof truss. i.e. 5 to 10% of the cost of truss can be omitted.

2. High secondary stresses exist only in some of the members and then only in the extreme fibers at the ends of the members.

3. If web member axis is inclined around 30 to 60^0, the secondary stresses due to bending of web member is only margined 25-30%, well within the yield strength of material.

4. Selection of configuration and truss type is important to minimize the secondary stresses. Pratt or Howe trusses may be avoided, instead sub divided fink truss is preferred.

List of symbols

Ae	:	effective sectional area
Pd	:	Design compressive strength
f_{cd}	:	Design compressive stress
f_y	:	Characteristic yield stress
f_u	:	Characteristic ultimate tensile stress
KL/r	:	Effective slenderness ratio
α	:	Imperfection factor (as per table 7 of IS 800 : 2007)
χ	:	Stress reduction factor (as per table 8 of IS 800 : 2007)
	=	$\frac{1}{(\phi + (\phi^2 - \lambda^2)^{0.5})}$
ϕ	:	Inclination of the tension field stress in web
	=	$0.5(1 - \alpha(\lambda - 0.2) - \lambda$
λ	:	Non dimensional effective slenderness ratio
	=	$\sqrt{f_y / f_{cc}}$
f_{cc}	:	Euler buckling stress = $\frac{\pi^2 E}{(\frac{KL}{r})^2}$
f_{cd}	:	$\chi \frac{f_y}{\gamma_{mo}} \leq \frac{f_y}{\gamma_{mo}}$
γ_{mo}	:	Partial safety factor for material strength
γ_{m1}	:	Partial safety factor against ultimate stress
f_{wn}	:	Nominal strength of fillet weld = $f_u/\sqrt{3}$

Introduction

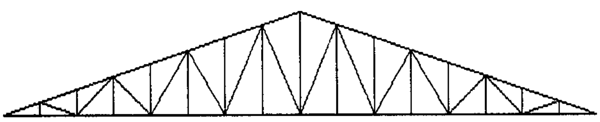
Introduction

1.1 General

For light and normal building roof loads, beams can span up to certain dimensions; beyond that they will be deeper and heavier. Also for large column free spaces for operational purposes, they are unsuitable. Te logical choice for relatively light loads and large spans are the trusses. Trusses are generally symmetrical in shape sine the loads coming on the trusses, except wind, are symmetrical about the centre line. The important principles in the design of a truss, considered as a plane frame are 1). A perfect truss is composed of straight members between node points, assumed to be frictionless joints, and arranged in such z manner that only axial forces occur in the straight members. 2). All the external forces on the trusses are assumed to be coplanar with the truss.

1.2 Truss forms and members

Type of trusses: (a) warren trusses, (b) Pratt trusses, (c) Subdivided Warren truss, (d) k-truss, (e) curved-chord Pratt truss, and (f) Pettit truss.



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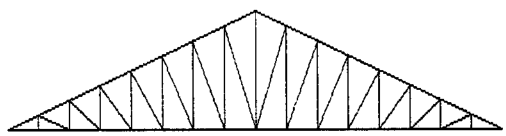

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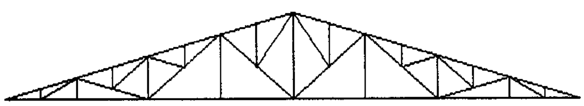

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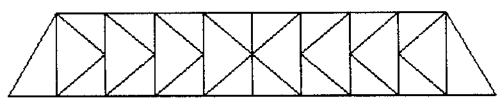

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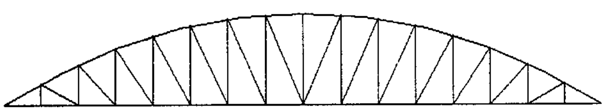

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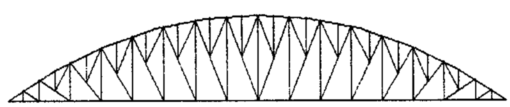


f

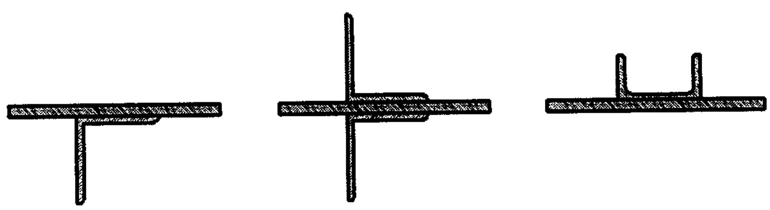
The most common form of truss is the warren truss (fig a). The vertical members carry only the panel loads .The warren truss has relatively High secondary stresses,

Pratt truss is considered more effective inorder to reduce secondary stresses. the economic height-to-span ratio of trusses is about one-sixth To one-nineth, varying with the type of truss, loadings, span length, etc. It Can be further shown that the optimum inclination of the diagonals is About 45^0. Slight variation from these proportions will not noticeably affect the over-all weight but excessive deviations may result in appreciable additional material for the truss. When truss spans are increased in length, Their economical height will also increase. Thus both the Warren and the Pratt trusses will result in along panel lengths if the diagonal inclination Remains about 45^0 . One way to shorten the panel length is to subdivide these trusses. A subdivided warren Truss is shown in fig. These subdivided trusses have the disadvantages of developing high secondary stresses.

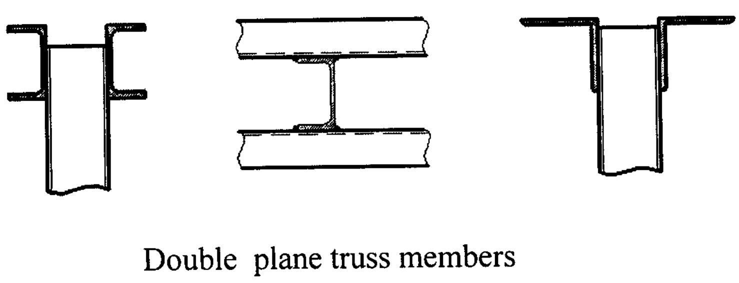
'K' trusses will Keep the desirable diagonals inclination, supply the required depth of trusses, and at the same time limit the panel length. Truss chords may be curved to carry part of the shear and to reduce Stresses in the diagonals (fig e and f). There is a slight increase in the Cost of fabrication compared to a parallel chord truss, but for medium and long spans the additional cost may be more than balanced by the saving in material.

Trusses can be of either the single- or the double-plane type. A single-Plane truss is one that has its gusset plates lying in one plane, that is, there is only one gusset plate at each joint. such connections are suitable for light bracing and trusses with light loads and small members. For most trusses, main members are composed of rolled steel channel (ISMC) and standard angle (ISA) sections. and gussets on two parallel planes will be needed. They are termed as double Plane trusses.

Members of single-plane trusses are either bars or single and double Angles Occasionally, four angles and single or double channels may be used. Double-plane trusses have wide-flange (WF) sections, double channel, or built-up box sections. To facilitate connections, the two planes of gusset plates must remain a constant distance apart; hence WF sections are not as easily connected to a double-plane truss as built-up sections made of angles and plates.



Single plane truss members



Double plane truss members

The shape and size of the members are determined by their stresses as well as their connection requirements. The distance between the main chords or the gussets should remain kept same for a specific purpose.

Bracing members are lighter than the main truss members, but they may need to possess a certain amount of rigidity. Hence it is often desirable to use latticed sections.

1.3 Truss connections

A common method of joining together the member of a truss is by gusset plates at the joints where the members meet. The members are connected to the gusset plates by riveting, bolting, or welding. The thickness of gusset plates is governed by several factors. First, a minimum thickness is necessary to develop the full strength of the bolts. If the bolts on the gussets are in single shear, a smaller bearing thickness is needed to develop their strength; whereas, if the bolts are in double shear, a greater thickness is required. For convenience in design and construction, it is preferable to use only one thickness of gusset plate for a truss. Plates are more commonly employed. It is desirable to check the stresses in gusset plates to determine whether they are within allowable limits. The usual practice in design has been to consider critical sections in the plate using approximate methods. For each Section the nominal stresses are determined by the usual beam theory, using direct load P and bending moment M :

$$f = p/a + mc/i$$

If at the critical section, there is shear in addition to direct load and moment, it is evident that the shearing stresses should also be considered. This would necessitate the determination of the maximum principal stresses.

This method yields stress values which do not accurately represent the actual stresses. Determination of actual stresses in the gusset plates is not possible because of load concentrations, warping of plate section, and local yielding. Compressive stresses along free edges of gusset plates may cause local buckling and stiffeners may be required to prevent this.

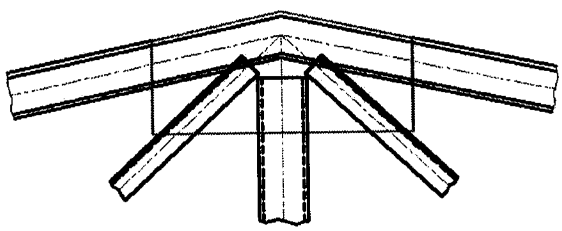
There are distinctly two different types of stress transmission through the gusset at a truss joint. The first occurs where the chord member is continuous through the gusset. Here the main portion of the stress in the chord is transmitted directly within the chord itself; only the difference of the chord stresses is carried through the gusset. This arrangement is often used in a truss in order to relieve the gusset plate of any excessive load. If chord splices are required, they are made outside of the joint in the lesser stressed member.

The second type of stress transmission in gussets occurs where the chord members are spliced right at the joints. The gussets at these joints are subjected to heavy stresses because they transmit the entire amount of the chord stresses. For compression chords, which bear against each other at the joints, the bearing surfaces are always milled so that a greater part of the load is transmitted directly through the member and not through the gusset. Compression members often spliced on all the four sides in order to hold the abutting parts in alignment, these splice plates will help to carry some of the load and gussets may be sufficient to carry the remainder. On the other hand, the gussets at tension splices can not be easily designed without excessive additional material. Hence direct splice for tension chord is generally limited to small spans.

A valuable series of tests on stress distribution in gusset plates was conducted at the University of Tennessee. The tests included only joints with continuous chords. For such gusset plates , it wad found that the usual beam formulas are applicable to certain sections but not to others where high localized stresses exist.

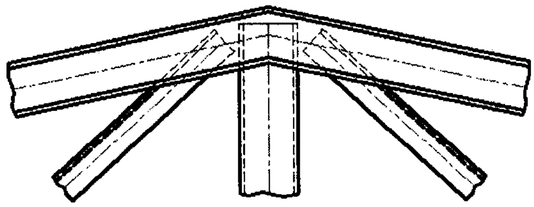
In the design of welded truss, the choice of the member sections is affected by fasteners and gussets. Rolled and built-up sections, which can not be connected conveniently by fasteners and gussets, can some times be easily welded together.

A typical gusset connection is shown in fig, the gusset is welded to the top chord and the web is extended inside the gussets and fillet welded to them. Gusset plates are usually cut with straight edges to reduce costs.



Gusset plate connection

Although directly welded trusses are often used for light roof construction, their application to heavy bridge work has been rather limited. One problem is the existence of local moment and torque at welded connections which may produce high stress concentrations and when repeated could result in failure. With bolted connections, there is more "give" in the joint so that high localized stresses will be relieved before any serious failure.



Directly welded connection

1.4 Stress repetitions and reversals

Several problems arise when a member of a truss is subject to stress reversal. First the member and its connections must be designed so they can take stresses of both types. Second the problem of fatigue may become important and must be considered. Such failures would be primarily due to stress concentrations introduced by the constructional details. To take care of stress reverals in the web members of a truss, counter members are used. The actual distribution of stress among the counters is a statically indeterminate problem. But for simplicity, it can often be assumed that one member is acting at a time, taking tension only. This assumption is nearly correct when the members are slender so that they buckle under compression and hence take only a small proportion of the stress.

1.5 Secondary stresses in trusses

Practically all triangular trusses are designed on the assumption that the members carry direct stresses only. This assumption is true when no transverse load are applied along the length of the members, and no moments are applied or transmitted at the joint. Direct axial stresses so computed are termed primary stresses, in contrast to bending stresses which are termed secondary.

Secondary stresses may be produced in members of a truss by the following Conditions:

a). Eccentricities in the member connections - when the centroids of the sections at a joint do not intersect at one point, moments will be produced. Trusses are generally detailed so that such eccentricities will be avoided or minimized.

b). Torsional moments – introduced by members not lying in the plane of the truss, such as floor beams in bridges. For usual designs, such stresses are neglected.

c). Transverse loads on a member, such as the weight of the member Itself. These are considered only when they are appreciable.

d). Truss distortion and rigidity of joints which together induce the Bending of the members.

The term secondary stresses in trusses, when used in its narrower sense, are often intended to denote only those produced by truss distortion and the rigidity of joints. The magnitude of these secondary stresses varies greatly. For common forms of trusses with members of high slenderness ratios, the secondary stresses generally range from 5 to 25% of the primary stresses. For subdivided trusses and warren truss with verticals, certain members may have secondary stresses as high as 40 to 100% of the primary stresses.

Although the magnitude of secondary stresses can be high, their significance is not necessarily comparable to that of primary stresses. High secondary stresses exist only in some of the members and then only in the extreme fibers at the ends of the members. Even when these localized stresses reach the yield point, they may not cause collapse of the structures. With the low value of basic allowable stresses used in design , such high localized stresses do not become a problem unless repeated often enough, in which case fatigue failure may occur.

When it is desired to limit or reduce secondary stresses resulting from truss distortions, the width of the members in the plane of bending should be reduced relative to the length of the members. It is wise to choose truss types with low secondary stresses. There is a certain amount of reserve in the basic design stresses to allow for secondary stresses of the usual magnitude only when secondary stresses exceed approximately 20% of the primary ones, need they be considered in design.

Literature Review

2.1 Secondary Stresses in Trusses

R.Shankar nair (*Principal in the firm of RTKL Associates Baltimore, Maryland.)*

This paper was developed as part of the program of the ASCE committee on steel building structures.

Secondary stresses in steel trusses may be neglected in most cases. It is important, however, that secondary stresses be defined properly and the analysis and design be consistent with each other, as follows:

1. If the truss members are designed for the axial forces that would occur if the members were pin-connected, then the flexural stresses indicated by a more refined analysis may be defined as secondary stresses and may be neglected, within reasonable limits.

2. If the axial forces for member design are obtained from an analysis that includes flexural effects, flexural stresses cannot be dismissed as secondary stresses, since the presence of flexural effects in the analysis might have reduced the axial forces indicated by the analysis. In this case, it is to neglect flexural stresses, must first judge whether (and by how much) the axial forces indicated by the analysis were affected by flexural effects. And then make appropriate corrections in the axial forces to be used for design.

In most trusses of customary shape and dimensions, flexural stresses will be lower than the allowable limit. However, in trusses with very large gusset plates or unusually stubby members (where the ratio of member width to free length outside connections is more than about 1/5), flexural stresses might be much higher than the recommended secondary stress limit and should be checked by analysis. If flexural stresses are found to be excessive, the "truss" should be regarded as a "frame" and members should be designed for axial force, flexure and shear.

This discussion, so far, has been restricted to trusses in which members meet concentrically at panel points and all loads are applied at those points. Similar reasoning may be used for other types of trusses, as explained here:

2.2 Truss with loads applied on chord between panel points

The loaded chord may be analyzed as a continuous beam on non-moving, knife-edge supports at the truss panel point locations. The truss may then be analyzed as a pin connected structure. The reactions from the chord/beam analysis should be applied as joint loads in the truss analysis. The flexural stresses in the chord from the beam analysis and the axial stresses in all members from the truss analysis are *primary* stresses. The additional flexural stresses in the chord and the flexural stresses in other members caused by truss deformation are secondary tresses and may be neglected. If design is based on a single analysis, it is not possible to separate primary and secondary stresses. A reasonable compromise, in this case, is to model the loaded chord as a continuous member and other elements as pin-connected members. All the resulting stresses should be regarded as primary stresses, even though the chord flexure would include truss-deformation effects.

