STRUCTURAL BEHAVIOUR OF TRUSSED-COLUMN BUILT UP USING THIN-WALLED STEEL PIPE

By

MUHAMMAD NAJIB BIN KAMARUDIN
6005

Dissertation Report submitted in partial fulfilment of
the requirements for the
Bachelor of Engineering (Hons)
(Civil Engineering)

July 2007

Universiti Teknologi PETRONAS,
Bandar Seri Iskandar,
31750 Tronoh,
Perak Darul Ridzuan.

CERTIFICATION OF APPROVAL

Structural Behaviour of Truss-Column Built Up Using Thin-Walled Steel Pipe

by

Muhammad Najib bin Kamarudin (6005)

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfillment of the requirements for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

(Assoc Prof. Dr. Narayanan Sambu Potty)

UNIVERSITI TEKNOLOGI PETRONAS
TRONOH, PERAK
July 2007

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

MUHAMMAD NAJIB BIN KAMARUDIN

ABSTRACT

The availability of high strength steels leads to the use of metal steel pipe in truss-column structure. The use of trussed column is not only restricted to beam support, but also extends to various uses such as road sign, transportation of petroleum and gas product, huge exhibition and many more. However, the use of thin walled steel pipes in composite built-up truss-columns gives a rise to a certain major problems such as local buckling that would appreciably reduce the strength and ductility performance of the members. This paper describes an experimental study on the compressive strengths of built-up truss-column using steel pipe sections. As the use of trussed column have gained momentum due to its strength and aesthetic value, two types of trussed column were analyzed, namely the triangular and rectangular trussed column, with different loading conditions and depth variation. Investigated is the static structural behavior of such columns when loaded by purely axial compressive concentrated forces acting at the supports. Geometric and material nonlinear analyses are performed to investigate the critical local and post-local buckling strengths of steel plates under compression and in-plane bending. Initial geometric imperfections and residual stresses presented in steel pipes, material yielding and strain hardening are taken into account in the nonlinear analysis. Result were tabulated and interpreted according to several parameters, such as weight, compression/tension ratio, magnitude of forces and its distribution, along with deflection of the trussed column. Comparison among the two trussed column (triangular and rectangular) were presented and discussed.

TABLE OF CONTENT

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 PROJECT BACKGROUND

The need to predict the behavior of structures up to and beyond failure of individual structural parts, has been the main driving force behind the development of new structural design and analysis methods in recent years. As the result, the practical usage of cold-formed steel section has grown rapidly and there has been an evolution in the application and procedure for the design process.

From the recent research, the application of steel pipe can be expandable consider to its materials and physical characteristics. Ether than just used on water supply distribution systems, people has studied on applying these materials on the structural development such as in truss structures.

Steel pipe has been used for water lines in the United State since the early 1850s. The pipe was first manufactured by rolling steel sheets or plates into shape and riveting the seams. Pipe wall thicknesses could be readily varied to fit the different pressure heads of a pipeline profile. As the research expanded, both riveting and lock bar methods gradually were replaced by welding. Steel pipe has been found satisfactory for many applications such as aqueducts, supply lines, transmission mains, penstocks, treatment-plant piping, self-supporting spans, forces mains, circulating-water lines and underwater crossings.

The availability of high strength steels leads to the use of thin-walled steel pipes in truss-column structure. However, the use of thin-walled steel pipes are mostly exposed to local buckling that would appreciably reduce the strength and ductility performance of the members. This paper studies the structural behavior of steel pipes by applied the compression forces at the end column.

Geometric and material nonlinear analyses are performed to investigate the critical local and post-local buckling strengths of steel pipes under compression and in-plane bending. Initial geometric imperfections and residual stresses presented in steel pipes, material yielding and strain hardening are taken into account in the nonlinear analysis.

Based on the results obtained from the test, a set of analysis are proposed for determining the structural behavior and ultimate strengths of the steel pipes. The accuracy of the proposed analysis is established by comparisons with available solutions.

1.2 PROBLEM STATEMENT

Steel sections are characterized by the thinness of the material and this result in a number of failure mode or behavior characteristic which, while they may be present in hot-rolled construction, one far less prominent. However, despite popular use of steel materials in constructions, the applications have the certain problems and limitations on the design process. In the process of design the column structure, the designer must carefully considered the physical characteristic such as its ability to sustain the compression force and its stability.

In this research, the truss-column was tested by applying a compression forces at both of the end of the column. As the result from the applied forces, the column was subjected to both internal and external pressures that can cause structure stability loss, leading to the flattening of an initially perfect thin-walled elastic body. One of the major effects from the compression forces is the phenomena of buckling.

Buckling is a failure mode characterized by a sudden failure of a structural member that is subjected to high compressive stresses where the actual compressive stresses at failure are smaller than the ultimate compressive stresses that the material is capable of withstanding. This mode of failure is also described as failure due to elastic instability. Mathematical analysis of buckling makes use of an eccentricity that introduces a moment which does not form part of the primary forces to which the member is subjected.

The primary effect of local buckling is to reduce the member stiffnesses to overall flexure and torsion. Consequently, the overall bifurcation load can be calculated by using the stiffnesses of the locally buckled cross-section rather than the stiffness of the undistorted cross-section. The result has been used widely to determine the flexural buckling of locally buckled doubly symmetric columns.

The applications of Carbon Steel Pipe in structure are still new and yet to be developing from time to time. Since erosion/corrosion from high temperature and pressure can cause wall thinning, it is important to evaluate the strength of the pipes with local wall thinning to maintain the integrity of the materials. The local wall thinning is simulated as erosion/corrosion metal loss. The eroded area of the wall thinning is subjected to tensile or compressive stress by applied bending moment. The deformations or fracture behaviors at maximum moments are found to be classified into three types. When the eroded area is subjected to tensile stress, ovalization or crack initiation/growth occurs at the maximum moment. When an eroded area is subjected to compressive stress, ovalization or local buckling occurs. The occurrence of ovalization, crack initiation/growth, or local buckling depends on the initial size of local wall thinning.

1.4 RESEARCH OBJECTIVE

In general, the objective of this research is to:

i. To study the most preferable truss column between rectangular and triangular truss column
ii. To understand the distribution of forces in the trussed column
iii. To find out what governs the design of the elements of the trussed column
iv. To compare the trussed column in terms of economic feasibility

1.5 SCOPE OF WORK

The scope of this study will be limited to the weight of the trussed column, magnitude and distribution of forces in the trussed column and deflection minimization, with the trussed column subjected to concentrated load. The behavior due to difference in depth of the trussed column and comparison between different trussed column (triangular and rectangular) will also be presented.

The trussed column used in the analysis will be of 6 bays, each measuring 0.5m wide and 3m, 4.5m and 6m height. Connection between elements is analyzed for welded connection.

CHAPTER 2

LITERATURE REVIEW

2.1 Application of Hollow Sections in Steel Structures

2.1.1 Introduction

Mankind has learnt the application of hollow tubular members as structural elements from nature. Many examples in nature show not only the use of a hollow cylinder to transmit a fluid, but also the excellent properties of the tubular shape with regard to loading in compression, torsion and bending in all directions. These advantages were quickly understood by our ancestors, when in their hands the bamboo pole became a light building component as well as a pipe for the supply of drinking water or for irrigation.

During the development of steel production and the manufacture of classical hot-rolled open sections such as I-, L- and U-profiles, the first methods for the fabrication of tubes or circular hollow sections were developed in the nineteenth century. The production of rectangular hollow sections was not started, however, until 1952 (by Stewarts & Lloyds in the United Kingdom).

Circular shaped tubes are made either from a solid lump of steel, producing seamless tubes, or from a flat strip, giving welded tubes. There is no fundamental difference between the production process for a circular section tube intended for use as a pipe and that for a similar hollow section intended for a structural use.

The so-called "form" tubes - square, rectangular, hexagonal or octagonal - are obtained by deforming, either hot or cold, a round tube as a blank. The circular blank tube is passed through forming roll cages working continuously and outwards only. This process gives the blank, usually after passing over several sets of rolls, the required shape, which is normally square and rectangular.

The selection of a particular profile in a steel structure is governed by many factors. It involves a comparison of the pros and cons with regard to mechanical properties, unit material costs and the costs of fabrication, erection and maintenance. The experiences of architects, designers and fabricators also affect the choice. It is therefore very important that those involved should understand the behavior of hollow sections and their connections.

2.1.2 Mechanical and Geometrical Properties of Hollow Sections

Hollow sections as steel profiles are not only in competition with concrete, but also they may substitute for other steel profiles due to their superiority with regard to strength and stability. The mechanical and geometrical properties of the hollow section indicate under which loadings a saving in material can be obtained.

2.1.3 Mechanical Properties

The material grades in which structural hollow sections are delivered according to Eurocode 3 are given in Table 2.1.

Table 2.1 Steel grades for structural steels

Steel grade	Minimum yield strength f_y (N/mm^2)	Tensile strength f_u (N/mm^2)	Minimum percentage elongation $L_o = 5{,}65\sqrt{S_o}$	
			Longitudinal	Transverse
S235	235	340-470	26	24
S275	275	370-540	24	22
S355	355	470-630	22	20
Fe 460*	460	550-720	17	15

In cold-finished sections, the increase in yield stress due to cold forming can be taken into account. Table 2.2 shows the recommendations and formulae for applying this increment.

Table 2.2 Increase of yield strength due to cold forming of hollow sections

Average yield strength: The average yield strength f_{ya} may be determined from full size section tests or as follows: $\mathbf{f_{ya} = f_{yb} + (k \cdot n \cdot t^2/A) \cdot (f_u - f_{yb})}$ where f_{yb}, f_u is the specified tensile yield strength and ultimate tensile strength of the basic material (N/mm^2)   t is the material thickness (mm)   A is the gross cross-section area (mm2)   k is the coefficient depending on the type of forming (k = 7 for cold rolling)   n is the number of 90° bends in the section with an internal radius <5t (fractions of 90° bends should be counted as fractions of n) f_{ya} should not exceed f_u or 1,2 f_{yb} The increase in yield strength due to cold working should not be utilized for members which are annealed* or subject to heating over a long length with a high heat input after forming, which may produce softening.
Basic material: Basic material is the flat hot rolled sheet material out of which sections are made by cold forming.

To allow welding in the region of the corners of cold finished rectangular hollow sections, the requirements given in Table 2.3 should be met.

Table 2.3 Minimum corner radii of cold finished RHS

Steel grade	Wall thickness, t (mm)	minimum $\frac{r}{t}$
S235	$12 < t \leq 16$	3,0
S275	$8 < t \leq 12$	2,0
S355	$6 < t \leq 8$	1,5
	$t \leq 6$	1,0

2.1.4 Geometrical Properties

The selection of hollow sections depends on their geometrical properties and thus on the member resistances for particular loading cases. Tolerances are in general lower than for open sections.

2.1.5 Tension Loading

The design strength of a member under a tensile loading depends on the cross-sectional area and the design yield stress, and is independent of the sectional shape. In principle, there is no advantage or disadvantage in using hollow sections from the point of view of the amount of material required.