2.3 Truss with eccentric joints

Some trusses are detailed with the centroids of web members meeting near the edge of the chord instead of at the centroid of the chord. Typically, these trusses have heavy chords (for loads applied between panel points) and light web members. There is no practical way of separating primary and secondary stresses in these trusses. A reasonable approach for design purposes is to model the chord as a continuous member with rigid stubs to the web member intersection points. The web members may be modeled as pin-connected elements. All the resulting stresses should be treated as primary stresses, even though the flexure in the chord would include truss-deformation effects.

In summary, the key to proper treatment of secondary stresses in steel trusses is to be consistent between analysis and design. If member forces for design are determined from an analysis that neglects certain stiffness components (such as flexural stiffness of some or all members), stresses corresponding to those stiffness components may be regarded as secondary stresses and may be neglected in design. This approach is consistent with *plastic* and *ultimate* design concepts. Limits on secondary stress need to be observed only to guard against local buckling, connection distress, fatigue and other problems which might occur in unusual cases.

2.4 Design of Steel Structures.

Boris Breslar, T.Y. Lin, John B. Scalzi
(*Publisher: Wiley Eastern Private Limited.*)

Secondary stresses may be produced in members of a truss by the following Conditions:

a). Eccentricities in the member connections - when the centroids of the sections at a joint do not intersect at one point, moments will be produced. Trusses are generally detailed so that such eccentricities will be avoided or minimized.

b). Torsional moments – introduced by members not lying in the plane of the truss, such as floor beams in bridges. For usual designs, such stresses are neglected.

c). Transverse loads on a member, such as the weight of the member Itself.

These are considered only when they are appreciable.

d). Truss distortion and rigidity of joints which together induce the Bending of the members.

The term secondary stresses in trusses, when used in its narrower sense, are often intended to denote only those produced by truss distortion and the rigidity of joints.

Finite Element Method

3.1 Introduction

The Finite Element Method (FEM) is a numerical analysis for obtaining approximate solutions to a wide variety of engineering problems. This has developed simultaneously with the increasing use of high-speed electronic digital computers and with the growing emphasis on numerical methods for engineering analysis. Although originally developed to study stresses in complex airframe structure, it has been extended and applied to the broad field of continuum mechanics. Because of its diversity and flexibility as analysis tool, it is receiving much attention in engineering field and in industry.

3.2 General Description of Finite Element Method

In finite element method, a body or a structure is divided into smaller elements of finite dimensions called "finite elements". The original body or the structure is then considered as an assemblage of these elements connected at a finite number of points called "nodes" or "nodal points". The properties of the elements are formulated and combined to obtain the solution for the entire body or structure.

The equations of equilibrium for the entire structure or body are then obtained by combining the equilibrium equation of each element such that the continuity of displacement is ensured at each node where the elements are connected. The necessary boundary conditions are imposed and the equations of equilibrium are solved for the nodal displacements. Having thus obtained the values of displacements at the nodes of each element, the strains and stresses are evaluated using the element properties derived earlier.

In this method attention is mainly devoted to the formulation of properties of the constituent elements. The procedure for combining the elements, solution of equations and evaluation of element strains and stresses are the same for any type of structural system or body. Hence the finite element method offers scope for developing general purpose programs with the properties of various types of elements forming an "element library" and the other procedures of analysis forming the common core segments.

3.3 Advantages of FEM

The Finite Element Method has many advantages of its own. Some of them are given below.

- Various types of boundary conditions are automatically handled in the formulation. They are systematically enforced just before the solution, for the nodal values of the field variables are obtained.
- Material anisotropy and non-homogeneity can be treated without much difficulty.
- Any type of loading can be handled.
- Higher order elements may be implemented with relative ease.
- The method can efficiently be applied to cater irregular geometry.
- Spacing of nodes need not follow a pattern or rule.

3.4 Disadvantage of FEM

The Finite Element Method has its own limitations. They are listed below.

- There are many types of problems where some other method of analysis may prove more efficient than the FEM.
- There are some trouble spots such as "Aspect Ratio" (ratio of the longer to smaller directions), which may affect the final results.
- The cost involved in the solution of the problem is high.
- The Finite Element Method is an approximate method, stress values may vary by 25% from finer mesh analysis to average mesh analysis.

3.5 Application of FEM

Applications of Finite Element Method divide into three categories, depending on the nature of the problem to be solved. In this first category are the problems known as equilibrium problems or time dependent problems. The majority of applications of Finite Element Method fall into this category.

For the solution of equilibrium problems in the solid mechanics area we need to find the displacement distribution and stress distribution for a given mechanical or thermal loading. Similarly for the solution of equilibrium problems in fluid mechanics, it is necessary to find pressure, velocity, temperature and density distributions under steady state conditions.

The second category is the Eigen value problems of solid and fluid mechanics. These are steady state problems whose solution often requires the determination of natural frequencies and models of vibration of solids and fluids. Examples of Eigen value problems involving both solid mechanics (eg. elasticity, plasticity, statistics and dynamics) and fluid mechanics (eg. Viscous and viscid) appear in Civil Engineering when the interaction of lakes and dams is considered and in Aerospace Engineering, when the sloshing of liquid fuels in flexible tanks is involved. Another class of Eigen value problems includes the stability of structures and the stability of laminar flow.

The third category is the multitude of time dependent or problems of continuum mechanics. This category is composed of the problems that result when the dimension is added to the problems of the first two categories.

The range of possible applications of the Finite Element Method extends to all Engineering disciplines but Civil, Mechanical and Aerospace engineers are the most frequent users of the method. In addition to the structural analysis other areas of applications include heat transfer, Solid mechanics, Electromagnetism, Biomechanics, Geo-mechanics and Acoustics. The method finds acceptance in multidisciplinary problems where there is coupling between heat transfer and displacements as well as aero plasticity where there is a strong coupling between external flow and the distortion of wing.

3.6 ANSYS

3.6.1 General

The ANSYS computer program is a general purpose Finite Element Modeling package for numerically solving a variety of mechanical problems. These problems include static and dynamic structural analysis (both linear and non-linear), steady state and transient heat transfer problems, mode-frequency and buckling analyses, acoustic and electro magnetic problems and various types of field and coupled-field applications. The program contains many special features which allow nonlinearities or secondary effects to be included in the solution such as plasticity, large strain, hyper elasticity, creep, swelling, large deflections, contact, stress stiffening, temperature dependency, material anisotropy and radiation.

An ANSYS has been developed, other special capabilities, such as sub structuring, sub-modeling, random vibration, kinetostatics, kinetodynamics, free convection fluid analysis, acoustics, magnetics, piezoelectrics, coupled-field analysis and design optimization have been added to the program. These capabilities contribute further to making ANSYS a multi-purpose analysis tool for varied engineering disciplines.

3.6.2 Program Overview

The ANSYS element library contains more than sixty elements for static and dynamic analyses, over twenty for heat transfer analyses and numerous magnetic field and special purpose elements. This variety of elements allows the ANSYS program to analyze two and three dimensional frame structures, piping systems, two dimensional plane and axisymmetric solids, flat plates, axisymmetric and three dimensional shells and non-linear problems including contact, interface and cables.

The program is divided into many processors where each processor has a particular job to perform.

- Pre Processor: This builds the model.
- Solution Processor: This is for assigning loads, constraints and finally to get Finite Element Solution.
- General Post Processor: This is for further processing and viewing the results over the entire model at specific time points.
- Time History Post Processor: Reviews results at specific points in the model as a function of time.
- Topological optimization: Execute several topological optimization iterations.
- ROM Tool.
- Design Optimization: This improves an initial design.
- Probabilistic Design: This accounts for the inaccuracies and uncertainties influencing the outcome of an analysis by the use of a random input variable.
- Radiation Matrix: This calculates radiation view factors and generates radiation matrix for thermal analysis.
- Run Time Statistics: Predicts CPU Time, Wave front Requirements etc for an analysis.
- Session Editor: Allows the user to modify or save commands issued since the last RESUME or SAVE command.

A Graphical User Interface (GUI) is available throughout the program, to guide new users through the learning process and provide more experienced users with multiple windows, pull-down menus, dialogue boxes, tool bars and online documentation.

In this study, the linear behavior of various truss and lattice models are analyzed by the general-purpose finite element software ANSYS 10.0.

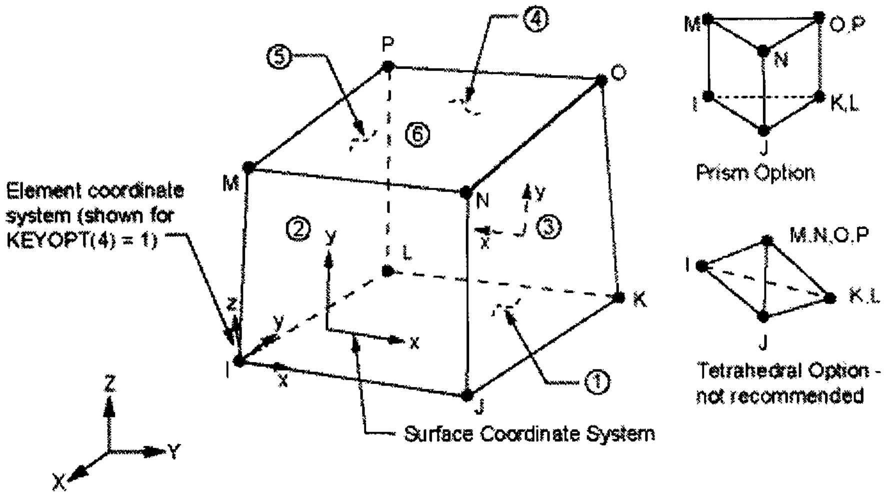
3.6.3 Convergence study

Selection of mesh density plays a vital role in finite element modeling. A preliminary convergence study has been carried out for steel section to determine optimum mesh density. A convergence of results is obtained when an adequate number of elements are used in a model. This is practically achieved when an increase in mesh density does not significantly alter the output information (Adams and Askenki 1998).

3.6.4 Element type : SOLID45

SOLID45 is used for the 3-D modeling of solid structures. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions.

The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities. A reduced integration option with hourglass control is available. A similar element with anisotropic properties is SOLID64. A higher-order version of the SOLLID45 element is SOLID95.



SOLID45 Geometry.

The geometry, node locations, and the coordinate system for this element are shown in Figure 1.0: "SOLID45 Geometry". The element is defined by eight nodes and the orthotropic material properties. Orthotropic material directions correspond to the element coordinate directions. Pressures may be input as surface loads on the element faces

Assumptions and Restrictions.

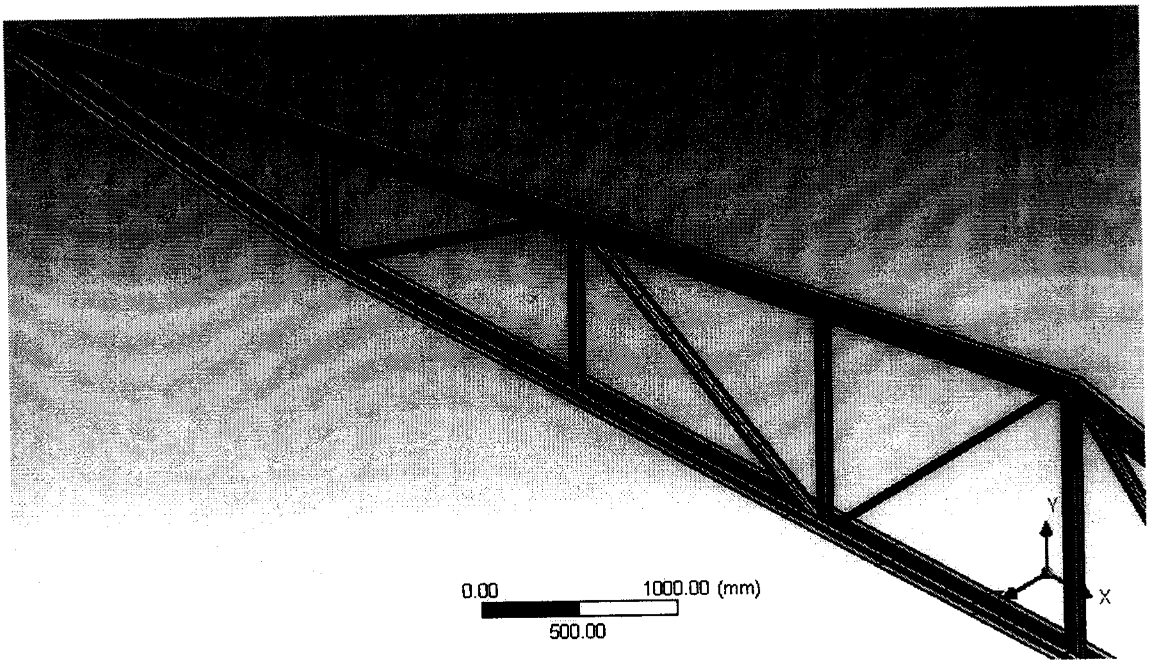
- Zero volume elements are not allowed.
- The element may not be twisted such that the element has two separate volumes. This occurs most frequently when the elements are not numbered properly.
- All elements must have eight nodes.
- A prism-shaped element may be formed by defining duplicate K and L and duplicate O and P node numbers (see Triangle, Prism and Tetrahedral Elements

3.6.5 Material properties

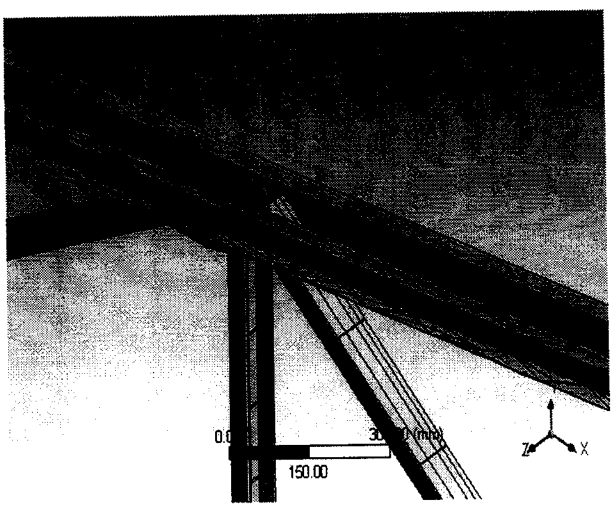
The SOLID45 element requires linear isotropic material properties to properly model mortar. The multilinear isotropic material uses the Von Mises failure criterion to define the failure of the steel. Poisson's ratio (μ) of steel was assumed as 0.30. Yield strength of the material is 250 mPa. The elastic modulus of the material is 200000 mPa.

3.6.6 Finite Element Discretization

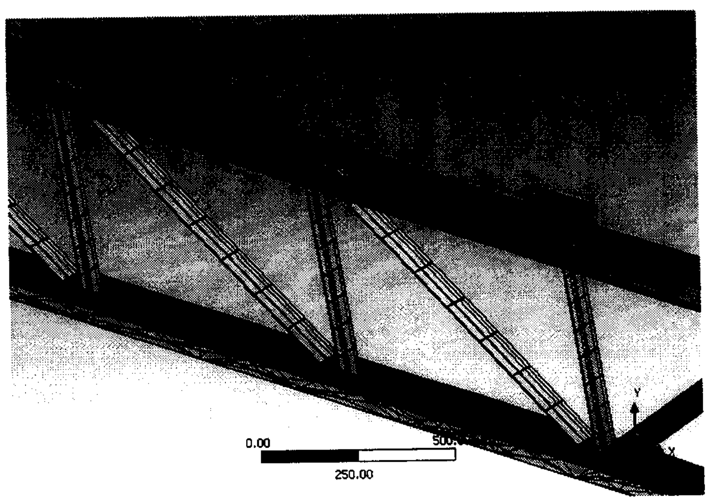
The finite element models were created as single volume and meshing was performed using free mesh option. The total number of line elements varies from specimen to specimen due to the variation in the dimension. For accurate results the discretization requires very finer meshing.



Pitched roof truss



Typical truss joint



Lattice truss

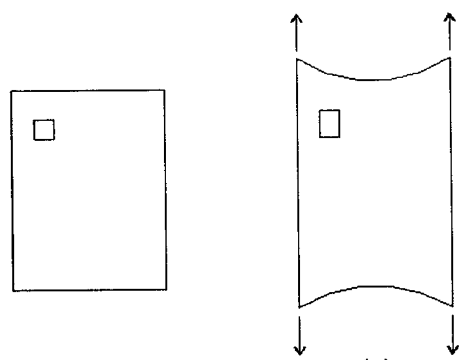
Shear Lag and its effects

Shear lag and its effects

Shear lag describes the behavior at an end connection of a tension member. Where some but not all, of the cross sectional elements are connected. The area that is effective in resisting tension may be less than full calculated net area for the cross section.

For example, a single angle tension member connected by only one leg. The adjacent leg at the connection dues very little in the way of resisting tension at the connection, but becomes fully effective beyond the connection act in the length of tension member.

The phenomenon of non confirm straining of web is called shear lag.



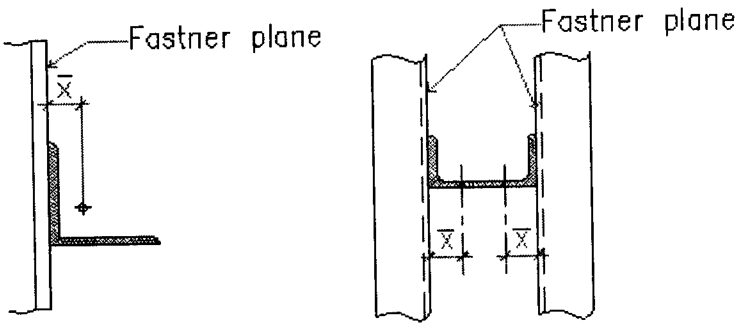
(a) (b)

The above figure show the web of member in the unloaded and loaded state, helps to explain the significance of this name. the four tones shown in fig(b) are the resultants of weld shear in the connections. Since the ends of web are free, the distribution will be as shown. Therefore an element such that at A in the unloaded will be deformed as shown in fig(b) when member is loaded. This is a shear deformation and the shear in web is said to lag because of it. The shear lag phenomenon can be analyzed by considering this deformation.

Since shear lag reduces the effectiveness of tension member components that are not connected directly to a gusset plate or other anchorage. The efficiency of a member can be increased by reducing the areas of such components relative to the areas of the member as a whole.

The distance from a fastener plane to the centre of gravity of the area tributary to it is a convenient measure of distribution of the cross sectional area of the member. For example, the tributary area of the single angle member of fig 2.0(a) is the entire area of the angle and the coordinate x from the fastener plane to the centroid of the area is a measure of the efficiency of the cross section.

Similarly in the double plane member of fig 2.0(b) the area tributary to each fastener plane is the area of half the cross section, and the coordinate x from each fastener plane to the centroid of each half cross section measures the relative importance of the unconnected web.



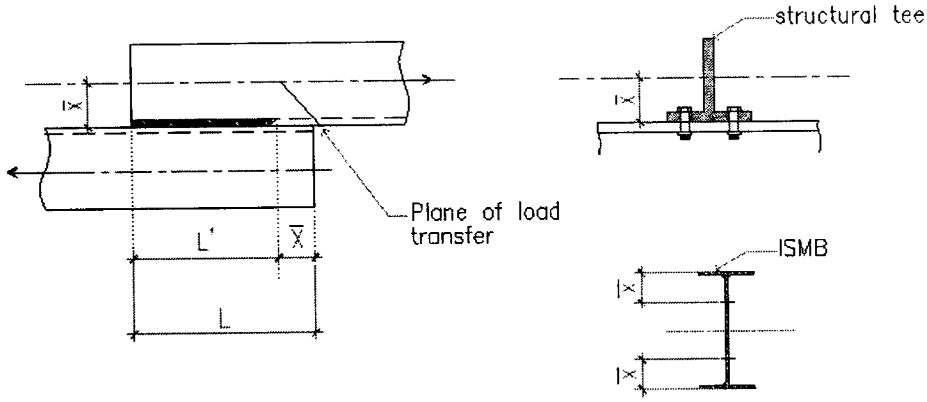
(a) (b)

Shear lag is also influenced by the length of connection. Net section effectiveness decreased with a decrease in the length of the fastener line. The effect of these two parameters can be expressed as an efficiency coefficient given by

$$U = 1 - x/L$$

Where L is the length of connection.

When a member is loaded in axial tension until failure occurs its net section it's actual tensile failure stress will be probably be less than the allowable tensile strength of the steel unless all of the various elements which make up the section are connected so stress is transferred uniformly across the section. The reason for the reduced strength of the member is the concentration of shear stress called shear lag. In such a situation the flow of tensile stress between the full member cross section and the smaller connected cross section is not 100% effective. The smaller the value of x the larger the effective area of the member, Several values of *x*



4.1 Calculation of rupture strength (6.3.3 of IS 800:2007)

The rupture strength of an angle connected through on leg is affected by shear lag. The design strength 'T_{dn}' as governed by rupture at net section is given by

$$Tdn = 0.9\, Anc\,(fu/\gamma) + \beta\, Ago\,(fy/\gamma mo)$$

Where $\beta = 1.4 - 0.076(w/t)(fy/fu)(bs/Lc) \leq (fu\,\gamma mo/fy\,\gamma ml)$

≥ 0.7

Where, w = outstand leg width

bs = shear lag width

Lc = Length of end connection (That is the distance between the outermost bolts in the end joint measured along the load direction or length of weld along the load direction.)

For preliminary sizing, the rupture strength of net section may be approx. taken as

$$Tdn = \alpha\, Ah\, fu\, /\, \gamma ml$$

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

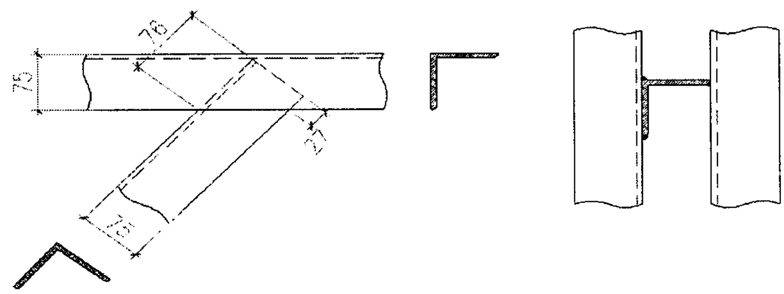
An net area of total cross section

Anc net area of connected leg

Ago gross area of outstanding leg

t thickness of leg.

4.2 Rupture strength (Tdn) of typical truss web chord ISA 75x75x6 due to shear lag



L_c= Length of the weld = 76x2 + 27=179 mm

b_s= 75 mm = w

t = 6 mm

$$\beta = 1.4 - 0.076\left(\frac{75}{6}\right)\left(\frac{250}{410}\right)\left(\frac{75}{179}\right)$$

$$= 1.1572 \leq \left(410 * \frac{1.10}{250} * 1.25\right) = 1.44$$

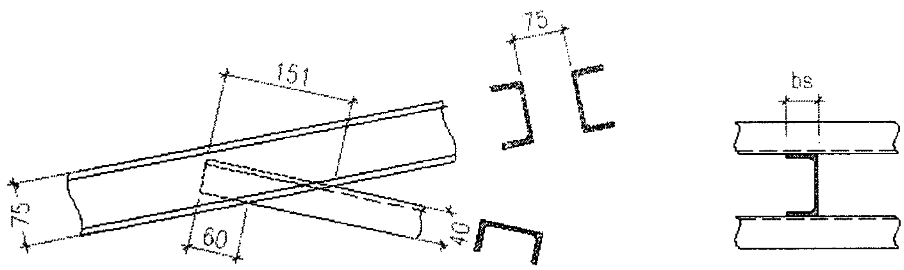
$$\geq 0.7$$

A_{nc}= (75–3) x 6 = 432 mm^2

$$T_{dn} = 0.9 * 432 * \left(\frac{410}{1.25}\right) + 1.1573 * 432 * \left(\frac{250}{1.10}\right)$$

T_{dn} = 241.15 kN. < 864 x 410 / 1.25 = 283.39 kN.

4.3 Rupture strength (Tdn) of typical truss web chord ISMC 75 due to shear lag



Length of the weld = (151+60) x 2 = 422 mm

b_s = 40 x 2 = 80 mm

t_f = 7.5 mm t_w = 4.8 w = 75 mm

$$\beta = 1.4 - 0.076 * \left(\frac{75}{4.8}\right)\left(\frac{250}{410}\right)\left(\frac{80}{422}\right)$$

= 1.263

≤ 1.44

≥ 0.7

A_{nc} = (40–2.4) x 7.5 = 282 mm^2 A_{go} = 75 x 4.8 = 360 mm^2

$$T_{dn} = 0.9 * 282 * \left(\frac{410}{1.25}\right) + 1.263 * 360 * \left(\frac{250}{1.10}\right)$$

T_{dn} = 186.58 kN. < 888 x 410 / 1.25 = 291 kN.

4.4 Effective area of tension member

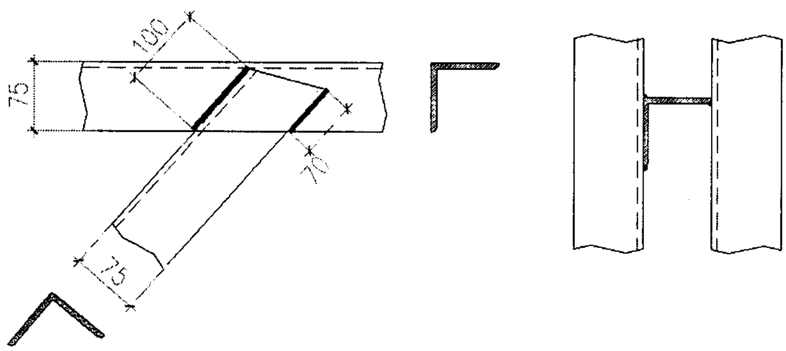
The area which becomes effective in resisting force depends on few factors.

a). End condition or joint through which the force is transmitted.

b). Directly connected area in relation to the gross area.

In the case of double plane angle truss, the both legs of angle web chord is directly welded to the main chords, and hence the entire cross section of angle is considered to be effective. The section is affected only by the shear lag. the effective cross section is reduced by a factor called shear lag factor U

For ISA 75x75x6 the shear lag factor U is calculated by,



$$U = 1 - x/L$$

X = Cyy = 21.0mm

L = Length of connection = 270mm

U = 1- (21.0 / 270)

= 0.922

Hence the effective area for ISA 75x75x6 is Aeff = Ag x U

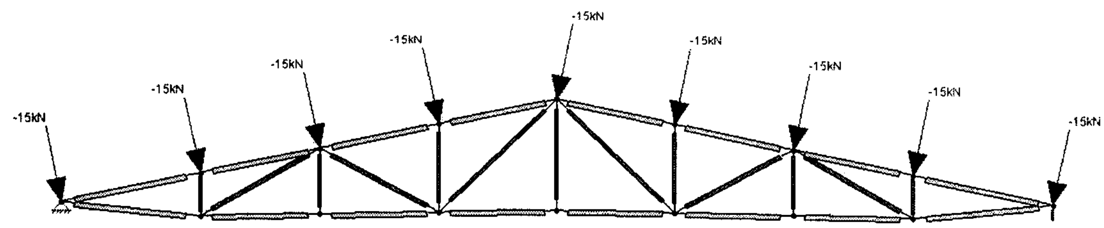
= 864 x 0.922 = 796mm^2

Analysis using ANSYS

Analysis of the various truss system have been made to study the behavior of members and connections. For this study following kinds of analysis were performed using ANSYS.
1) Analysis of pitched roof truss with connection eccentricity, 2) Analysis of pitched roof truss without connection eccentricity using gusset plates. 3) Lattice truss with application of loads at nodes and 4).Lattice truss with application of loads at chords.

Due to connection eccentricity of the members, truss joint is subjected to secondary stresses from the finite element analysis using ANSYS. It is intended to find whether the resulting stresses are within the allowable limit.

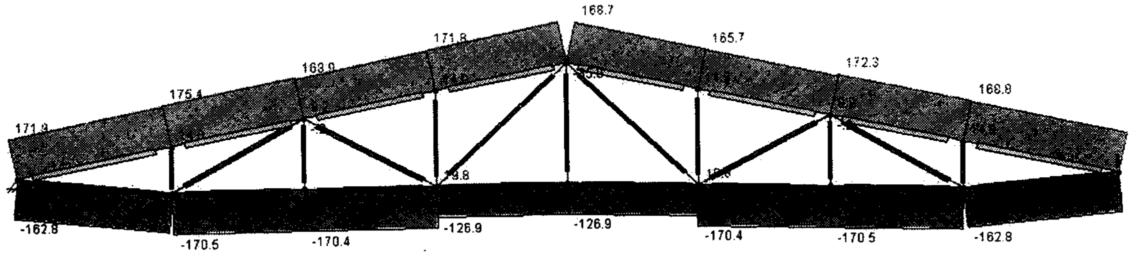
5.11 Pitched roof truss with connection eccentricity



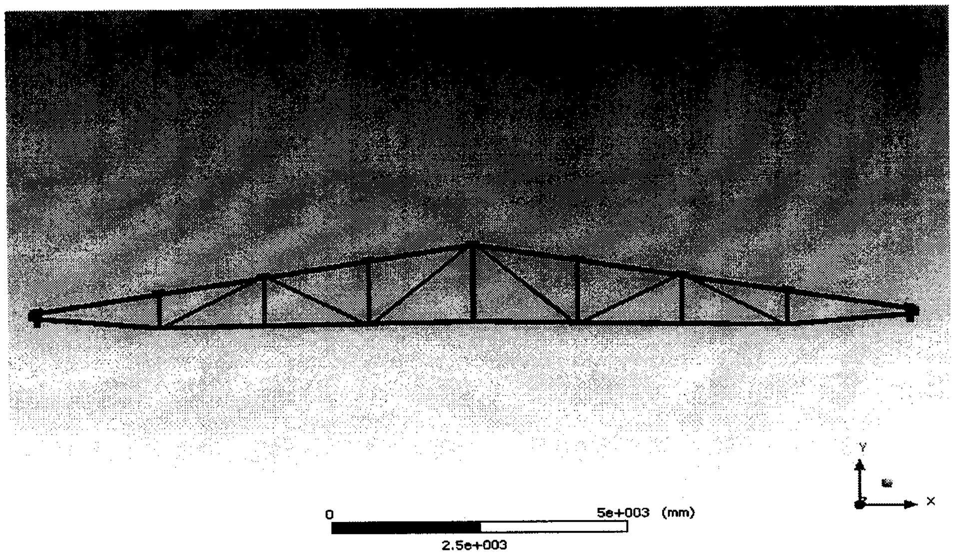
Analytical model using conventional software

Truss joints are modeled as pin jointed and the members are analysed in the conventional system. But in practice the assumed pin joint is not possible to achieve. When modeling the load is applied exactly over the nodal points. It is also not achived in the field, because the load is transferred through the cleats which are placed eccentric to the main chords.