2.1.6 Compression Loading

For centrally loaded members in compression, the critical buckling load depends on the slenderness λ and the section shape.

The slenderness λ depends on the buckling length l and the radius of gyration r.

$\lambda = l/r$

The radius of gyration of hollow sections, (in relation to the member mass) is generally much higher than for that about the weak axis of open sections. For a given load this difference results in a lower slenderness for hollow sections and thus a lower mass when compared with open sections.

The buckling behaviour is influenced by initial eccentricities, straightness and geometrical tolerances, residual stresses, inhomogeneity of the steel and the stress-strain relationship.

Based on an extensive investigation by the European Convention for Construction Steelwork, "European buckling curves" (Figure 2.1) are established for various steel sections including hollow sections.

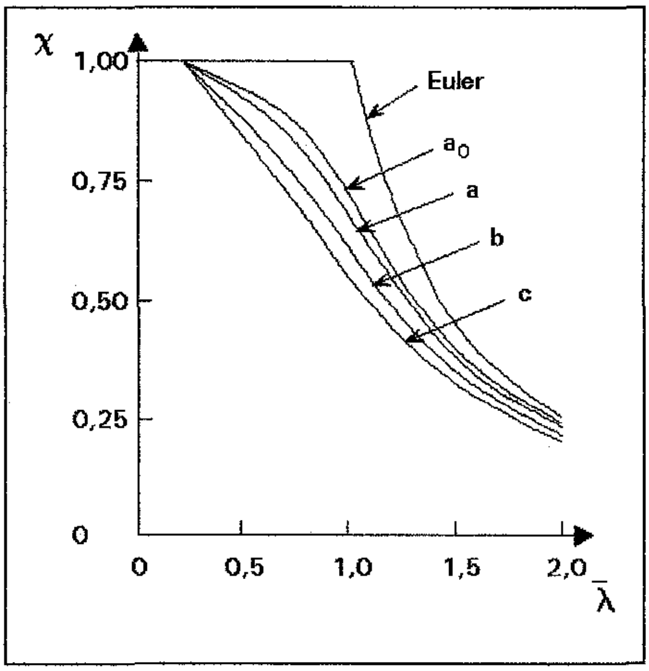


Figure 2.1 Buckling Curve

The reduction factor χ shown in Figure 2.1 is the ratio of the design buckling resistance $N_{b,Rd}$ to the axial plastic resistance $N_{pl,Rd}$:

$$\chi = \frac{N_{b,Rd}}{N_{pl,Rd}} = \frac{f_{b,Rd}}{f_{yd}}$$

where

$f_{b,Rd} = \frac{N_{b,Rd}}{A}$ (the design buckling stress)

$f_{yd} = f_y / \gamma_M$ (the design yield strength)

γ_M is the partial safety factor

The non-dimensional slenderness $\bar{\lambda}$ is determined by $\bar{\lambda} = \lambda / \lambda_E$

where $\lambda_E = \pi \sqrt{\frac{E}{f_y}}$ (Euler slenderness).

The buckling curves for the hollow sections are classified according to Table 2.4.

Table 2.4 European buckling curves according to manufacturing processes

Cross-section	Manufacturing process	Buckling curves
Fig. T2.4-1	Hot forming	a
	Cold forming (f_{yb}* used)	b
	Cold forming (f_{ya}** used)	c

* f_{yb} = yield strength of the basic material

** f_{ya} = yield strength of the material after cold forming

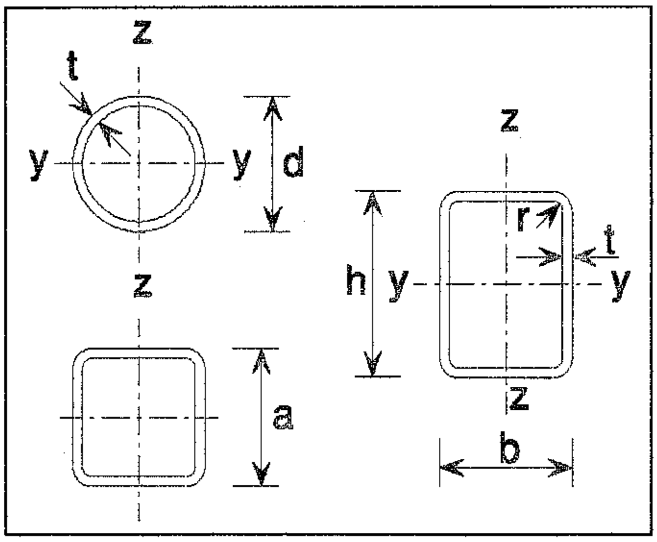


Figure T2.4-1 Cross-Section of Hollow Sections

Most open sections fall under curves "b" and "c". Consequently, for the case of buckling, the use of hot-formed hollow sections generally provides a considerable saving in mass.

Figure 2.2 shows a comparison between the required mass of open and hollow sections for a given load.

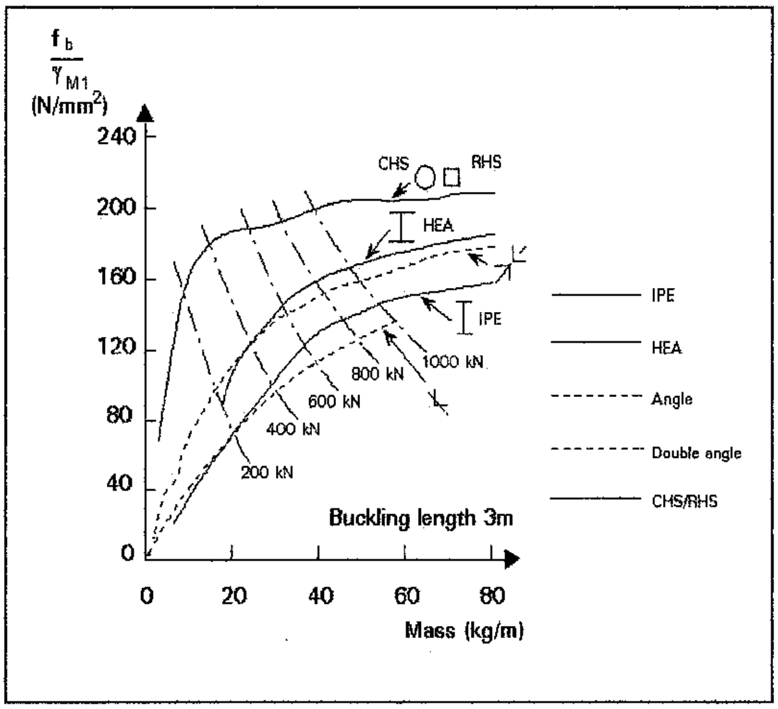


Figure 2.2 Comparison of the Masses of hollow and open sections under compression in relation to the loading

The overall buckling behaviour of hollow sections improves with increasing diameter or width-to-wall thickness ratio. However, this improvement is limited by local buckling. To prevent local buckling, d/t, or b/t limits are given in Eurocode 3 for plastic as well as elastic design (Table 2.5).

Table 2.5 Local buckling limits for hollow sections

Section	○	□
Plastic design sections	$\frac{d}{t} \leq 50\varepsilon^2$	$\frac{b}{t} \leq 33\varepsilon$
Compact sections	$\frac{d}{t} \leq 70\varepsilon^2$	$\frac{b}{t} \leq 38\varepsilon$
Elastic design sections	$\frac{d}{t} \leq 90\varepsilon^2$	$\frac{b}{t} \leq 42\varepsilon$
	Local buckling check for sections with larger d/t or b/t ratios	
$\varepsilon = \sqrt{235/f_y}$		

In the case of thin-walled sections, interaction between buckling and local buckling should be considered.

In addition to the improved buckling behaviour due to high radius of gyration and the enhanced design buckling curve, hollow sections can offer other advantages in lattice girders. Due to the torsional and bending stiffness of the members in combination with certain joint fixity, the effective buckling length of compression members can be reduced.

Laterally unsupported bottom chords of lattice girders have a reduced buckling length due to the improved torsional and bending stiffness of the tubular members. These factors make the use of hollow sections in girders even more favourable.

2.1.7 Torsion

Hollow sections, especially CHS have the most effective cross-section for resisting torsional moments, because the material is uniformly distributed about the polar axis. A comparison of open and hollow sections of nearly identical mass in Table 2.6 shows that the torsional constant of hollow sections is 200 to 300 times larger than that of open sections.

Table 2.6 Torsional strength of various sections

Section	Mass, kg/m	Torsion constant J
150 UC 23	23,4	48,7
200 UB 25	25,4	61,2
L 127 x 127 x 13	24,0	166
□ 127 x 127 x 6,3	23,9	11200
ϕ 168 x 6,3	25,3	21200

2.1.8 Bending

In general, the UB and UC sections are more economical under bending (I max larger than for hollow sections). Only in those cases in which the design stress in open sections is largely reduced by lateral buckling, can hollow sections offer an advantage. It can be shown by calculations that for circular hollow sections and for rectangular hollow sections with b/h >0.25, which are normally used, lateral instability is not critical. Hollow sections used for elements subjected to bending can be more economically calculated using plastic design.

2.1.9 Fatigue

The fatigue behavior of hollow section joints is influenced largely by the geometrical stress or strain concentration factor (SCF or SNCF).

A structure made of hollow sections should be designed and detailed so that the SCF or SNCF is low. In this way, economical design of hollow section joints is possible even under fatigue conditions, particularly when assessed in terms of low load coefficients for wind and wave, reduced mass and corrosion protection.

2.2 Fabrication and Erection

2.2.1 Aspects of Fabrication

After the Second World War, riveted tubular structures had many joints with gusset plates. In the last thirty years the ratio of labor to material costs has increased rapidly in industrialized countries. For this reason, more attention should be paid to detailing simple joints.

As far as possible, joints should be designed without stiffeners or gusset plates. This, however, means that the designer should consider the joint strength of non-reinforced joints at the preliminary design stage.

2.2.2 Welding

Welding is the most important jointing technique for hollow section structures. In general, the same welding procedures can be used as for open steel sections. Circular hollow sections can be joined with fillet welds if the diameter ratio between the sections to be connected does not exceed 0.33 and the weld gap does not exceed 3 mm. For larger ratios the weld can change smoothly from a fillet weld at the crown to a butt weld at the saddle, or a full butt weld over the entire perimeter can be used, see Figure 2.3.

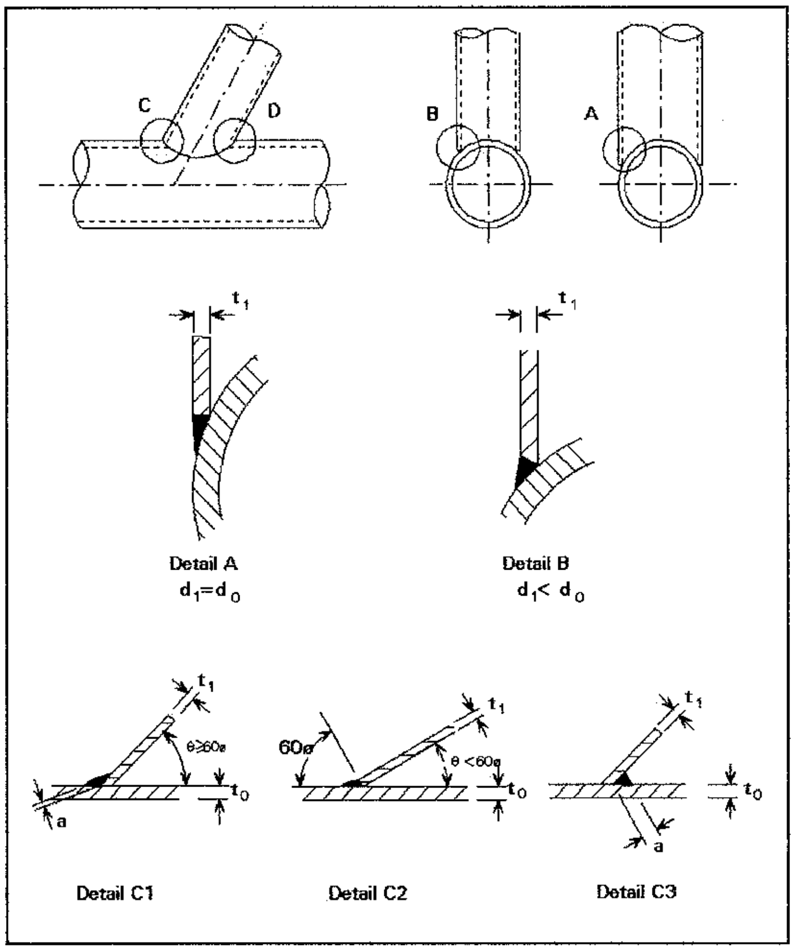


Fig.2.3 Weld detailed for a circular section joints

2.2.3 Bending

Bending operations for hollow sections are carried out in the hot or cold condition. It should be considered that the wall thickness of the hollow section at the external radius of curvature can be decreased, whereas at the inner side of the curvature wall crippling may occur. Further, care should be taken that the ovality of a tube which can occur, is kept as small as possible.

The minimum inside radii recommended in the U.K. for rectangular hollow sections are given in Table 2.7. The bending radii for circular hollow sections of up to 159 mm external diameter are recommended. The bending operation is commonly carried out by roller benders with three rollers.

Table 2.7 Minimum bending radii for Rectangular Hollow section (RHS)

Diameter (mm)	Thickness (mm)	Radius (r1)
20	2.6	5D
30	2.6	
40	2.6	
50	3.2	
60	4.0	
70	5.0	6D
80	5.0	
90	6.3	
100	6.3	
120	6.3	
150	10.0	
180	10.0	7D
200	10.0	
250	12.5	
300	16.0	
350	16.0	

2.3 Design Application

2.3.1 Column

The magnitude of the end moment determines the structural arrangement that is called for. It is always worthwhile to examine first the simplest solution with a single end plate without any stiffener, even if a fairly thick plate is required. If this simple solution is not appropriate, more complex arrangements with stiffeners can be envisaged.

Figure 2.4 shows the base of a single plate lattice column. Figure 2.5 shows an arrangement for connecting internal rain water down pipe at the foot of a hollow section column. Precautions should be taken to protect the inside of the column against corrosion. The hollow section can be galvanised or a seal can be made at the head and the foot of the column.

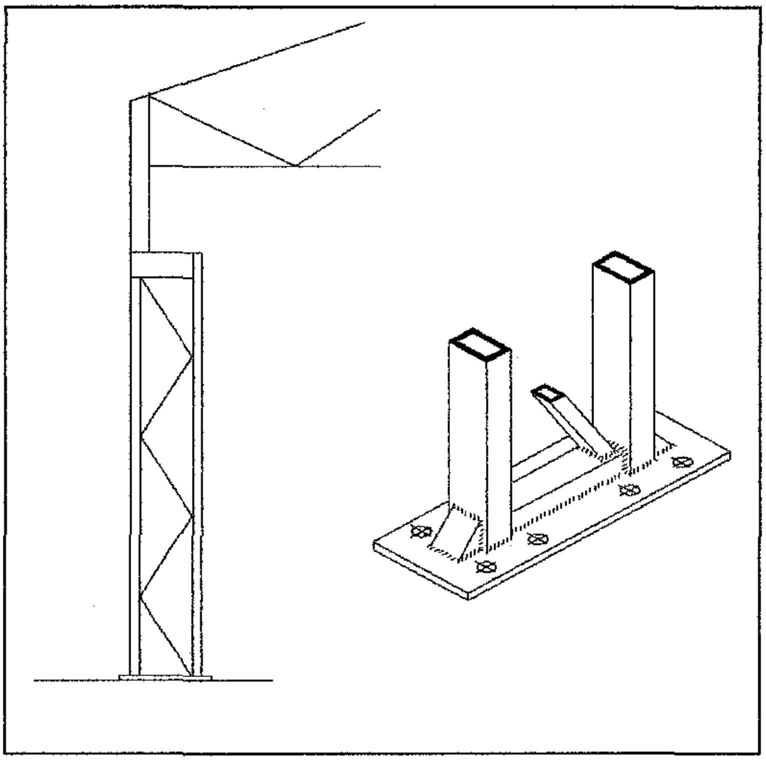


Fig. 2.4 Single Plate Lattice Column

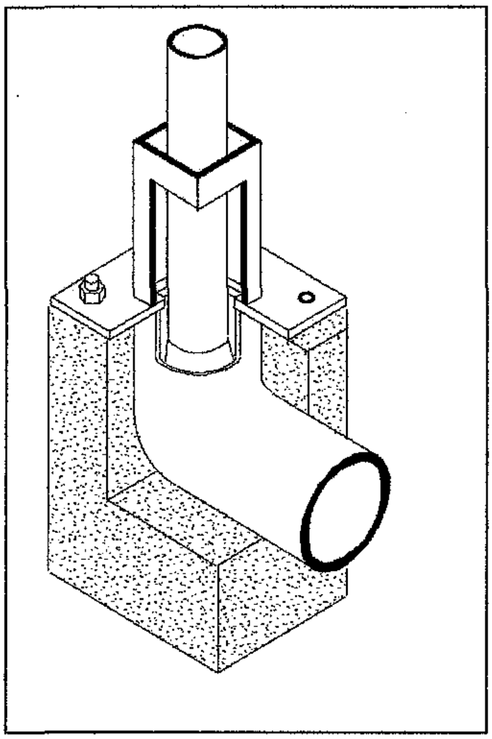


Fig. 2.5 Column with Internal Rain Water down Pipe

2.3.2 Uniplanar Trusses

Lattice girders are light and economical and they are fairly simple to design. Usually they comprise of an upper and a lower chord and a lattice consisting of bracing members (Figure 2.6). The chords may or may not be parallel.

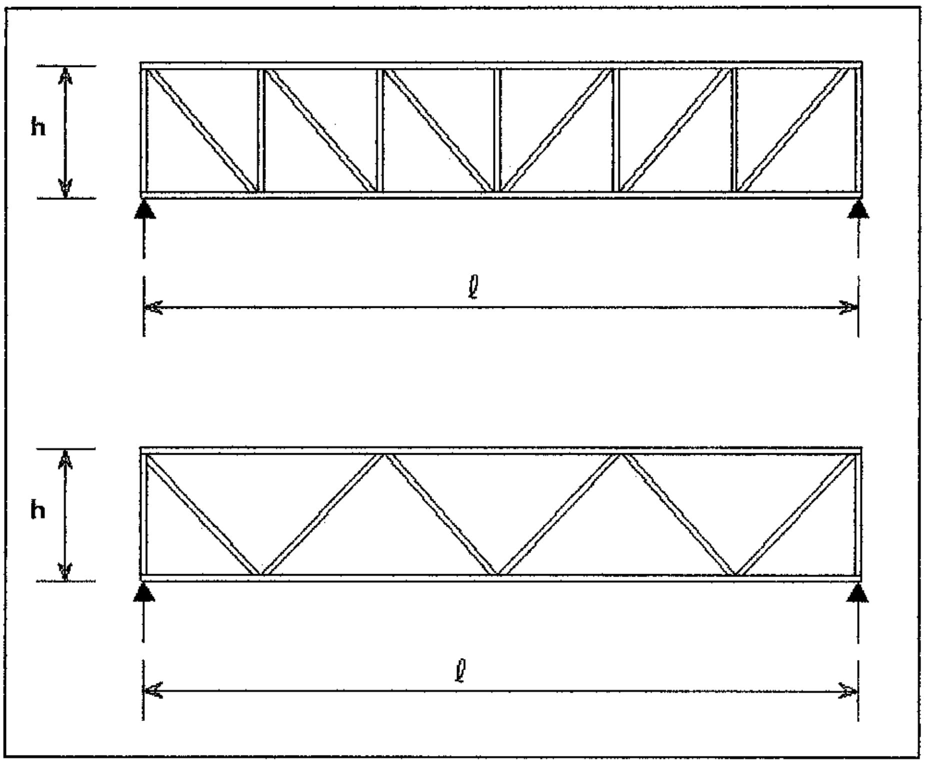


Fig. 2.6 Uniplanar Lattice Girders

Lattice girders are characterised by the span "l", the depth "h", the lattice geometry, and the distance between the joints. The depth "h" is determined in relation to the span, loads, maximum deflection, etc. An increase in "h" reduces the loads in the chords, but increases the effective lengths of the bracings. The value of "h" is usually between l/10 and l/16. The joints are preferentially located at the load application points.

A lattice structure is usually designed so as to transmit the applied loads purely through axial loadings in the members. In hollow section girders, however, the chords are usually continuous and the bracings are welded onto the chords. Secondary bending moments are produced in both the members and the joints. It is nevertheless well accepted that, if the members and the joints are capable of redistributing these secondary moments in a plastic manner, the load analysis can be based on the pinned framework assumption that the framework is pinned.

Bending moments, on the other hand, must be considered when the axes of the members do not converge at a point of junction producing positive or negative eccentricity (see Figure 2.7).