Here the truss is modelled such that one end is pinned and the other end is roller, all the inner joints are free and the load is applied over the nodes. The members are designed for the axial compression or tension corresponding to the loads.Allowable stresses are calculated by considering the cross sectional area of the member and its slenderness effects with out considering the effect of eccentricity.



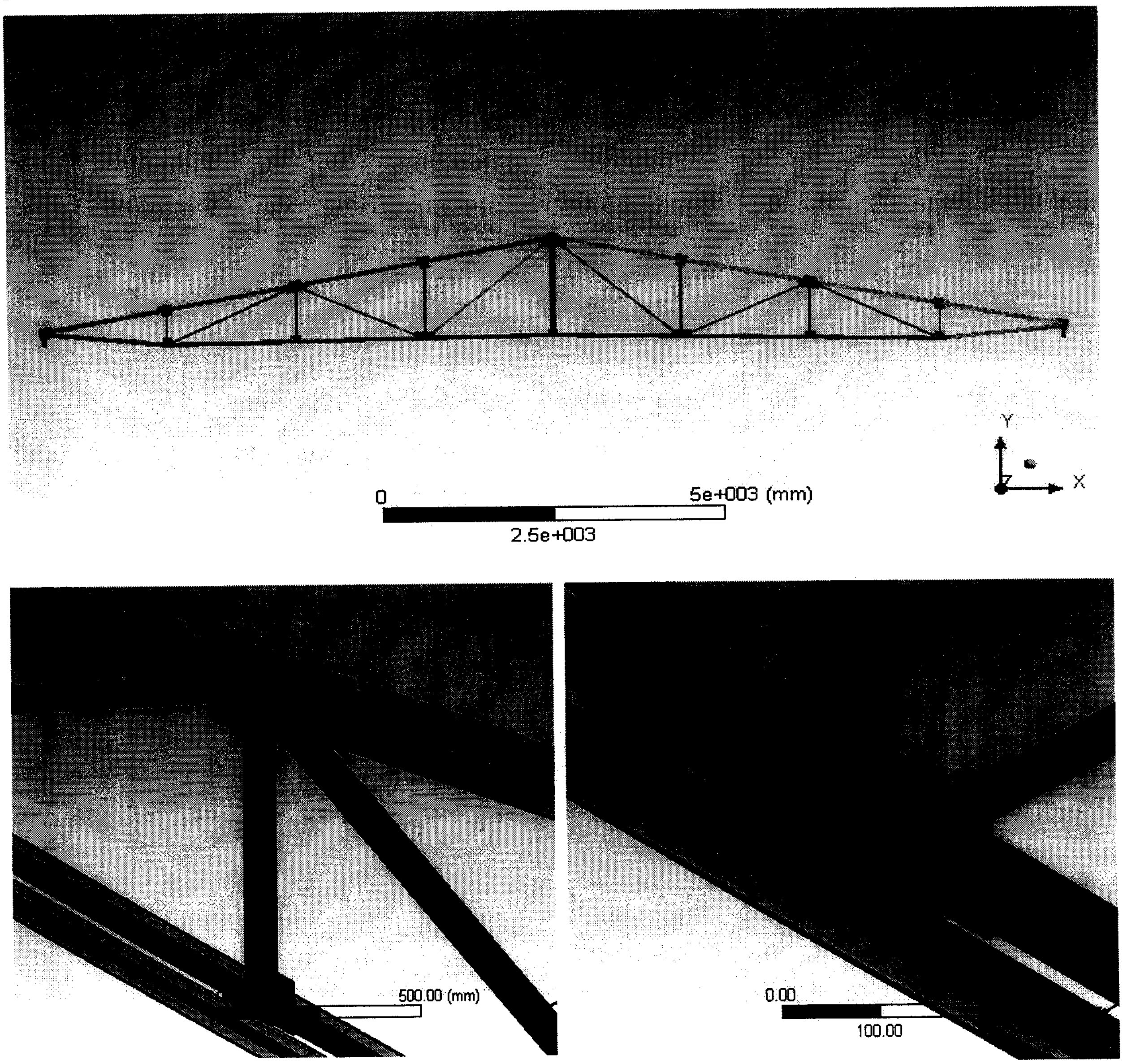
Axial force diagram



Finite Element model of truss using element BEAM 3 in ANSYS

5.12 Pitched roof truss with concentric connection (with gusset plates)

The truss is modelled with 2 nos of ISMC 75 Back to back with 99mm inner spacing and the inner web chords as ISMC 75. Channel flanges are turned outside to provide greater lateral rigidity and this is the common arrangement for compression members. Gusset plates of 12mm thickness are provided inside the main chords and the web chords are inserted and welded to the plates. such that centroidal axes of all the members meet concentrically. Nodal loads are applied through the cleat plate of size 200x50 which is placed on one of the top chord. Size of the gusset plates are arbitrarily arrived.



Finte Element model

5.13 Parallel chord lattice truss with loads at nodes

Lattice truss is often desirable for long members transmitting relatively small loads. The function of the lattice is to assure integral action of the solid longitudinal segments, usually called "main" segments. In designing lattice members the following conditions must be considered:

1). Buckling of the member as a whole under axial load.

2). Buckling or yielding of individual segments.

3). Strength of lattice frame and distortion of the cross section.

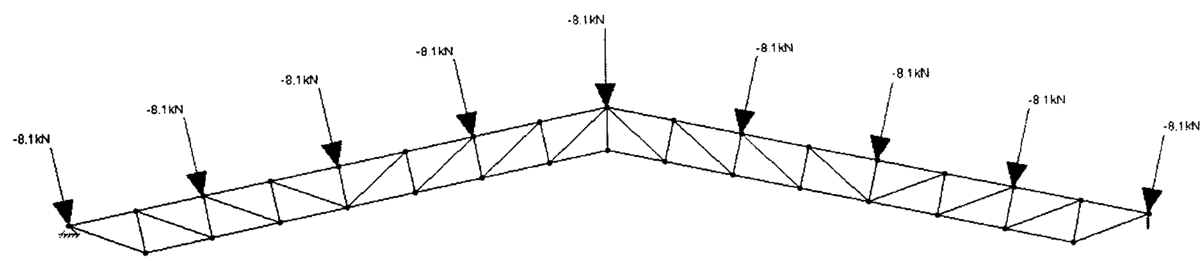
In order to assure integral action of main segments, the lattice must be capable of resisting the following loads:

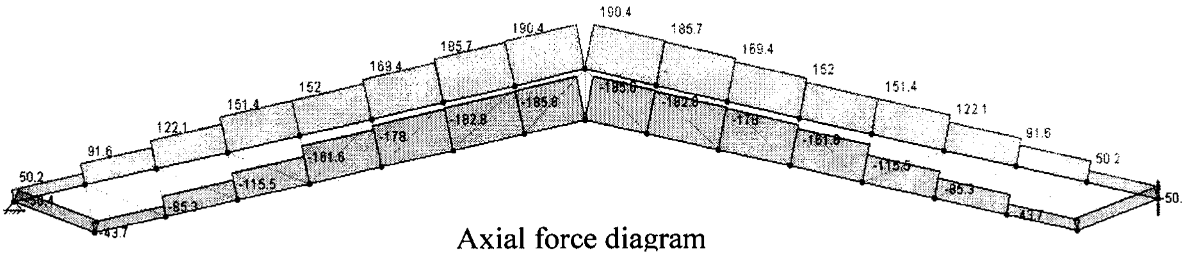
1). Stresses due to external transverse loads,

2). Stresses due to deflected shape of the frame.

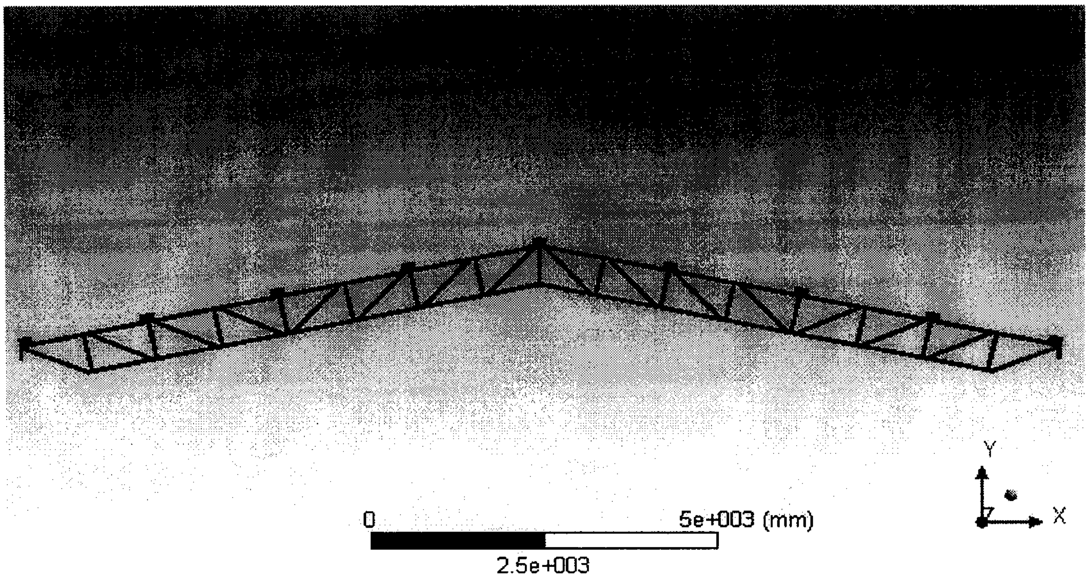
In addition to over all buckling of the frame and the strength of latticed webs, the possibilities of distortion in the plane of cross section must be considered. Usually such distortion is prevented by the use of transverse diaphragms or bracing. Axial and bending stresses in the main segments and in the lacings should not exceed safe limits. Shearing stresses in the main segments and in the lacings should also be checked.

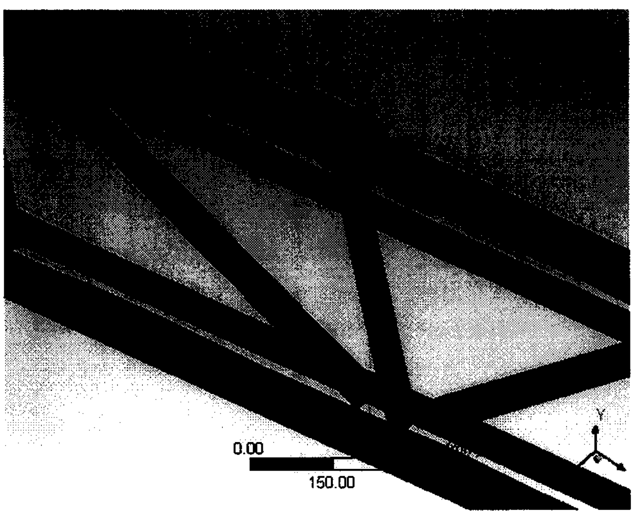
In this analysis the depth of lattice is kept 0.75m. Main chords are modeled with 2 nos of ISMC 75 Back to back with 75mm inner clear spacing and the inner web chords as ISMC 75. Channel flanges are turned outside. Loads are applied though the cleats of size 200x50mm which are placed over the top chords. The inclined web chords are configured such that it carries only axial tension and the vertical web chords carries axial compression.

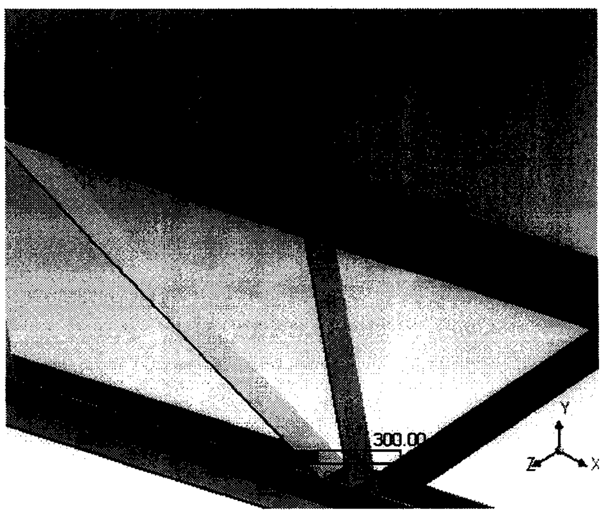




Axial force diagram







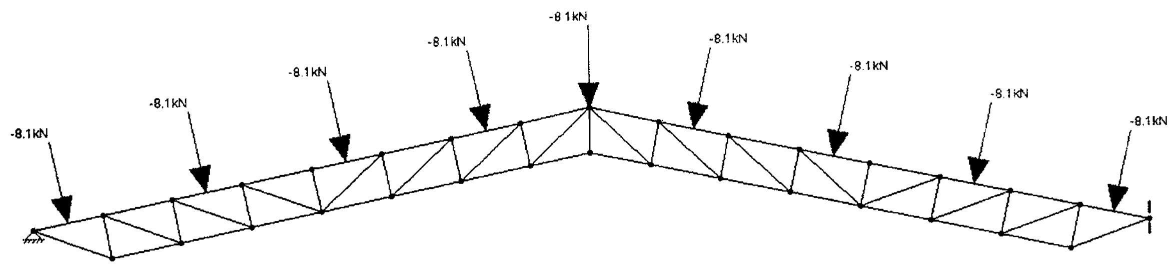
Finite Element model

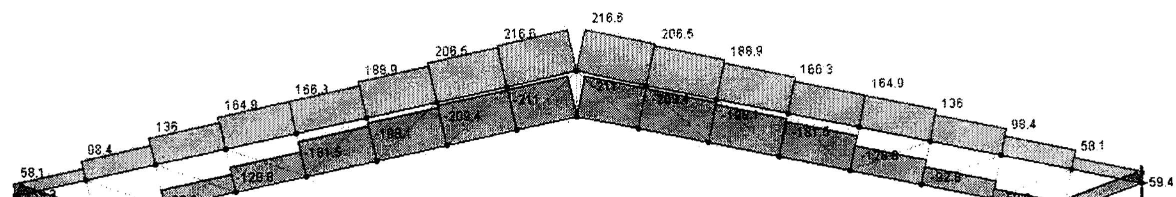
Application of load

5.14 Parallel chord lattice truss with loads at chords

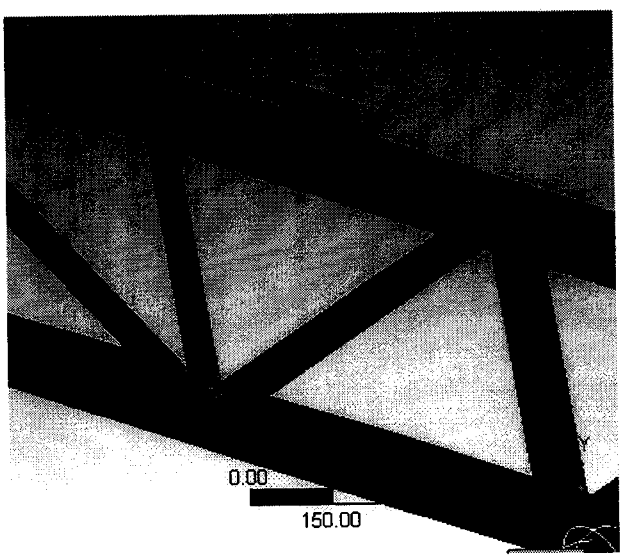
In this analysis the depth of lattice is kept 0.75m. Main chords are modeled with 2 nos of ISMC 75 Back to back with 75mm inner clear spacing and the inner web chords as ISMC 75. Channel flanges are turned outside. Loads are applied though the cleats of size 200x50mm which are placed in between the panel points. i.e. on top main chords.