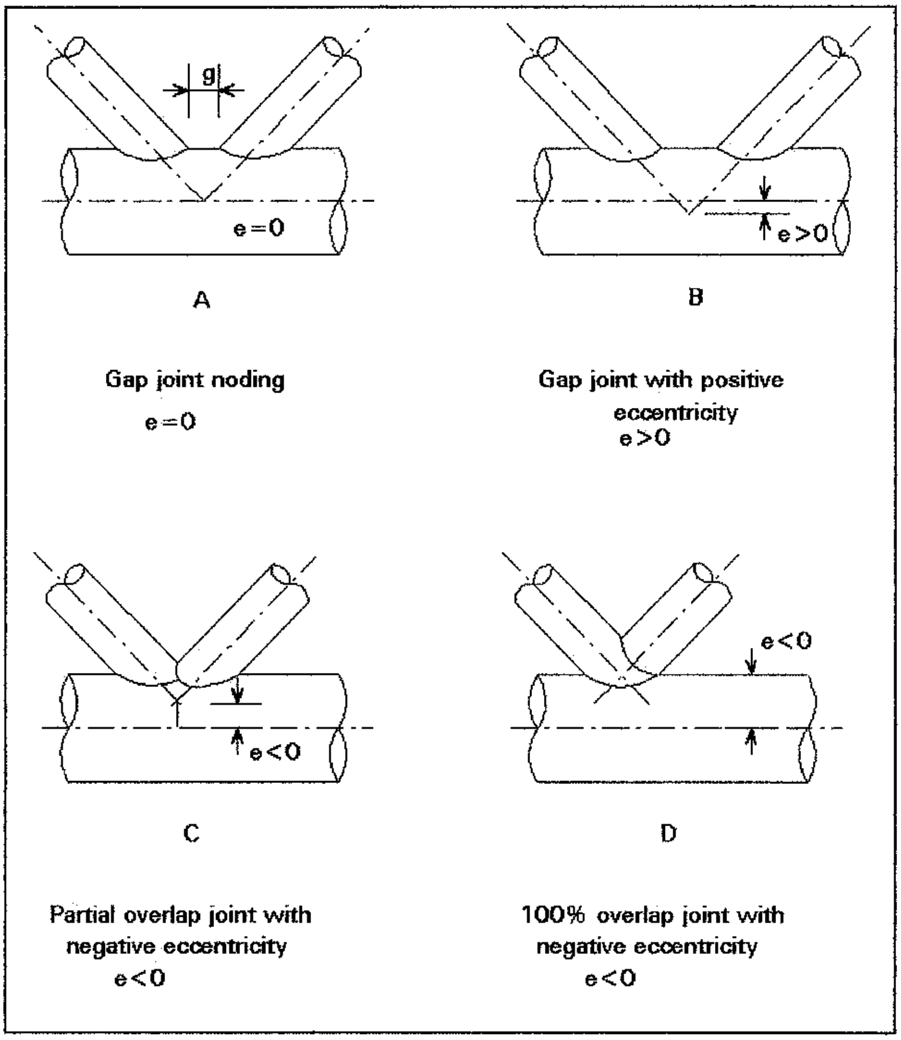


Fig. 2.7 Joint Eccentricities

2.3.3 Multiplanar Trusses

Multiplanar trusses are, in general, represented by triangular and quadrangular girders. They are inherently stable, i.e. they require no external bracing of any kind and constitute autonomous load-bearing elements. These girders offer a spatial type of strength, which means they can withstand loads and bending moments from all directions. The depth of a girder is usually between l/18 and l/15 of the distance l between the supports.

The joint arrangement depends on the nature of the chord (circular, square, rectangular sections), the type of connection (bolted to gusset plates or welded, with or without flattening of bracing ends).

2.3.4 Space Structures

Space structures consist of identical elements, designated as modules, joined together to make a load-bearing framework. The module can be linear, plane or three dimensional (Figure 2.8). The members of a space structure are often in an isotropic state as regards buckling and bearing loads which are either tension or compression. Hollow sections, especially circular ones, are extremely well adapted as space frame members.

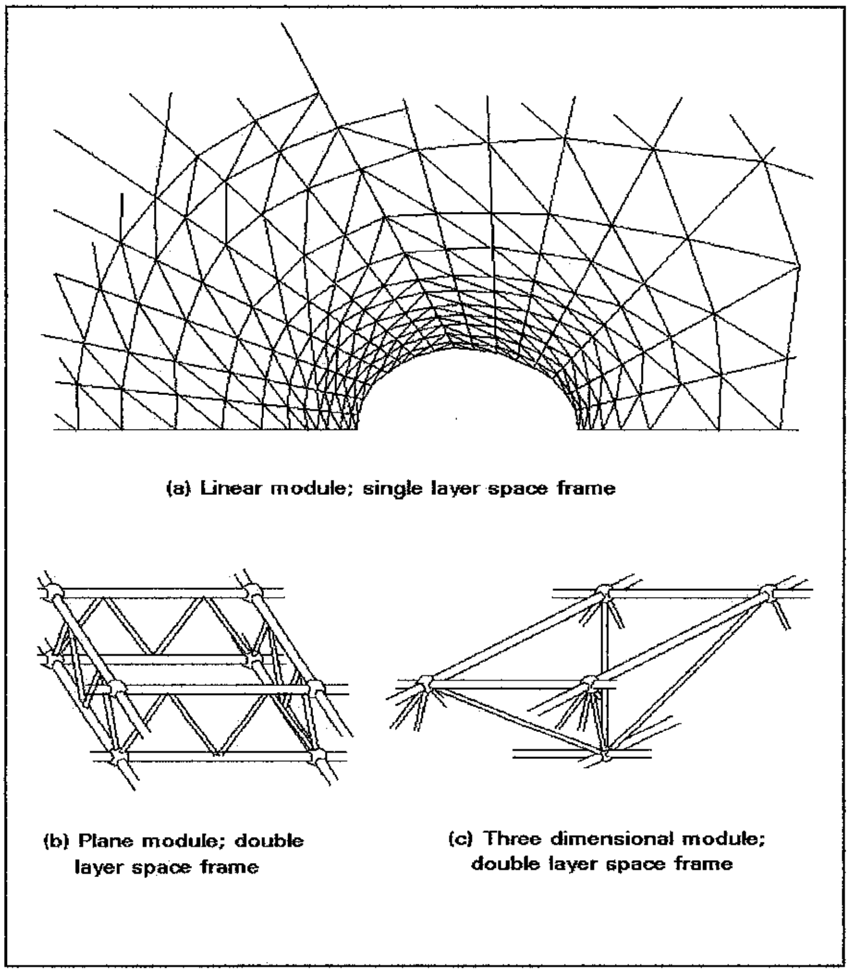


Fig. 2.8 Space Structure Types

Due to the special end shaping needed for the direct connection of the hollow sections, special connectors have been developed. The development of space structures was stimulated by the availability of these prefabricated connectors and later by the development of computers and matrix calculation methods.

Although space frames with connectors are characterized by their economy due to manufacture of the structural parts on a mass production basis and simplified assembly through repeated similar operations, they are still relatively expensive. Therefore, they are applied mostly when an architect prefers them for aesthetic appearance or some special requirements, such as very large span.

CHAPTER 3

METHODOLOGY

3.1 Research

- Cover all the study through reading and understanding from literature review, case study, journal, fiction book, text book, website, articles and other reading materials. It is a self study to detail understand and to get as much as knowledge and information about the project.
- Prepare all the information and materials needed for software analysis.

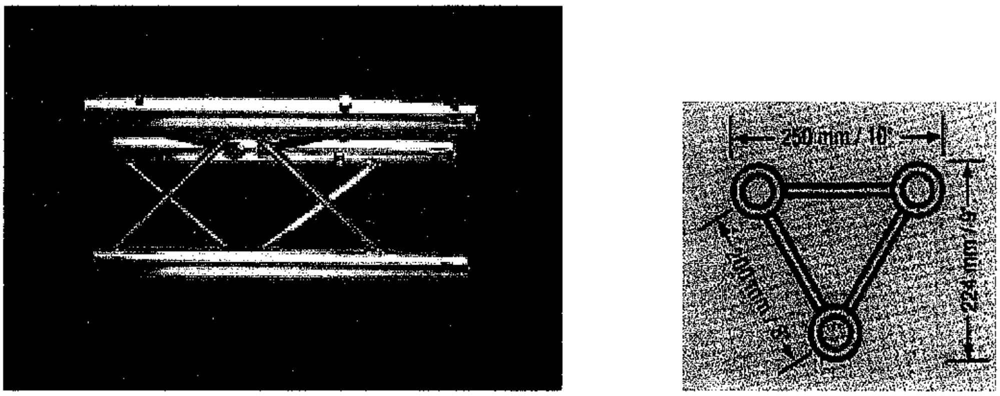
3.2 Modeling

The modeling procedure is an important step in analysis of a structure. Modeling provides a basis for the analysis, in other words, a general idea of what is to be done in the latter stages of the research. Ideas on how the structure is to be put together, the connection details, the loads that will be imposed on the structure and variations of the structure must be put together in the modeling process so that the analysis could be conducted in a smooth and proper manner to ensure reliable results.

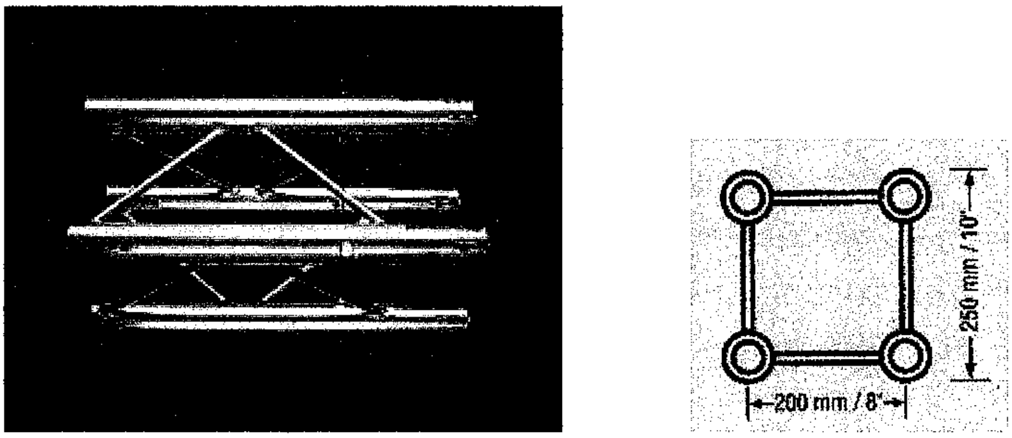
This section will deal with all the elements listed above, and preliminary calculation of members would also be put forth, so appropriate steel member sections can be used in the preliminary design of the trussed column. Several load conditions will be imposed, and certain parameters will be looked into to gather useful results needed for the analysis of the trussed column.

3.2.1 The Trussed Column

Two types of trussed column will be used in the analysis, namely the triangular and rectangular trussed column. Figure 3.1 shows the different types of trussed column used in the analysis. It comprises 6 bays; each measuring 3m, 4.5m and 6m height, the two trussed columns shown in this figure comprises the main chords and the bracing members.



3 Chord Trussed Column (triangular)



4 Chord Trussed Column (rectangular)

Figure 3.1 Types of Trussed Column

The main chords are the vertical members spanning the length of the column. As shown in Figure, a trussed column can consist of 2, 3 or 4 main chords, each having its own advantages and disadvantages. The secondary members are the horizontal ones, transferring the loads from the top main chord to the bottom main chords. The secondary members can be stiffeners, or even column for the top main chords, just like mini frame systems, with beam and columns joining together acting as a frame. Bracing members are diagonal members located at the bays of the trussed column, stiffening the trussed column. Figure 3.2 shows the different groupings of the trussed column members.

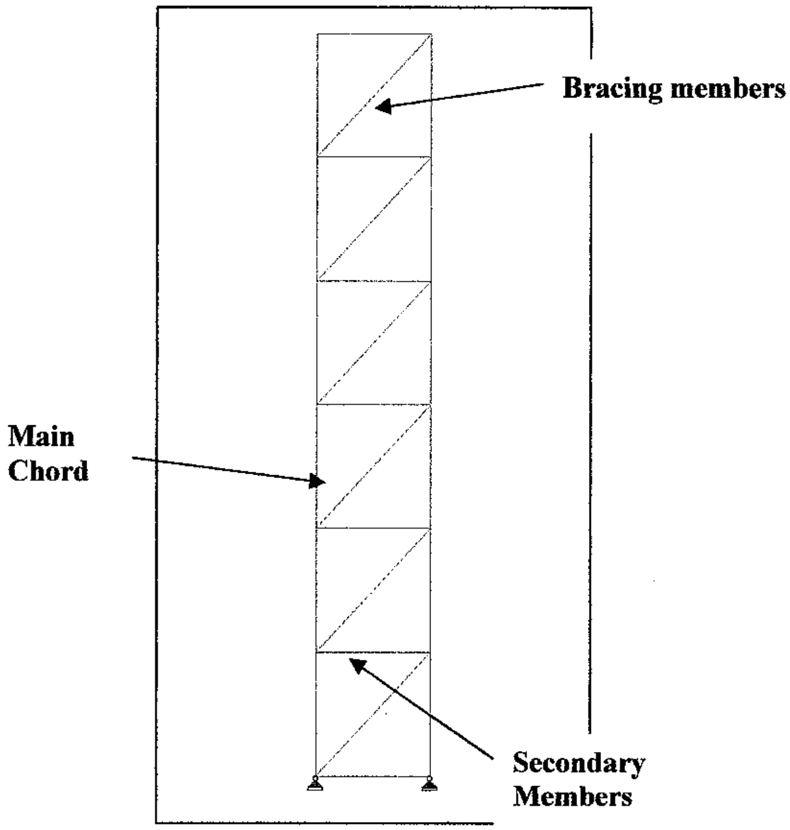


Figure 3.2 Grouping of Trussed Column

3.2.2 Connection of members

In trussed column, the individual members are welded together to keep them in place. For example, the secondary members are welded to the main chords to form the trussed column. Proper connections are essential to achieve a satisfactory load transfer system without the risk of structural failure. The connections are deemed to be rigid connections, as welding provides restraint towards moments.

3.2.3 Support

In a framed structure using trussed column, the ends of the top and bottom chord of the trussed column are anchored to the beam and the floor. Thus, in the analysis, the ends of the top and bottom chords of the trussed columns are supported by pinned connections.

3.2.4 Loading

The trussed column is subjected to different loads to see their behavior due to certain loads. In the analysis, the two trussed columns were subjected to axial loads only.

3.2.5 Point Loads

There is 5 point loads, consisting of 100KN, 200KN, 300KN, 400KN and 500KN were imposed on both trussed column. The diagram is shown in the Figure 3.3(a) and 3.3(b).

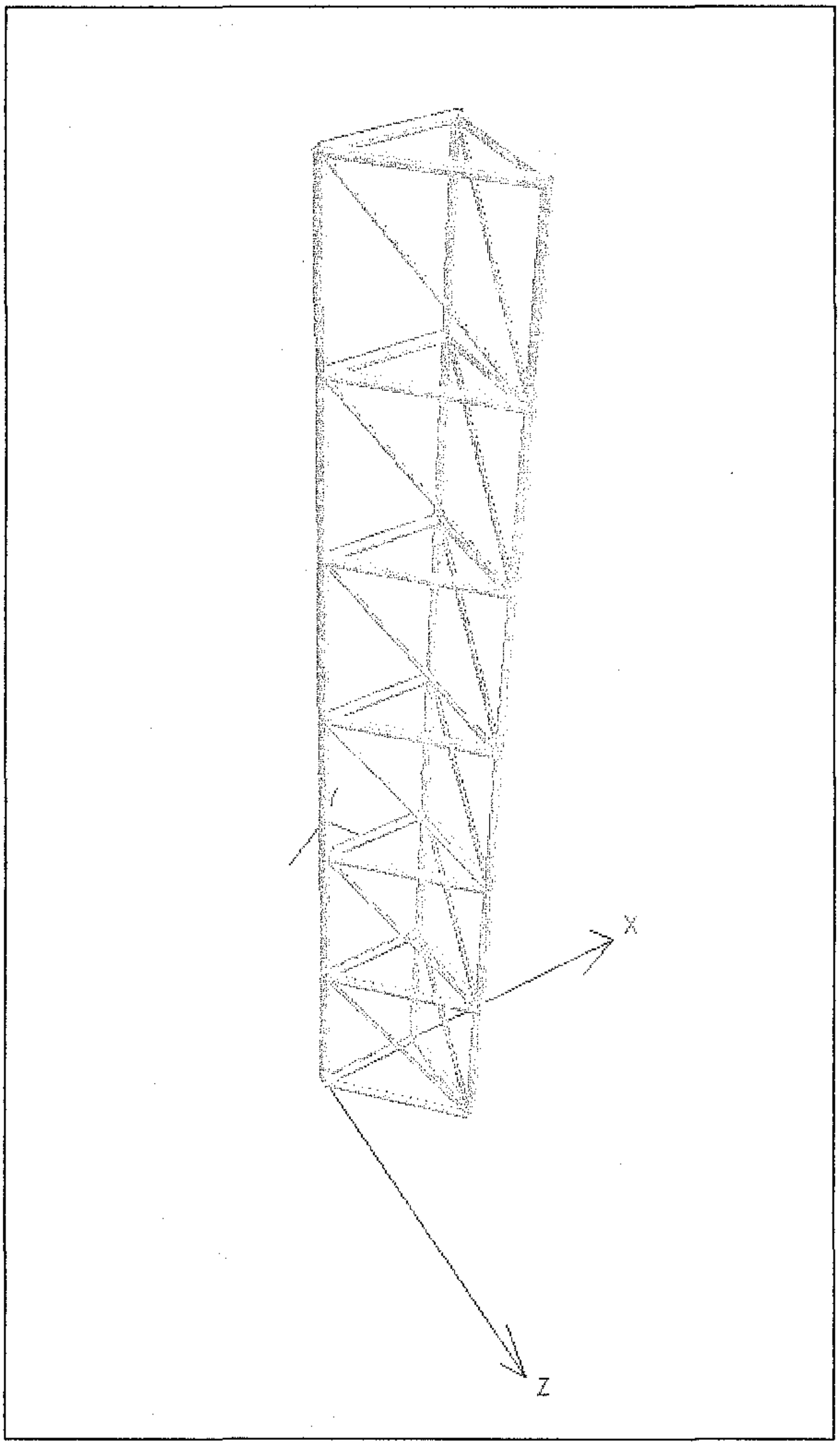


Figure 3.3 Triangular truss column

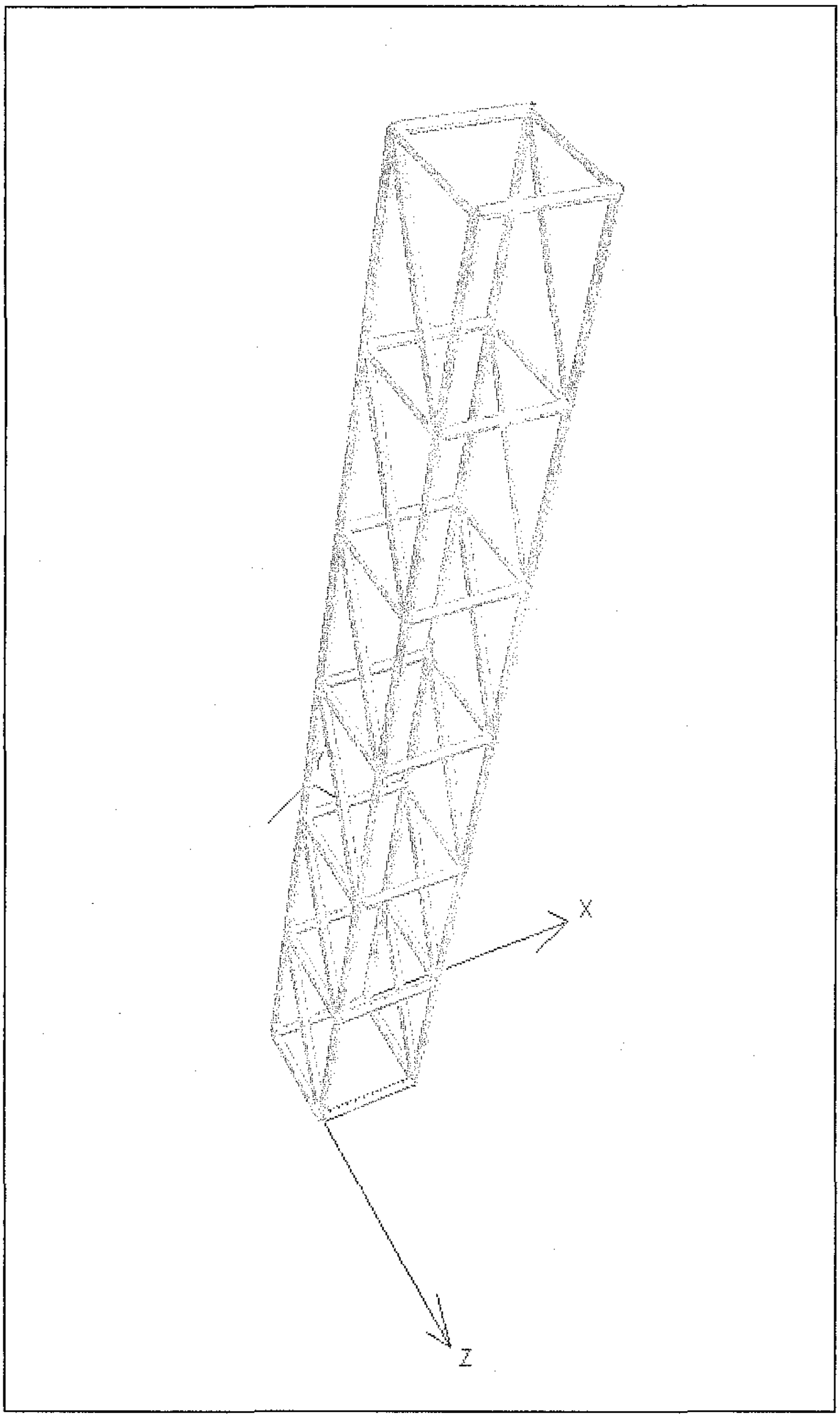


Figure 3.4 Rectangular Truss Column

3.3 Software Analysis

Using STAAD PRO 2004 to do the analysis on the truss-column

- Several model was constructed using the software
- Using different design loads, the result was compared according to;
  - Load distribution
  - Shear forces
  - Tensile strength
  - Torsion
  - Compression and tension in truss members
  - Etc.