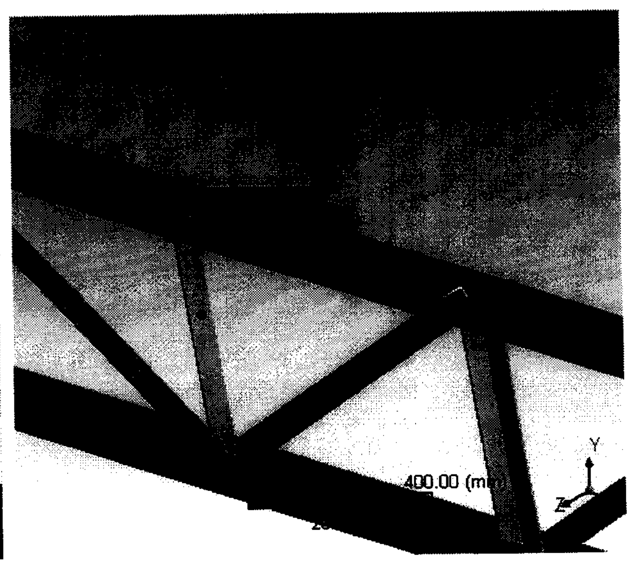
In this analysis it is intended to find the buckling or yielding of individual segments and whether axial and bending combined stress is within the safe limit.





Axial force diagram





Finite Element model *Application of load*

5.15 Analysis and design of pitched roof truss

Load calculation

Spacing of truss	=	9.7	m
Spacing of Purlin	=	1.8	m

Dead load

Dead load of truss system	=	0.20	kN/m^2
Weight of roofing sheet (AC sheet)		0.16	kN/m^2
Imposed Load			
Live load (as per relevant code spcification)	=	0.50	kN/m^2

Nodal loads at purlin points (DL+LL)	=	(9.7x1.8x0.86)	kN
	=	15.0	kN
Cleat plate size	=	200x10	Mm
Pressure	=	15x1000/(200x10)	Mpa
	=	7.5	Mpa

5.16 Design of main chord (Double channel back to back subjected to compressive load (as per IS 800-2007)

Axial compression (working load)		P	=	**175**	kN
Load Factor			=	**1.5**	
Effective length factor (major)		K_z	=	**0.85**	
Effective length factor (minor)		K_y	=	**0.85**	
Partial safety factor for material strength		g_{mo}	=	**1.1**	
Factored load		Pu	=	**262.5**	kN
Yield strength of material		fy	=	**250**	Mpa
Modulus of elasticity of material	mPa	E	=	200000	Mpa
Buckling class			=	**c**	
Imperfection factor		a	=	0.49	

try with section	**ISMC**	**75**		
Clear Spacing	S	=	**75**	mm
Section properties				
	Area	=	1776	mm^2
	y_o	=	13.5	mm
	Ixx	=	1530900	mm^4
	Iyy	=	4802916	mm^4
	rxx	=	29	Mm
	ryy	=	51	mm
Length of member about major axis	L z-z	=	**1762**	mm
Length of member about minor axis	L y-y	=	**1762**	Mm
Slenderness ratio abot major axis	K_zL_z/r_{xx}	=	51	
Slenderness ratio abot minor axis	K_yL_y/r_{yy}	=	29	
Eulers buckling stress	fcc_z	=	759	Mpa
	fcc_y	=	2290	Mpa
Non dimensional eff. Slenderness ratio	l_z	=	0.574	
	l_y	=	0.330	
inclination of tension field stress in web	f_z	=	0.76	
	f_y	=	0.59	
Stress reduction factor	c_z	=	0.80	
	c_y	=	0.93	
Design compressive stress(major)	fcd_z	=	182.0	Mpa
Design compressive stress (minor)	fcd_y	=	212.2	Mpa
Design compressive stress		=	182.0	Mpa
Capacity of the section		=	**323.2**	kN

5.17 Design of web chord (single channel subjected to compressive load (as per IS 800-2007)

Axial compression (working load)		P	=	**35**	kN
Load Factor			=	**1.5**	
Effective length factor (major)		K_z	=	**0.85**	
Effective length factor (minor)		K_y	=	**0.85**	
Partial safety factor for material strength		g_{mo}	=	**1.1**	
Factored load		Pu	=	**52.5**	kN
Yield strength of material		fy	=	**250**	Mpa
Modulus of elasticity of material	mPa	E	=	**200000**	Mpa
Buckling class			=	c	
imperfection factor		a	=	0.49	
try with section	**ISMC 75**				
Section properties					
		Area	=	888	mm^2
		Ixx	=	765450	mm^4
		Iyy	=	90546	mm^4
		rxx	=	29	mm
		ryy	=	10	mm
Length of member about major axis		L z-z	=	**1800**	mm
Length of member about minor axis		L y-y	=	**1800**	mm
Slenderness ratio about major axis		K_zL_z/r_{xx}	=	52	
Slenderness ratio about minor axis		K_yL_y/r_{yy}	=	152	
Euler's buckling stress		fcc_z	=	727	Mpa
		fcc_y	=	86	
Non dimensional eff. Slenderness ratio		l_z	=	0.586	
		l_y	=	1.705	
inclination of tension field stress in web		f_z	=	0.77	
		f_y	=	2.32	
Stress reduction factor		c_z	=	0.79	

	c_y	=	0.26	
Design compressive stress(major)	fcd_z	=	180.3	Mpa
Design compressive stress (minor)	fcd_y	=	58.3	Mpa
Design compressive stress		=	58.3	Mpa
Capacity of the section		=	**51.8**	kN

5.18 ANSYS stress results and comparison of stresses

Element Designation	Member	Member length (mm)	Vonmises stress (Mpa)			member force as per analysis	Actual area provided	stress induced as per calc.	Allow. stress as per calc.
			Max	Location mm	at mid section				
Top chord									
A	ISMC 75 B/B	2121	237.37	325	106.51	171.9	1776	96.79	166.60
B	ISMC 75 B/B	1880	165.4	400	113.83	175.4	1776	98.76	176.80
C		1800	127.92	550	111.51	168.9	1776	95.10	180.30
D		1800	127.34	950	127.34	171.8	1776	96.73	180.30
Bottom chord									
E	ISMC 75 B/B	2026	185.47	695	124.23	162.8	1776	91.67	170.30
F		1762	135.97	705	120.8	170.5	1776	96.00	182.00
G		1762	98.07	250	88.94	170.4	1776	95.95	182.00
H		1762	82.26	851	82.263	126.9	1776	71.45	182.00
Web chord									
I	ISMC 75	637	195.58	150	14.82	14.82	888	16.69	178.00
J		2029	99.47	250	27.36	9.23	888	10.39	47.70
K		975	150.16	end	4	2.1	888	2.36	133.00
L		2000	102.91	end	23.35	19.8	888	22.30	48.90
M		1312	130.94	end	38.3	14.905	888	16.78	94.00
N		1800	187.49	end	32.49	35	888	39.41	34.70

5.19 Design of fillet weld joint for web chord of ISMC 75

Member tensile force = 130 kN (tensile capacity of the section)

Properties of member ISMC 75

A = 888 mm^2

tf = 7.5mm

Cy = 13.51mm

Design strength of Fillet weld as per (10.5.7.1.1 of IS 800:2007)

Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by:

$$f_{wd} = f_{wn} / \gamma_{mw}$$

Where

Nominal strength of fillet weld f_{wn} = $f_u / \sqrt{3}$

f_u = smaller of the ultimate stress of the weld or of the parent metal, and

γ_{mw} = Partial safety factor.

Calculation of weld length

f_u = 410 mPa

γ_{mw} = 1.5

Let Size of the weld (s) = 4.5 mm

Throat area of the weld = 4.5 x 0.7 = 3.15 mm^2

Strength of the weld per mm = $\frac{\frac{410}{\sqrt{3}}}{1.5} x\ 3.15 = 497.09 \frac{N}{mm}$

Length of the weld required = $130\ x\ 1000 / 497.09 = 261.51 mm$

The CG of weld length should coincide with that of the channel.

The weld length ratio need to be : $Cy/(40 - Cy) = 13.51/26.49$

$= 0.510$

Both flange of Member ISMC 75 is directly welded on the web of main chords at four edges.

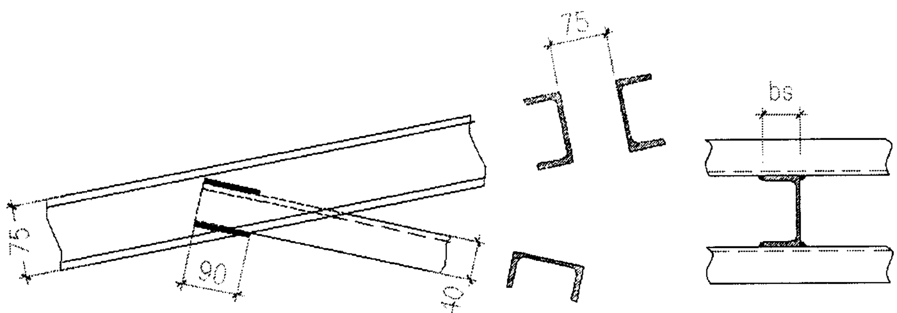
Weld length required at each edge = 261.50 / 4

= 65.4mm

= 65.4 + (4x4.5)

= 83.4mm

Provide weld length of 90mm at each end.



5.2 Analysis and design of Lattice truss

Load calculation for lattice

Spacing of truss	=	6.5	m
Spacing of Purlin	=	1.92	m
Dead load			
Dead load of truss system	=	0.10	kN/m^2
Weight of roofing sheet (Gal sheet)	=	0.05	kN/m^2
Imposed Load			
Live load (as per relevant code spcification)	=	0.50	kN/m^2
Nodal loads at purlin points (DL+LL)	=	(6.5x1.92x0.65)	kN
	=	8.10	kN
Cleat plate size	=	200x10	mm
Pressure	=	8.1x1000/(200x10)	Mpa
	=	4.05	Mpa

5.21 Design of main chord (Double angle back to back subjected to compressive load (as per IS 800-2007)

Axial compression (working load)	P	=	**175**	kN
Load Factor		=	**1.5**	
Effective length factor (major)	K_z	=	**0.85**	
Effective length factor (minor)	K_y	=	**0.85**	
Partial safety factor for material strength	g_{mo}	=	**1.1**	
Factored load	Pu	=	**262.5**	kN
Yield strength of material	fy	=	**250**	mPa
Modulus of elasticity of material	E	=	**200000**	mPa
Buckling class		=	c	
Imperfection factor	a	=	0.49	

try with section		**ISA**	**75 6**		
Clear Spacing	mm	S	=	**75**	

Section properties

	Area	=	864	mm^2
	y_o	=	21.0	mm
	Ixx	=	467694	mm4
	Iyy	=	6849036	mm4
	rxx	=	23	mm
	ryy	=	63	mm
Length of member about major axis	L z-z	=	**1917**	**mm**
Length of member about minorr axis	L y-y	=	**1917**	**mm**
Slenderness ratio abot major axis	K_zL_z/r_{xx}	=	70	
Slenderness ratio abot minorr axis	K_yL_y/r_{yy}	=	26	
Eulers bukling stress	fcc_z	=	402	Mpa
	fcc_y	=	2947	Mpa
Non dimensional eff. Slenderness ratio	l_z	=	0.788	
	l_y	=	0.291	
inclination of tension field stress in web	f_z	=	0.95	
	f_y	=	0.56	
Stress reduction factor	c_z	=	0.67	
	c_y	=	0.95	
Design compressive stress(major)	fcd_z	=	152.2	Mpa
Design compressive stress (minor)	fcd_y	=	216.7	Mpa
Design compressive stress		=	152.2	Mpa
Capacity of the section		=	**263.0**	**kN**

5.22 Design of web chord (single angle subjected to compressive load (as per IS 800-2007)

Axial compression (working load)	P	=	**26**	kN
Load Factor		=	**1.5**	
Effective length factor (major)	K_z	=	**0.85**	
Effective length factor (minor)	K_y	=	**0.85**	
Partial safety factor for material strength	g_{mo}	=	**1.1**	
Factored load	Pu	=	**39**	kN
Yield strength of material	fy	=	**250**	Mpa
Modulus of elasticity of material	E	=	**200000**	Mpa
Buckling class		=	c	
Imperfection factor	a	=	0.49	
try with section **ISA**	**75 6**			
Clear Spacing mm	S	=	**75**	**mm**
Section properties				
	Area	=	864	mm^2
	y_o	=	21.0	mm
	Ixx	=	467694	mm4
	Iyy	=	467694	mm4
	rxx	=	14	mm
	ryy	=	14	mm
Length of member about major axis	L z-z	=	**600**	mm
Length of member about minor axis	L y-y	=	**600**	mm
Slenderness ratio about major axis	K_zL_z/r_{xx}	=	36	
Slenderness ratio about minor axis	K_yL_y/r_{yy}	=	36	
Euler's buckling stress	fcc_z	=	1541	Mpa
	fcc_y	=	1541	
Non dimensional eff. Slenderness ratio	l_z	=	0.403	
	l_y	=	0.403	

inclination of tension field stress in web	f_z	=	0.63	
	f_y	=	0.63	
Stress reduction factor	c_z	=	0.90	
	c_y	=	0.90	
Design compressive stress(major) mPa	fcd_z	=	203.6	Mpa
Design compressive stress (minor) mPa	fcd_y	=	203.6	Mpa
Design compressive stress mPa		=	203.6	Mpa
Capacity of the section		=	**175.9**	kN

5.23 ANSYS stress results and comparison of stresses

Element Designation	Member	Member length (mm)	Vonmises stress (Mpa) Max	location	at mid section	member force as per analysis	Actual area provided	stress induced as per calc.	Allow. stress as per calc.
Top chord									
A	ISA 75x6 B/B	1917	191.37	-	41.6	91.6	1728	53.01	152.20
B		1917	112.48	-	77.426	151.4	1728	87.62	152.20
C		1917	123.73	-	116.66	169.4	1728	98.03	152.20
D		1917	148.84	-	125.79	185.7	1728	107.47	152.20
Bottom chord									
E	ISA 75x6 B/B	1917	66.587	-	59.199	171.8	1776	96.73	152.20
F		1917	102.84	-	80.795	179.6	1776	101.13	152.20
G		1917	135.4	-	113.51	179.6	1776	101.13	152.20
H		1917	121.34	-	123.99	138.1	1776	77.76	152.20
Web chord inclined at support									
I	ISA 75x6	1131	285.02	end	54.7	18.93	864	21.91	178.00
Web chord inclined at center									
I	ISA 75x6	1131	245.94	end	23.57	19.98	864	23.13	178.00
Web chord vertical									
I	ISA 75x6	600	321.9	end	25.116	25.96	864	30.05	178.00

5.24 Design of fillet weld joint for web chord of ISA 75x75x6

Member tensile force = 130 kN (tensile capacity of the section)

Properties of member ISA 75x75x6

A = 864 mm^2

tf = 6mm, Cy = 21.0mm

The CG of weld length should coincide with that of the angle.