3.3.1 Computer Application Used for Analysis

The Analysis will be done using a computer application due to the difficulties involved if hand calculation was to be done. The STAAD Pro software fulfils this requirement, and design were based on the code for steel design, BS5950 : Part 1 : 1990.

All three member group (main chord, secondary members, bracing members) are modeled using column elements. STAAD Pro is capable of simulating the connection, supports and applied loads without any difficulties. The input mechanism can be done by the Graphic User Interface (GUI) method or the text based method. The GUI involves the user to just click on the commands listed in the toolbar and apply them to the model onscreen, while the text based method is more suitable for advance users, where the command names and short forms are understood by the user. In the text-based method, user will have to type in all the commands in a text-based program, such as WordPad or Notepad. Then, the software will generate the model based on what is on the text file.

The output from the STAAD Pro software is extensive, so certain parameters are selected to limit the output to certain useful ones for comparison. The parameters selected are weight, compression or tension forces in the members, graphical output to see what the distribution of forces is like, and the suitability of members selected with respect to conforming to the code checking procedure.

3.4 Analysis and Interpretation of Result

The model described in section 3.2 has been analyzed along with appropriate loading using the STAAD Pro structural analysis software. Result of the analysis is presented in the form of tables and graphs, depending on the suitability for interpretation purposes. The result obtained is divided into several categories:

i. Trussed column subjected to point loads
ii. Deflection of the trussed column
iii. Others structural analysis behavior of the trussed column

The 3 cases listed above are analyzed using different criteria, namely:

i. Type of support of the trussed column
ii. Compression/tension ratio of the trussed column
iii. Maximum compression and tension forces of the elements of the trussed beam and their distribution
iv. Deflection check

3.5 Result

3.5.1 Maximum compression stress

Graphs show the maximum value of compression stress from all section of the rectangular and triangular truss column.

Graph maximum compression stress against loading for 0.5m bay

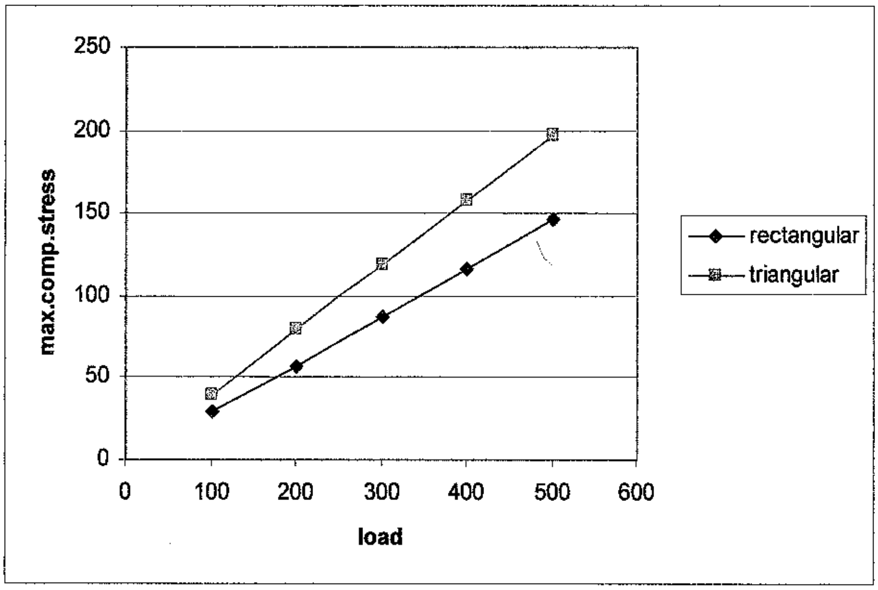


Figure 3.5 Maximum compression stresses for 3m column

Graph maximum compression stress against loading for 0.75m bay

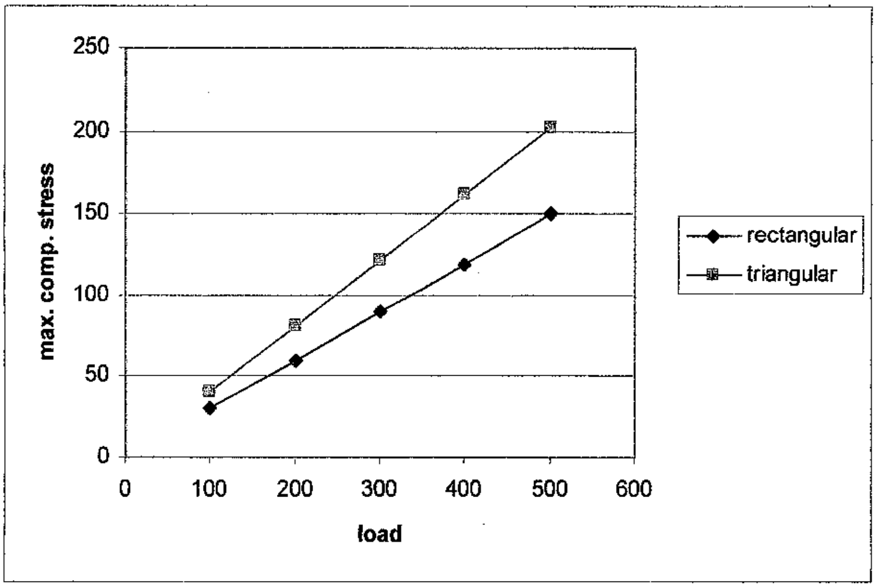


Figure 3.6 Maximum compression stresses for 4.5m column

Graph maximum compression stress against loading for 1.0m bay

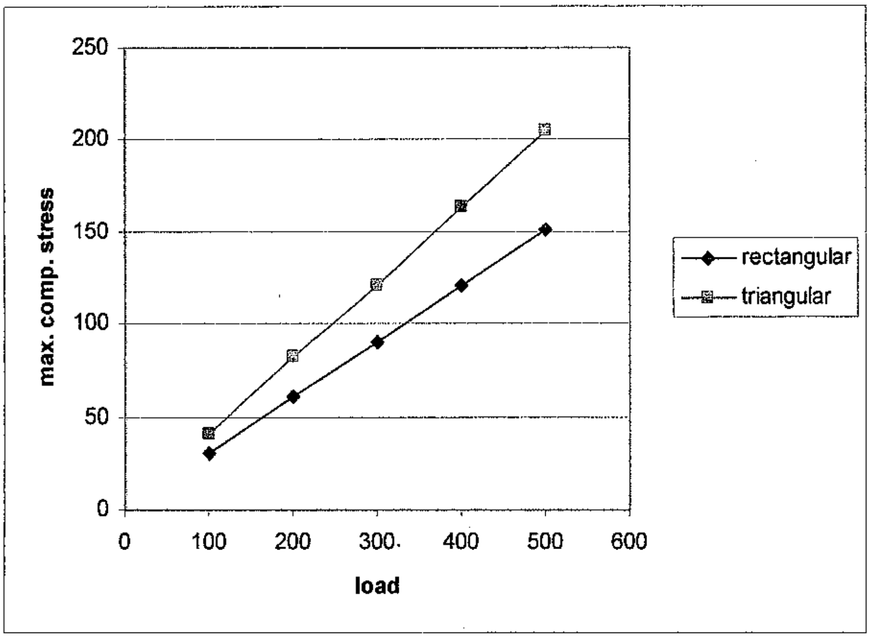


Figure 3.7 Maximum compression stresses for 6m column

From the graph, it shows that maximum compression stress in column member increase as the height of the column increase. The entire graph shows similar pattern and nothing much different between the value for compression stress for rectangular and triangular truss column. Even though the different in value is slightly different, it is clearly shows that compression in triangular column is greater than the compression in rectangular column.

3.5.2 Maximum displacement

Graphs show the maximum value of displacement from all section of the rectangular and triangular truss column.