The weld length ratio need to be : $Cy/(75 - Cy) = 21.0/54.0$

= 0.40

Weld length required at connected leg = 261.50 x 0.4

= 104.6mm

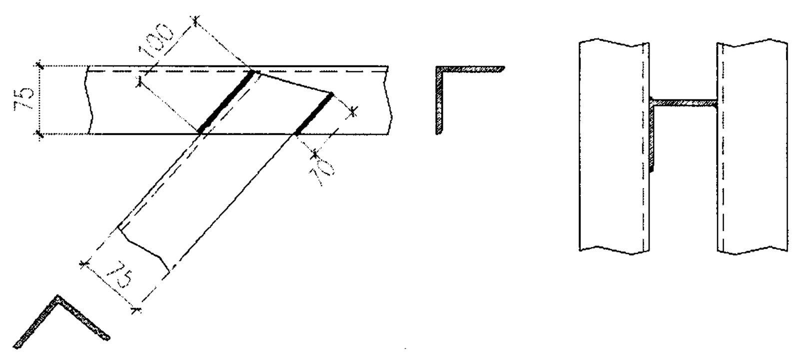
Weld length on one edge (out standing leg) = 52.3+18 say 70mm

Weld length on one edge (connected leg) = 261.50 x 0.6 / 2

= 78.50mm

= 78.50+18

= 96.5 say 100mm



Results & Discussion

Chapter 6

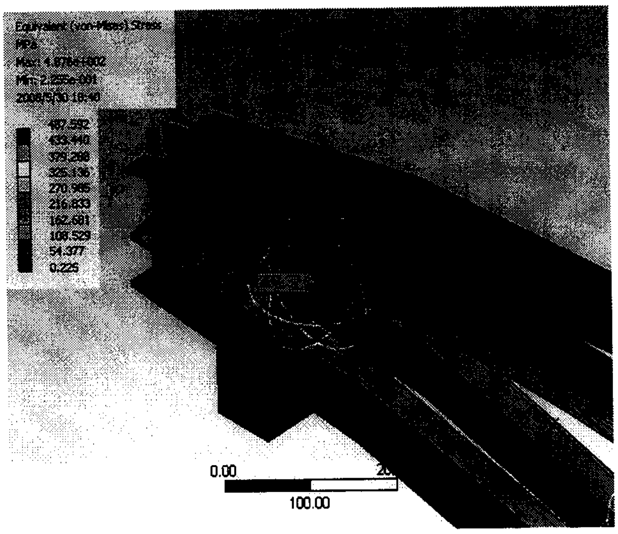
Results and Discussion

6.1 Truss with eccentric connection

When a web member is directly welded to chord members, the central lines of chord members and web members will not meet concentric with a result eccentric connection exists, eventually the gusset plates could have been avoided and due to its expense. However in the absence of gusset plate the eccentricity will result in a moment, the web member is subjected to three types of stresses namely

1) Major axial stress,
2) Bending tension and
3) Warping tension

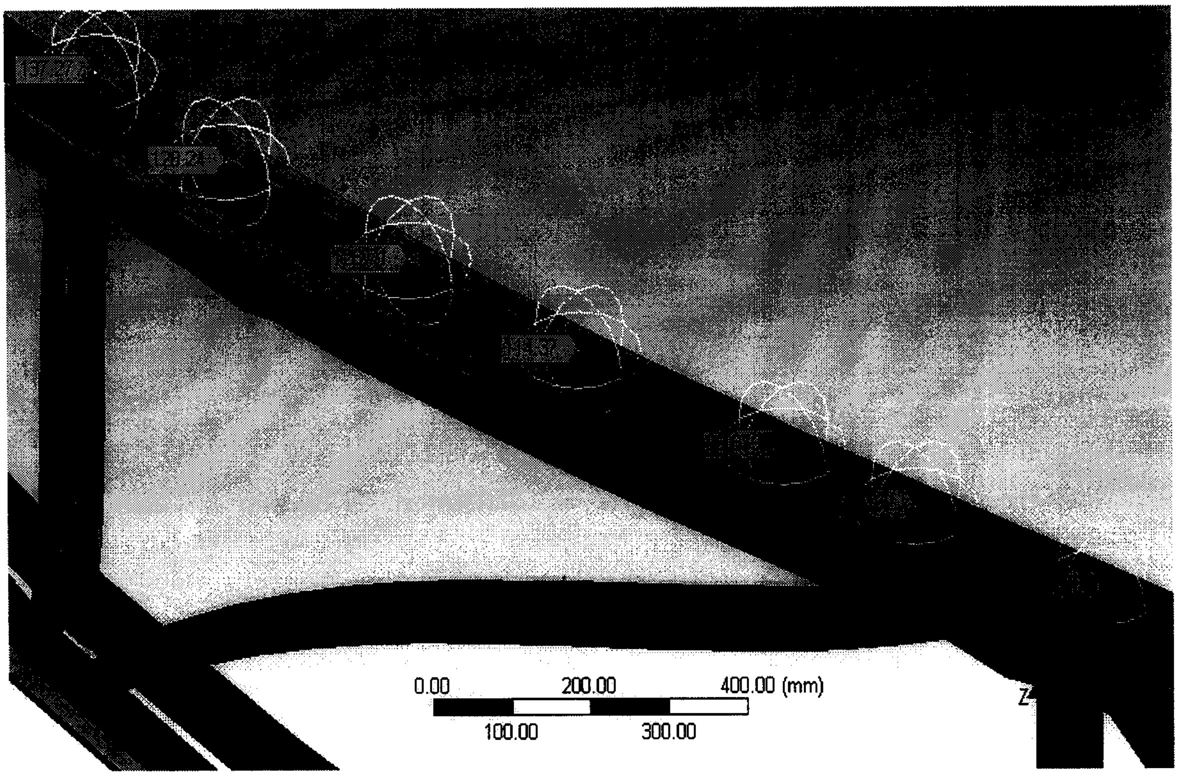
due to bending of the directly connected flange. The resulting stresses are combined and equivalent vonmises (failure) stresses. When main chord and web member sizes are equal and they are inclined greater than 30^0, all joint stresses are less than the failure stress. However the resulting stress 25-30% higher than the allowable stress (165mpa), but less than the yield stress of 250 mpa.



END JOINT

INTERIOR JOINT

6.2 In plane distortion effects in the web chords.

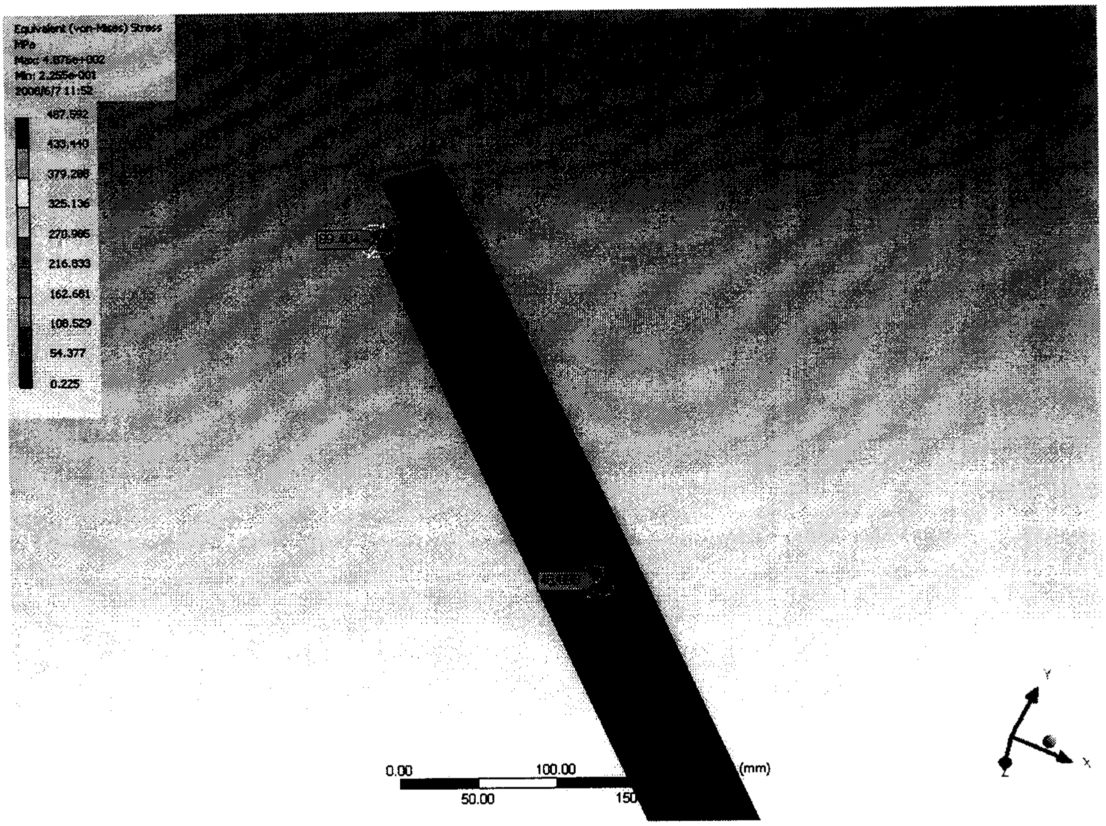


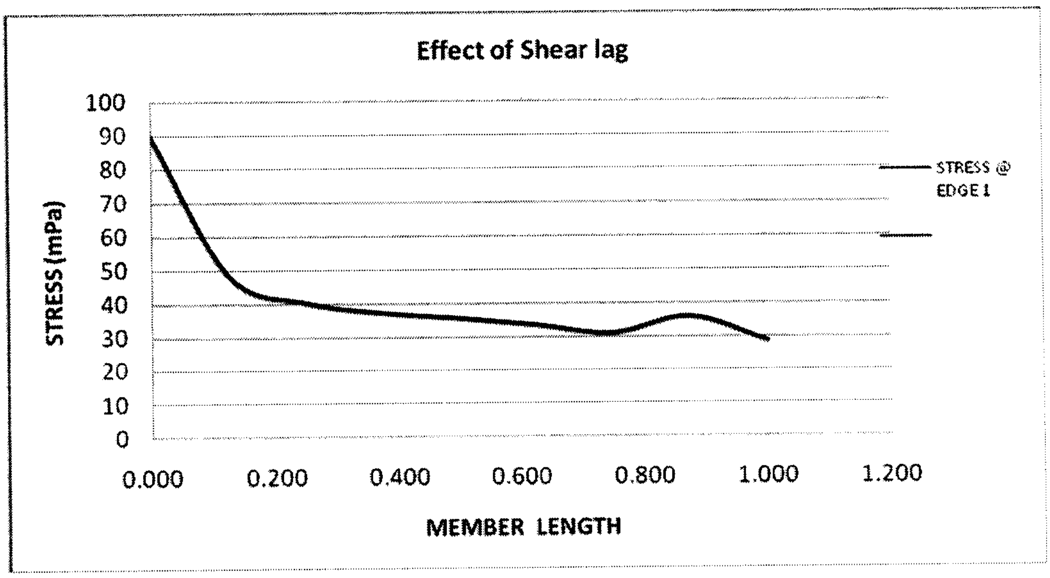
Web chord members are inserted in to the main chord members and welded firmly around the contacted edges. If the member is subjected to compressive force and due to its end fixity, the bending of the member takes place about the minor axis in the plane of truss (in plane bending). The distortion of joint also occurs in the same plane due to connection eccentricity. The member is relatively stiff in the other direction. Since the truss is laterally restrained by means of purlins, out plane bending is avoided.

If the load is applied just away from the symmetrical line of the truss the web member experiences the out plane bending stress.

Generally truss members are analyzed by assuming the ends are pinned, the effective length is taken as 1.0 L for the design of members, but from this analysis the effective length factor may be taken as 0.5, this is due to the fixity at the ends by means of weld (IS 800-2007 recommends that in in plane direction the effective length taken as 0.7-1.0 L).

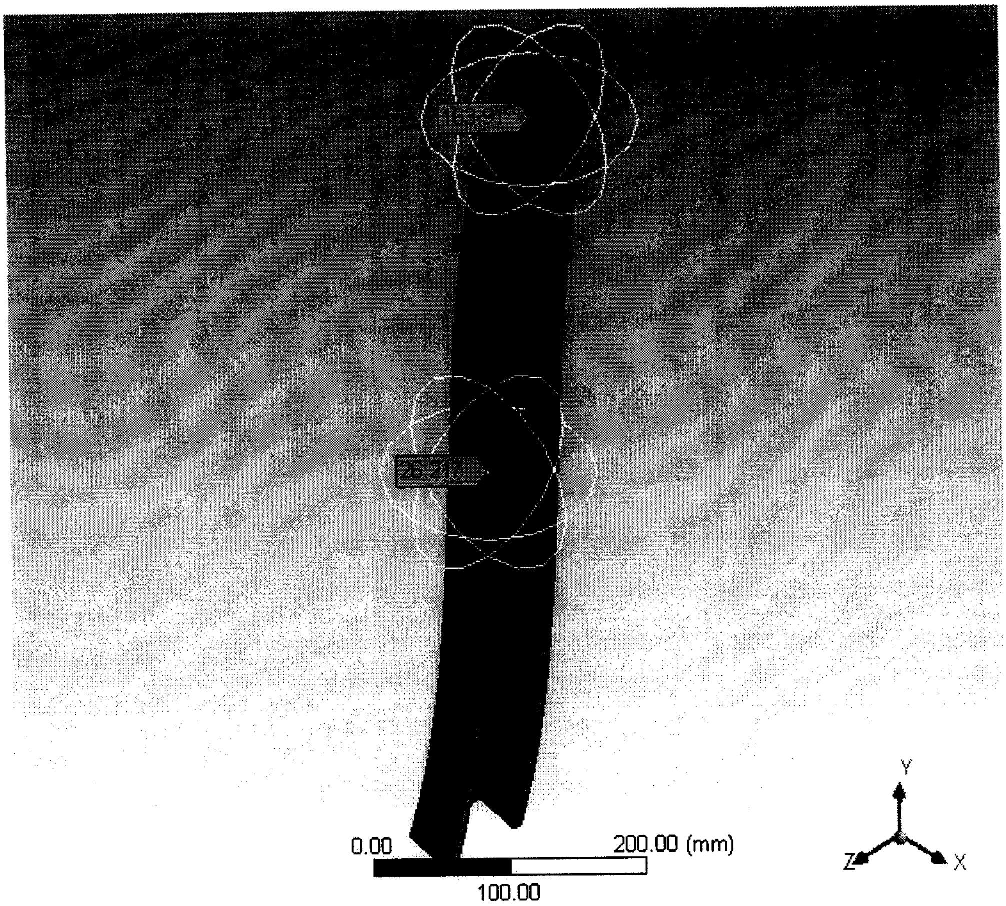
6.3 Effect of shear lag in typical web chord of truss





The shear lag is found to be a distance of 0.2L from the ends of the tension member in the connected faces. The effective area in resisting tension may be less in that region.