Graph maximum displacement against loading for 0.5m bay

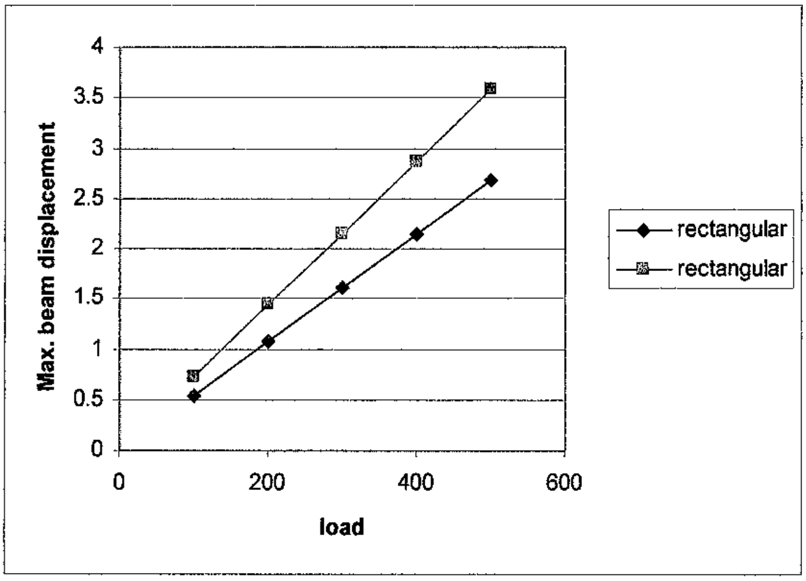


Figure 3.8 Maximum displacements for 3m column

Graph maximum displacement against loading for 0.75m bay

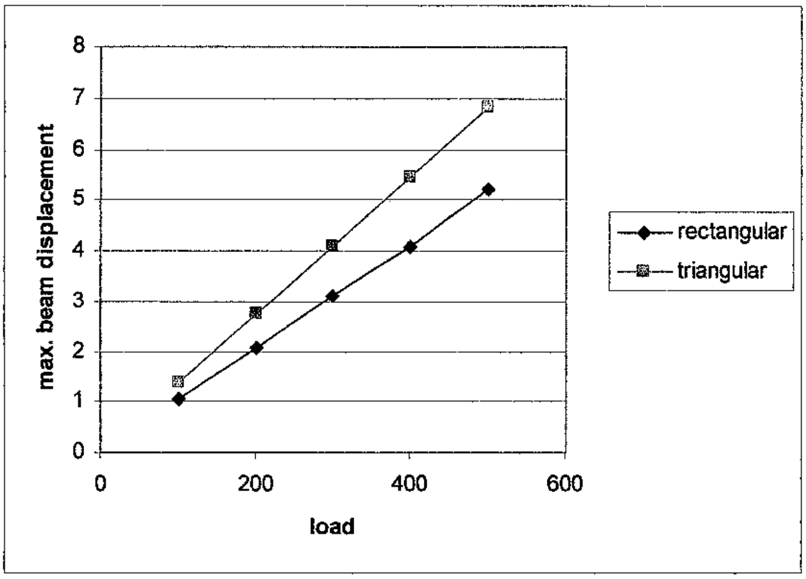


Figure 3.9 Maximum displacements for 4.5m column

Graph maximum displacement against loading for 1.0m bay

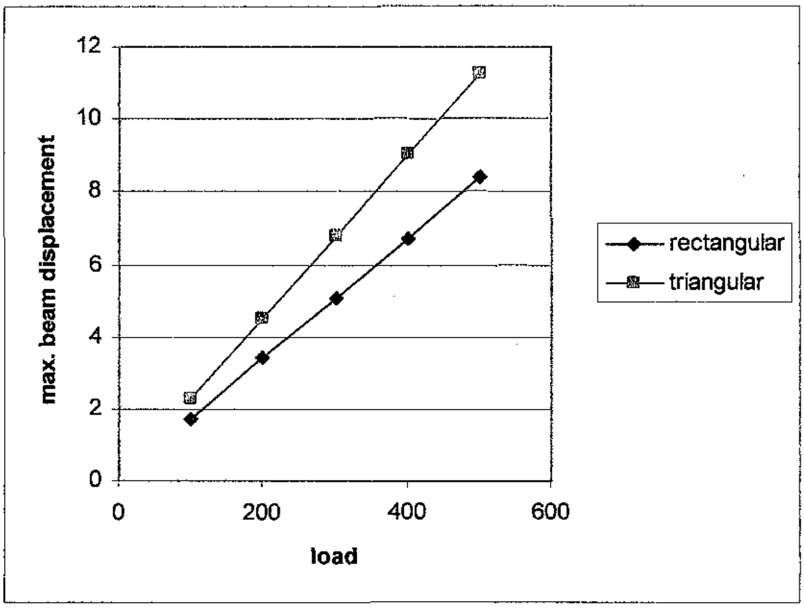


Figure 3.10 Maximum displacements for 6m column

The pattern of the graph indicate that the displacement in column member gradually increase when the length of each bays is increase. The value for 1m bays column is slightly bigger that the value form 0.5m and 0.75m bays column. For the type of the column, once again the triangular truss column experience larger displacement than the rectangular column.

3.5.3 Maximum Shear Forces

Graphs show the maximum value of shear forces from all section of the rectangular and triangular truss column.

Graph maximum shear forces against loading for 0.5m bay

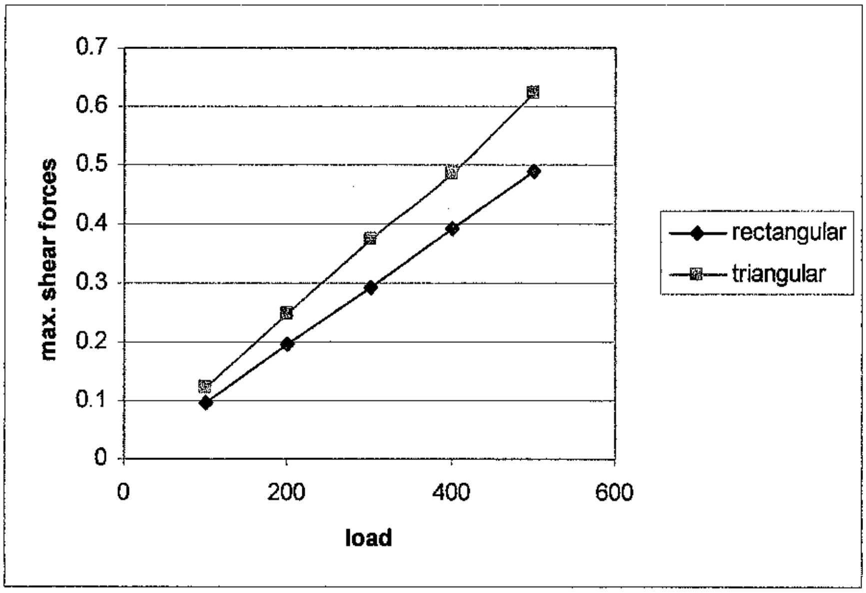


Figure 3.11 Maximum shear forces for 3m column

Graph maximum shear forces against loading for 0.75m bay

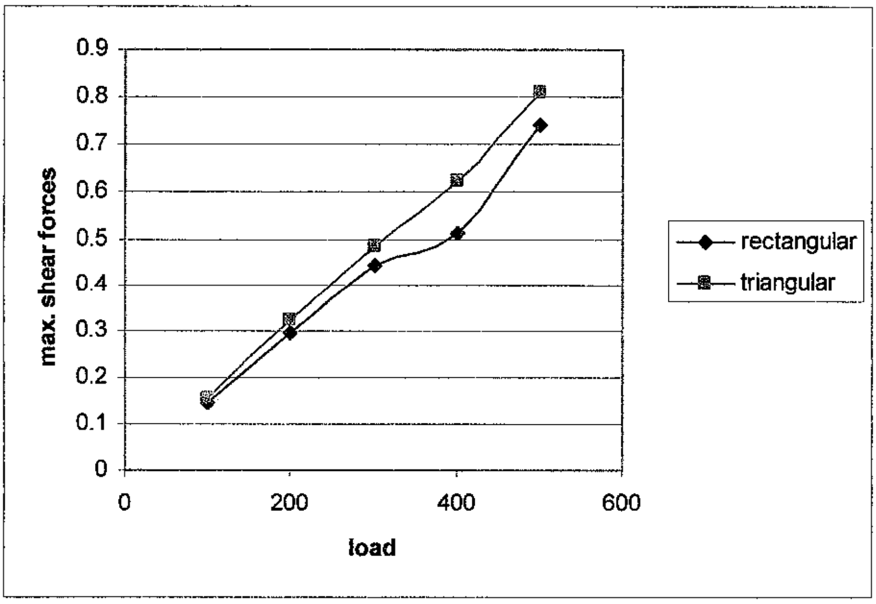


Figure 3.12 Maximum shear forces for 4.5m column

Graph maximum shear forces against loading for 1.0m bay

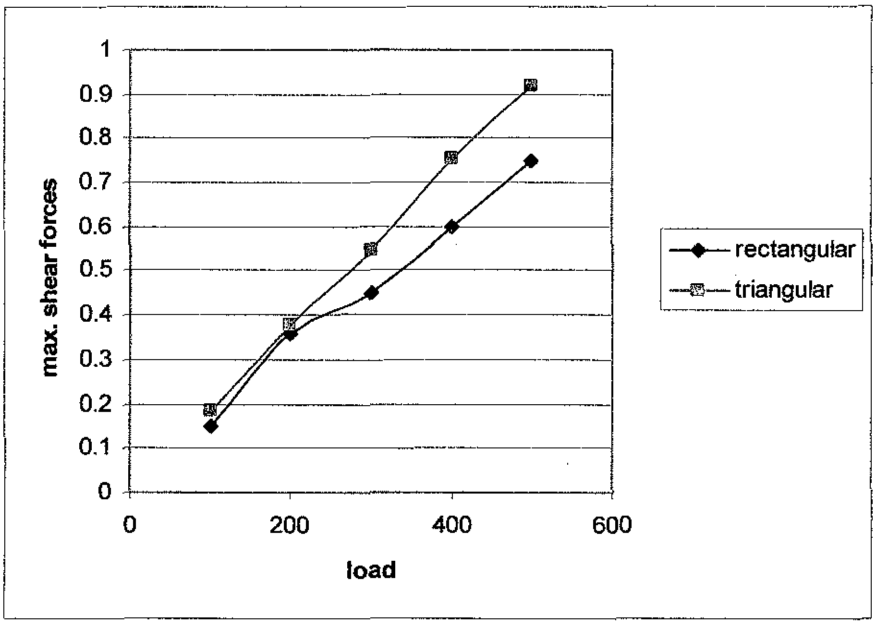


Figure 3.13 Maximum shear forces for 6m column

Graph obviously indicates that maximum shear forces within triangular truss column are higher than shear forces in rectangular truss column. All graphs again showing similar pattern and the value are not much different between each types of column.

3.5.4 Maximum displacement for fixed and pinned support

In this case, the load 300KN was taken as the reference load to study the maximum displacement in the truss member for both types of column. The Graphs show the maximum displacement of the column section for both fixed and pinned support.