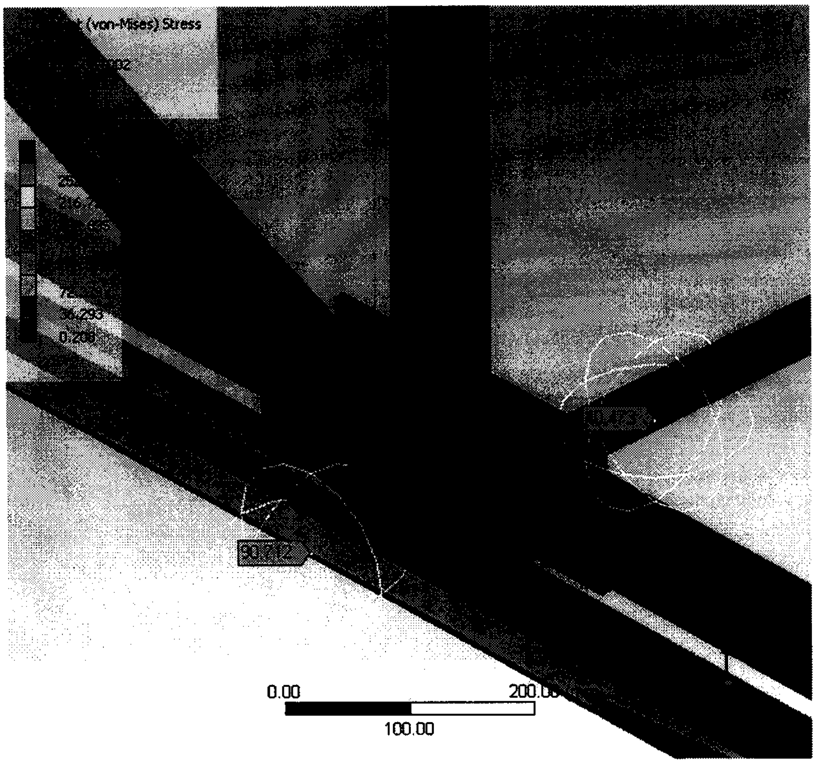
6.4 Buckling & curvature effects

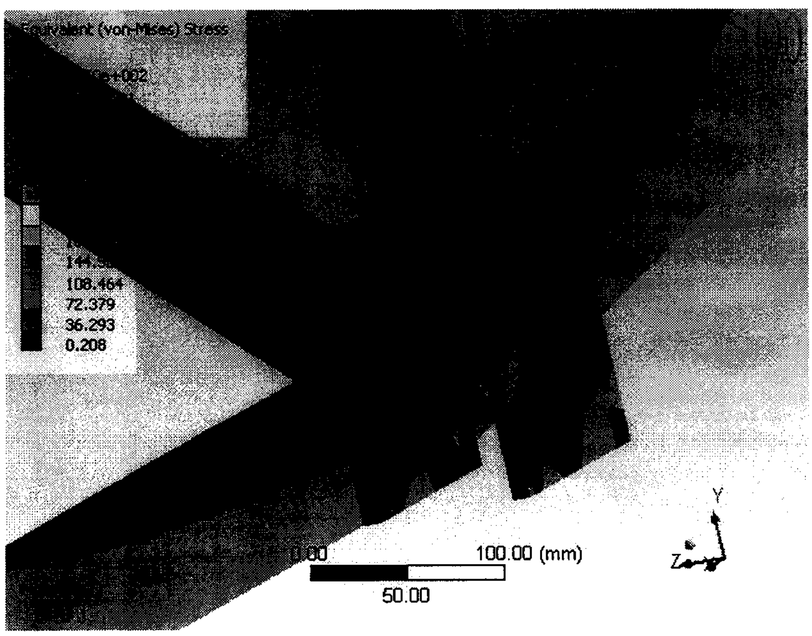


In pin jointed concentric connection trusses, the web members are subjected to maximum stress at mid section of the length due to buckling in single curvature. Here stresses are increasing towards the mid section and minimum at the ends. The capacity is gouverned by single curvature buckling effect.

But in the case of eccentrically connected directly welded truss connections, from the ANSYS finite element software analysis it is found to be most of the web members are influenced by double curvature buckling effect due to end fixity. Due to this behavior, slenderness ratio of member gets highly reduced which resulting the member subjectd to comparatively less combined stresses at the mid section and higher value at the ends. This is found to be advantageous for taking higher panel loads relatively with lesser cross section.

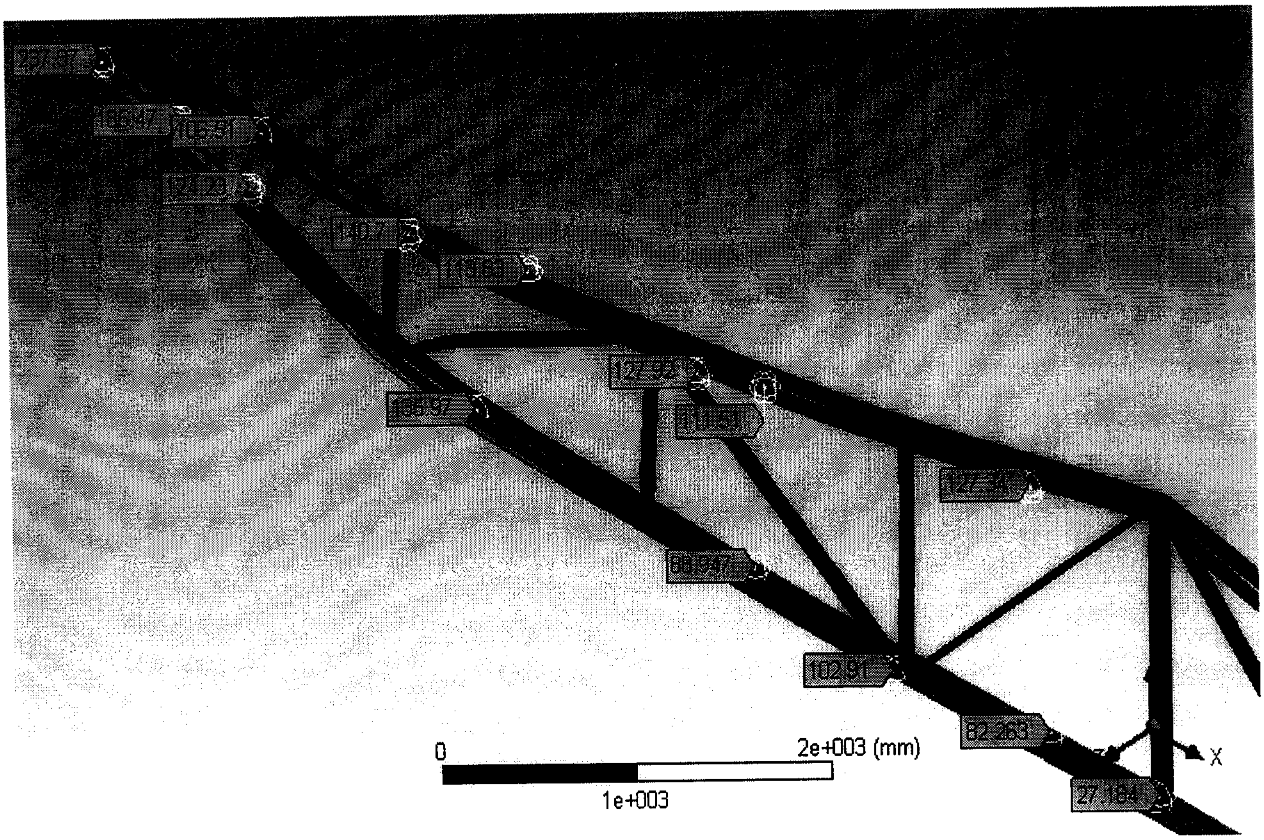
6.5 Gusset plates





Gusset plates in concentrically connected truss are subjected to highly complicated stresses. There is no standard methodology or method of analysis for finding the internal stresses of the gusset plates. The size and thickness are arrived arbitrarily and it is still complicated for the analysis and design where number of members are meeting at a particular joint. From the past observations it is very clear that the failure have been occurred due to the failure of gusset plates. Gusset plates increases the cost of the truss by 5-10% and also weld consumables twice.

6.6 Beam action of top chords

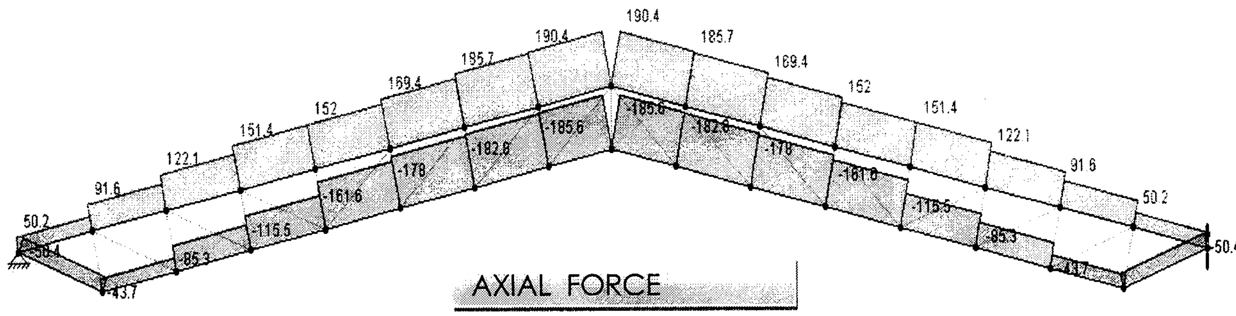


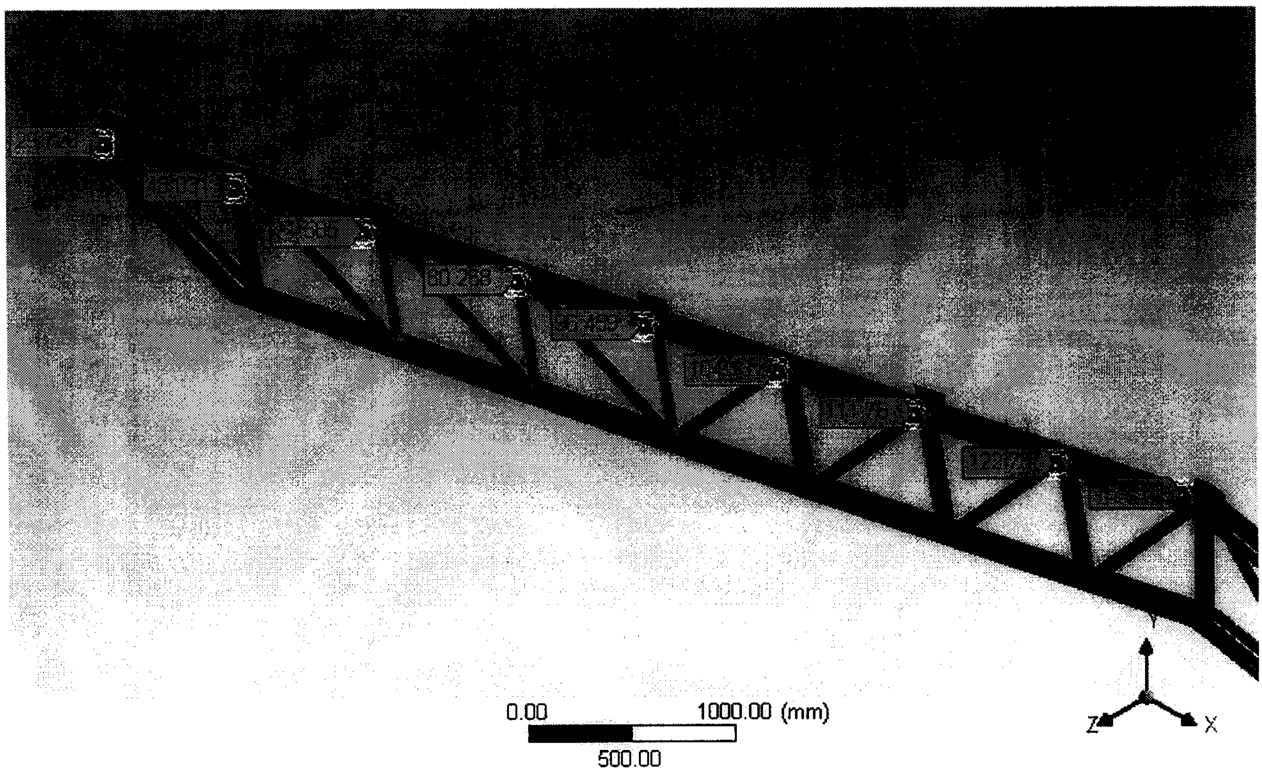
The vonmises (failure) stresses have been marked in the top & bottom chords. From the analysis it has been found that every panel point of the truss is act as a support and the top chord is act as a continuous beam over the panel points. due to the local eccentricity and joint distortion the combined stresses (axial & bending) are developed.

From the truss configuration it has been found that the stress is minimum in the main chords at the centre due to larger depth and higher stress at the end due to minimum depth.

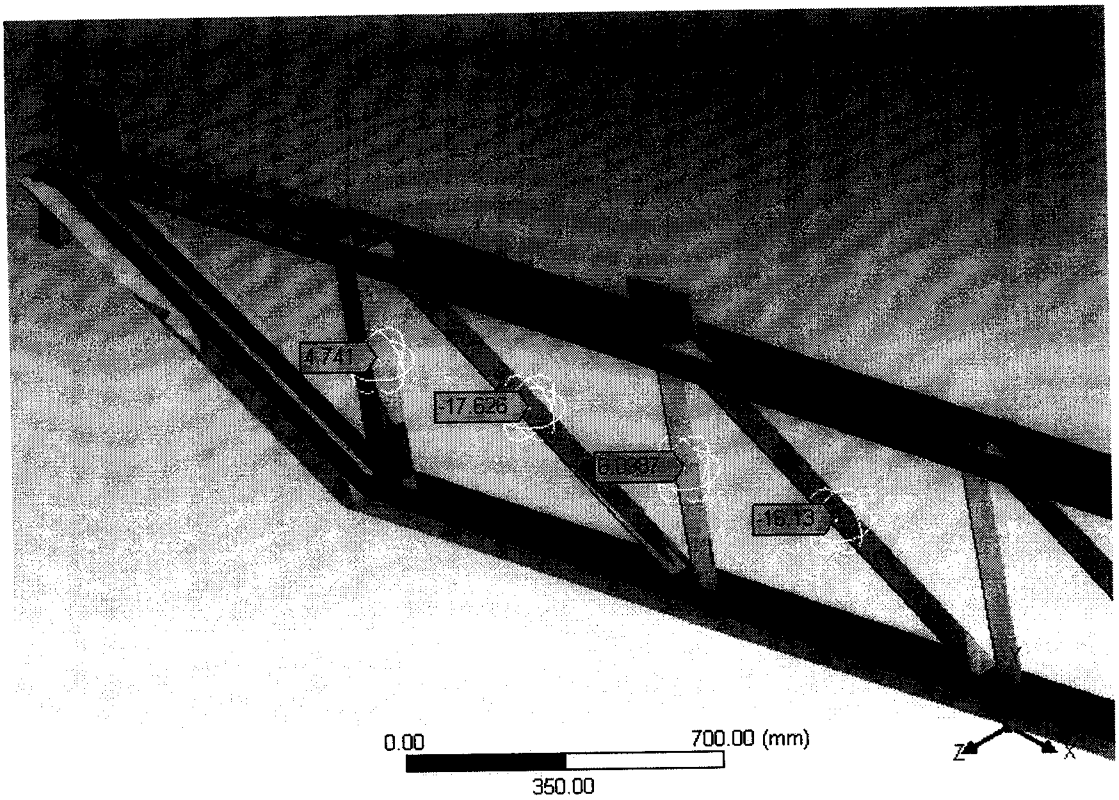
Since the loads are applied even with a minimum eccentricity at the panel points and also due to connection eccentricity of the web members the effect of local bucking is minimum at the mid & end section of the members which resulting the combined axial & bending stresses are within the allowable limit.

6.7 Behavior of lattice





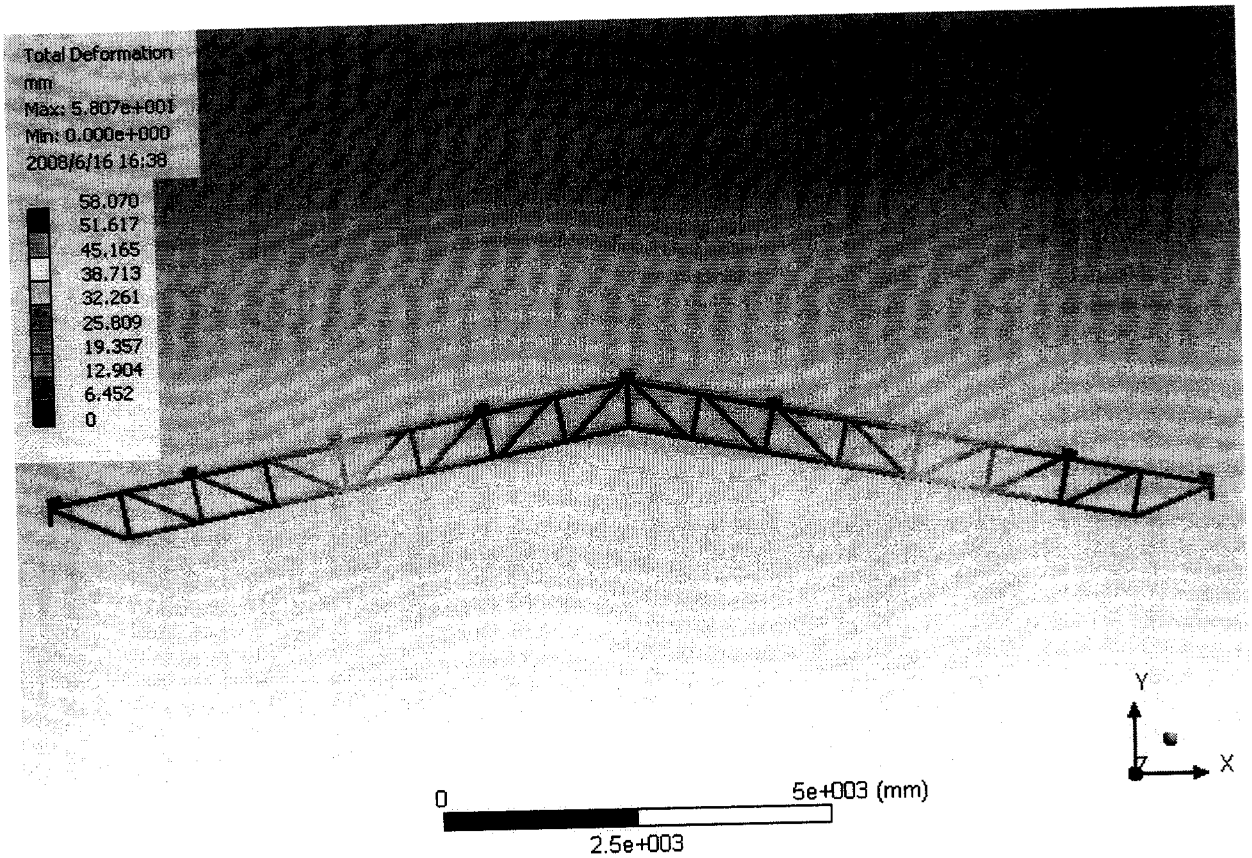
The lattice structure is behaving like a simply supported beam and hence the von mises stress variation is found to be maximum at the center chords of truss and gradually reduced to minimum at the end chords, the calculated depth of lattice is given such that the combined stress is within the allowable limit.(150 mpa).



Shear force is transferred as tensile or compressive force for all internal web chords. Each member should be checked for both tension & compression when subjected to reversal of loads.

From ANSYS, Deflection of the lattice is found to be 58mm which marginally exceeds the allowable limit (span / 325 = 46mm). Due to this exceeded deflection at the centre of the truss no joint distortion has been found. Hence due to connection eccentricity no secondary stresses are formed in the members near the joints. (ref fig (1) & (2)). The lattice depth may be increased to reduce the overall deflection.

The deflection of the lattice can be reduced and the lattice members can be optimized if both ends of the lattice is pinned. But the lattice will behave a two pinned arch and it possesses a lateral thrust to the supporting member. And hence the supporting member whether reinforced concrete column or steel member should have sufficient stiffness to resist such lateral thrust.


Total Deformation
mm
Max: 5.807e+001
Min: 0.000e+000
2008/6/16 16:38
58.070
51.617
45.165
38.713
32.261
25.809
19.357
12.904
6.452
0
0
5e+003 (mm)
2.5e+003
Y
X

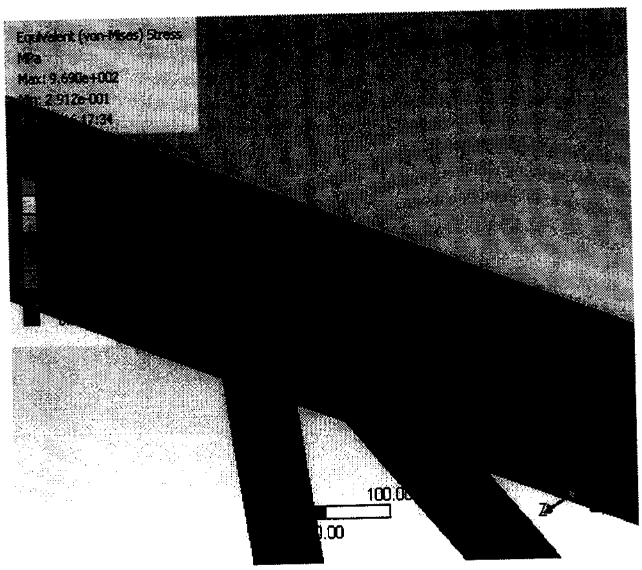
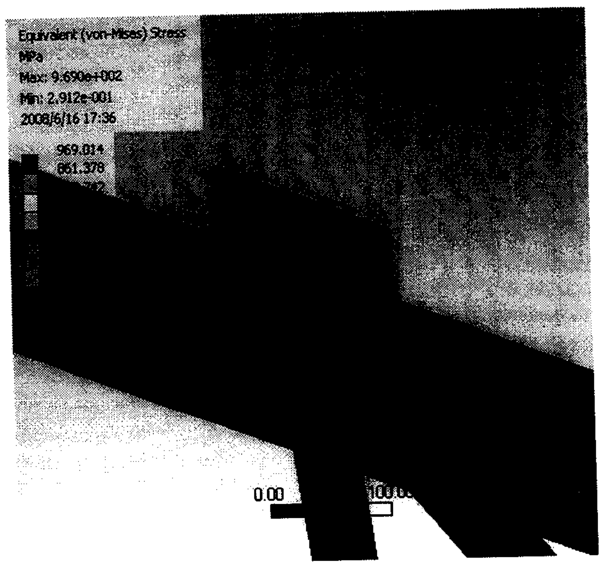
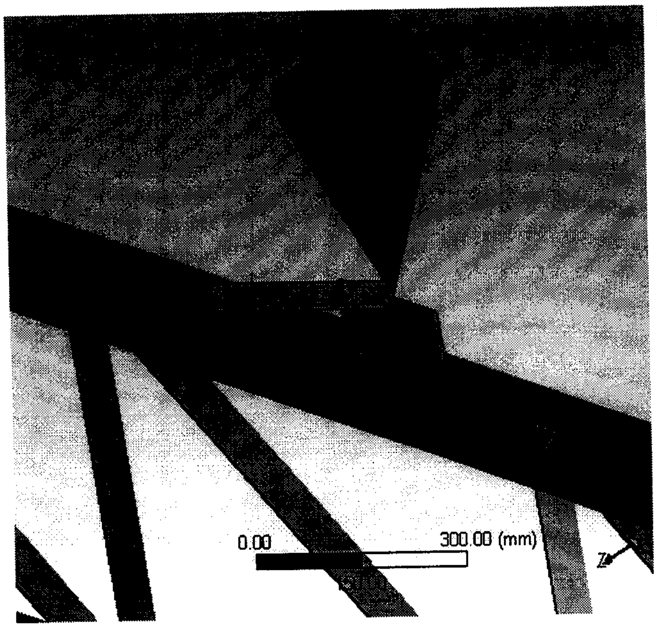
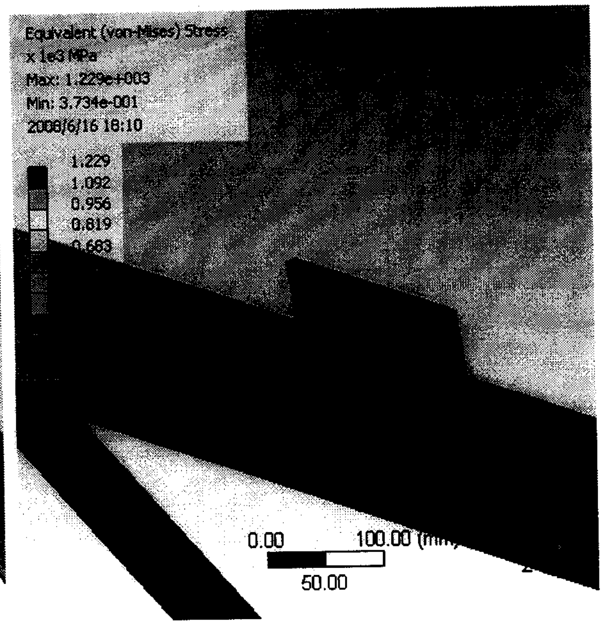

Equivalent (von-Mises) Stress
MPa
Max: 9.690e+002

FIG (1)


Equivalent (von-Mises) Stress
MPa
Max: 9.690e+002
Min: 2.912e-001
2008/6/16 17:36
969.014
861.378
0.00

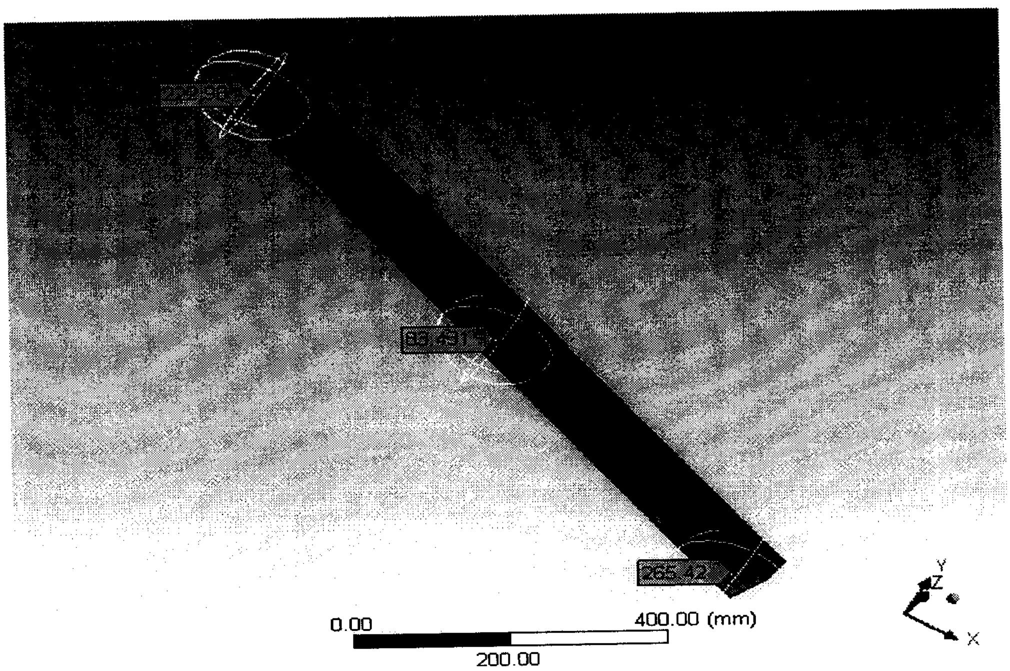
FIG

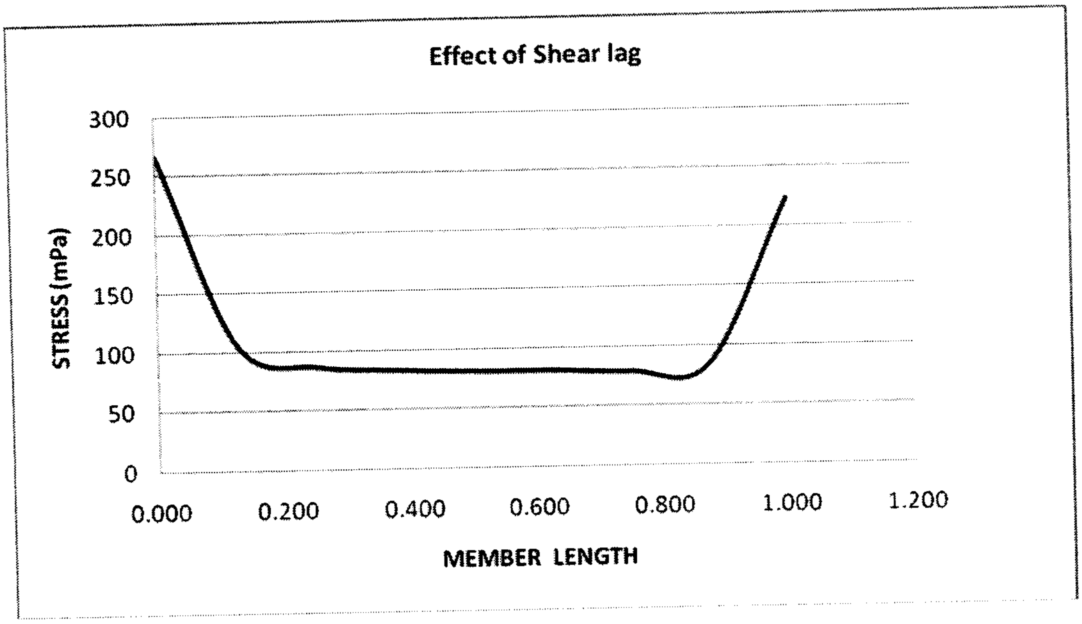




Application of nodal loads in between the panel points doesn't influences any increase in the secondary stresses, since the main and web chords integrally act as a beam. hence the main chords should be designed as beam column.

6.8 Effect of shear lag in typical web chord of lattice





Shear lag effect influences up to a length of 0.15L from both the ends of the tension member.

Conclusions

Conclusions

1. Elimination of gusset plates will result a considerable economy in the total cost of the roof truss. i.e. 5 to 10% of the cost of truss can be omitted.
2. If web member axis is inclined around 30 to 60^0, the secondary stresses due to bending of web member is only margined 25-30%, well within the yield strength of material.
3. Even when yield takes place, the joint will rotate with very little increase in deflection of the truss nodes.
4. High secondary stresses exist only in some of the members and then only in the extreme fibers at the ends of the members.
5. With the low value of basic allowable stresses used in design, such high localized stresses do not become a problem unless repeated often enough, in which case fatique failure may occur.
6. When it is desired to limit or to reduce the secondary stresses resulting from truss distortion, the width of the member in the plane of bending should be reduced relative to the length of the members. So that shear lag is reduced.
7. Selection of configuration and truss type is important to minimize the secondary stresses. Pratt or Howe trusses may be avoided, instead sub divided fink truss is preferred.
8. End vertical leg connecting the truss and base plate should be kept as slender member. (this is for the reason, generally trusses are designed with assumptions that one end is pinned and the other end is roller, this may be achieved practically by providing slender leg at the ends to allow lateral deformation to avoid higher stresses in the main leg)
9. The simplicity of welded joints and their comparative rigidity has often resulted in the omission of gusset plates when they are not required for strength purposes.
10. Eccentricity of connection in trusses having better performance in resisting seismic forces.

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2. Design of Steel Structures : by *Boris Breslar, T.Y. Lin, John B. Scalzi*
3. Steel Designers Manual prepared for the Constructional Steel research & development organization.
4. Design of Steel Structures : by *E.H. Gaylord, and C.N. Gaylord.*
5. Analysis & Behavior of connections by *E.H. Gaylord*

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CERTIFICATE

This is to certify that V. BALAJI RANGANATHAN

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