Graph maximum displacement against length for rectangular column

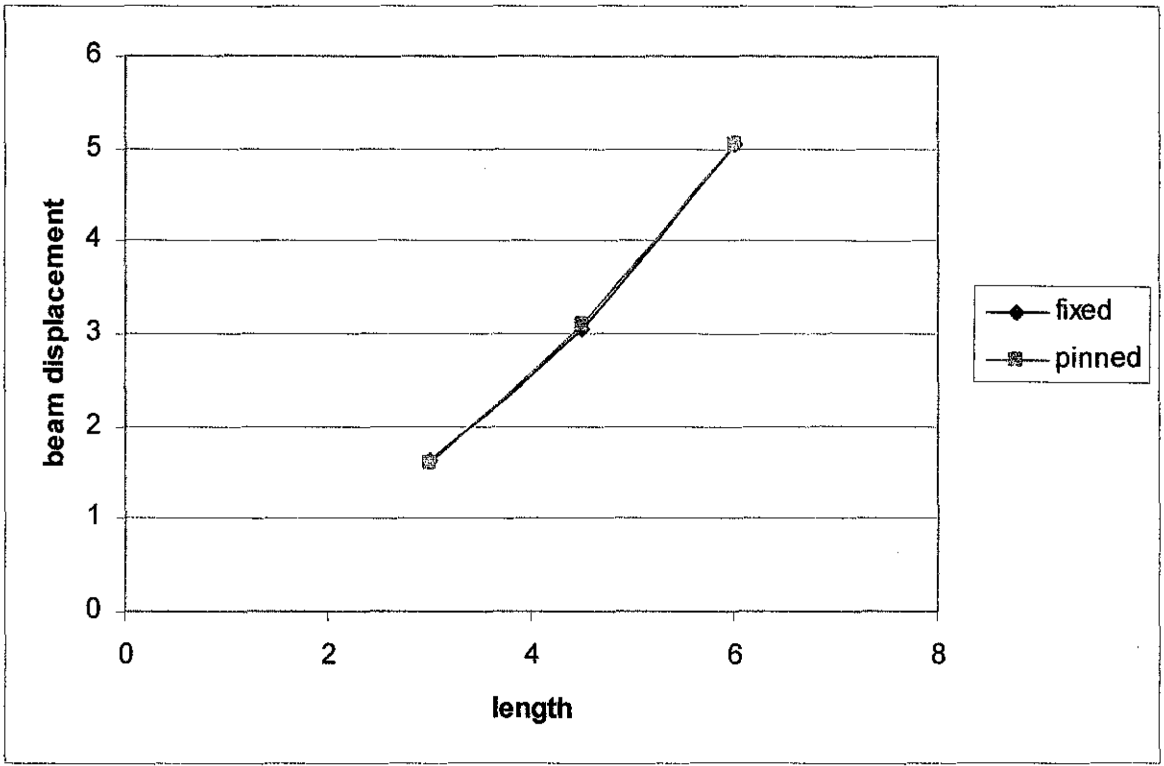


Figure 3.14 Maximum displacements for rectangular column

Graph maximum displacement against length for triangular column

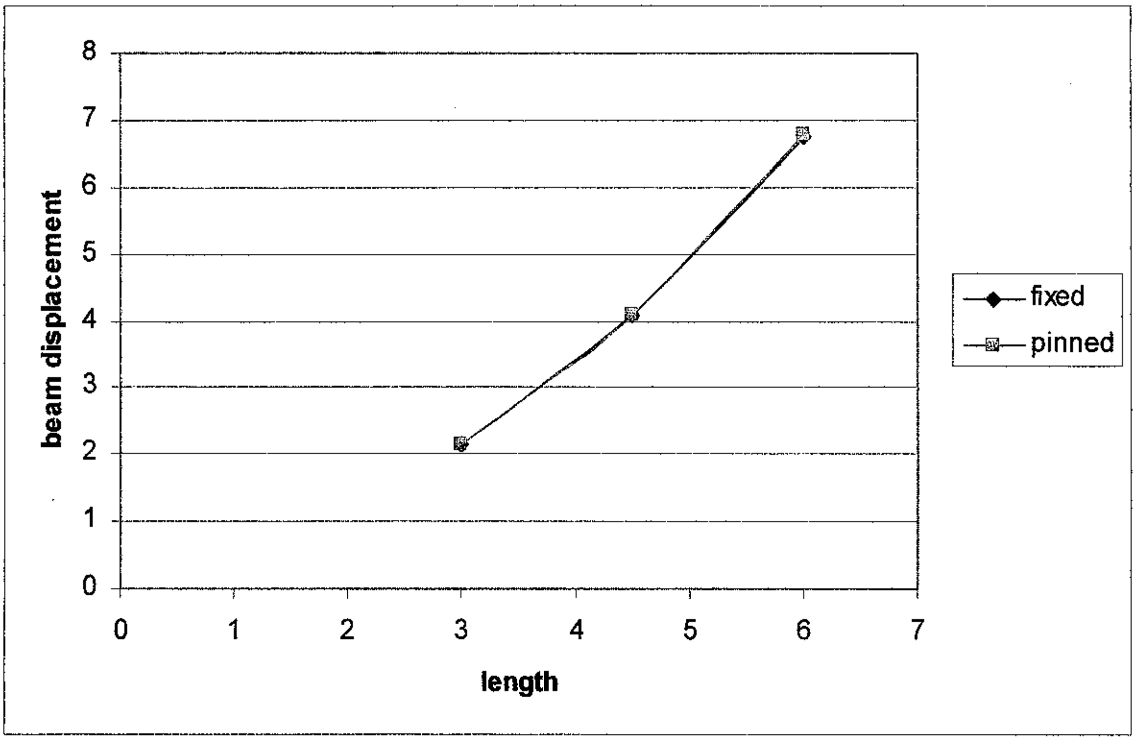


Figure 3.15 Maximum displacements for triangular column

Graphs shows that the maximum displacement in truss member for the certain length of the column. The analysis is made for the two types of support that is pinned and fixed. As shown on the graph, there is not much different in displacement of truss member for the fixed and pinned support. But as for the value, pinned support contributes to the higher displacement than the fixed support with the applied load. For both types of support, triangular column experience higher displacement that the rectangular column.

3.6 Discussion

According to the graphs, a few discussions can be concluding such as:

- Triangular truss column experienced higher compressive stress, tensile stress, and displacement and shear forces than the rectangular truss column.
- Pinned support give higher stress to the column member than the fixed support
- The different between both types of column is not significantly different, so both columns are reliable and preferable.
- This analysis is conducted using Staad Pro software only. Different result may occur if the real trussed column is constructed and test using real equipment.

CHAPTER 4

4.1 CONCLUSION

Trussed column have been widely used in the world of structural engineering, its usage ranging from simple 2-chord trussed column for simple structure purposes, to huge rectangular trussed column for crane gantry girders. Trussed column are more widely used today compare to 50 years ago, because the analysis of the trusses are getting less complicated with structural design software widely available in the market. The ease of analysis, coupled with expertise in construction gained by experience of fabrication makes it even more desirable in the world construction.

The availability of high strength steels leads to the use of thin walled steel pipes in built-up truss-column. However, the use of thin walled steel pipes in truss-column gives a rise to local buckling and other structural failure that would appreciably reduce the strength and ductility performance of the members. This paper studies the structural behavior of steel pipes in truss-column by applied the compression forces at the end of the column. Geometric and material nonlinear analyses are performed to investigate the critical local and post-local buckling strengths of steel pipes under compression and in-plane bending. Initial geometric imperfections and residual stresses presented in steel pipes, material yielding and strain hardening are taken into account in the nonlinear analysis. Based on the results obtained from test, a set of design formulas are proposed for determining the critical local buckling and ultimate strengths of steel pipes in truss-column.

4.2 RECOMMENDATION

Since the use of trussed column has gained momentum in the past few decades, more studies should be done to better understand its behavior under conditions which are not analyzed in this thesis. One of the most obvious would be the arched trussed column. More and more trussed column are constructed for its aesthetic purposes, and the arched trussed column exudes a sense of class and excellent to the structure. Hence, the behavior of the arched trussed column should be studied in further researches.

This research deals with the theoretical aspect of the analysis, meaning all simulations are done by computer software. Further studies could include a real-life model, and tested according to the parameters analyzed in this thesis. The difference between computer simulation and real-life can be compared and analyzed. This could provide a basis for prediction of real-life trussed beam in action after the design is done by the engineer using a computer application.

Besides the shape of the trussed column and difference between simulation and real-life models, the trussed column can also be tested under several environmental aspects. The effects of temperature, exposure to the surface and moisture could also be studied for more precise results.

The connection details can also be studied for the exact transfer of bending moments in the trussed column elements. The behavior of pinned and hinged connection was studied in this thesis, but on a limited scale. More elaborate approach could be adopted, so that different parameters regarding to the methods of connection could be studied.

REFERENCES

1. www.sciencedirect.com

2. http://www.efunda.com/formulae/solid_mechanics/columns/intro.cfm
3. **Bradford M.A., 1998**, "*Behavior and Design of Steel Structures to AS4100*", Taylor and Francis.

4. www.ce.ufl.edu

5. Manual of Water Supply Practices – M11, Fourth Edition, 2004, "Steel Pipe-A Guide for Design and Installation

6. **Hassinen P. (1999)**. "*Light-Weight Steel Aluminium Structures,*" ICSAS '99, Elsevier.

7. www.ntrs.nasa.gov/archive/nasa/casi.ntrs.nasa.gov/19800014247

8. Hibbeler R. C. , 2002, "Structural Analysis", Prentice-Hall.

9. **Babich B.** and **Allen D**, Truss Bracing – Design Guidelines for Bracing of Cold-Formed Steel Trusses, 2004.

10. http://www.kuleuven.ac.be/bwk/materials/Teaching/master.htm

11. http://www.struturemag.org

12. **Knowles, P. (1977),** "*Design of Structural Steelwork.*" London: Surrey University Press.

13. **Morris, L.J. and Plum, D.R. (1996).** "*Structural Steelwork design.*" Essex: Longman.

14. **Salmon, C.G. and Johnson, J.E. (1996)**. "*Steel Structures, Design and Behavior, 4th Edition.*" New York: HarperCollins College Publishers.

15. **BS5950 : Part 1 : 1990**. British Standard Institution

APPENDIX 1

Input code for rectangular truss column

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 09-May-07
END JOB INFORMATION
INPUT WIDTH 79
*******************************************
*       STAAD.Pro Generated Comment       *
*******************************************
*******************************************
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0.5 0 0; 3 0.5 0 0.5; 4 0 0 0.5; 5 0 0.5 0; 6 0.5 0.5 0;
7 0.5 0.5 0.5; 8 0 0.5 0.5; 9 0 1 0; 10 0.5 1 0; 11 0.5 1 0.5; 12 0 1 0.5;
13 0 1.5 0; 14 0.5 1.5 0; 15 0.5 1.5 0.5; 16 0 1.5 0.5; 17 0 2 0; 18 0.5 2 0;
19 0.5 2 0.5; 20 0 2 0.5; 21 0 2.5 0; 22 0.5 2.5 0; 23 0.5 2.5 0.5;
24 0 2.5 0.5; 25 0 3 0; 26 0.5 3 0; 27 0.5 3 0.5; 28 0 3 0.5;
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 1; 5 5 6; 6 6 7; 7 7 8; 8 8 5; 9 9 10; 10 10 11;
11 11 12; 12 12 9; 13 13 14; 14 14 15; 15 15 16; 16 16 13; 17 17 18; 18 18 19;
19 19 20; 20 20 17; 21 21 22; 22 22 23; 23 23 24; 24 24 21; 25 25 26; 26 26 27;
27 27 28; 28 28 25; 101 1 5; 102 2 6; 103 3 7; 104 4 8; 201 5 9; 202 6 10;
203 7 11; 204 8 12; 301 9 13; 302 10 14; 303 11 15; 304 12 16; 401 13 17;
402 14 18; 403 15 19; 404 16 20; 501 17 21; 502 18 22; 503 19 23; 504 24 20;
601 21 25; 602 22 26; 603 23 27; 604 24 28; 1001 4 5; 1002 3 6; 2001 8 9;
2002 7 10; 3001 12 13; 3002 11 14; 4001 16 17; 4002 15 18; 5001 20 21;
5002 19 22; 6001 24 25; 6002 23 26;
*Member from node to node
MEMBER PROPERTY BRITISH
1 TO 28 TABLE ST PIPE OD 0.04 ID 0.02
101 TO 104 TABLE ST PIPE OD 0.04 ID 0.02
201 TO 204 TABLE ST PIPE OD 0.04 ID 0.02
301 TO 304 TABLE ST PIPE OD 0.04 ID 0.02
401 TO 404 TABLE ST PIPE OD 0.04 ID 0.02
501 TO 504 TABLE ST PIPE OD 0.04 ID 0.02
601 TO 604 TABLE ST PIPE OD 0.04 ID 0.02
1002 TABLE ST PIPE OD 0.02 ID 0.01
1001 TABLE ST PIPE OD 0.02 ID 0.01
2002 TABLE ST PIPE OD 0.02 ID 0.01
2001 TABLE ST PIPE OD 0.02 ID 0.01
3002 TABLE ST PIPE OD 0.02 ID 0.01
3001 TABLE ST PIPE OD 0.02 ID 0.01
```

```
4002 TABLE ST PIPE OD 0.02 ID 0.01
4001 TABLE ST PIPE OD 0.02 ID 0.01
5002 TABLE ST PIPE OD 0.02 ID 0.01
5001 TABLE ST PIPE OD 0.02 ID 0.01
6002 TABLE ST PIPE OD 0.02 ID 0.01
6001 TABLE ST PIPE OD 0.02 ID 0.01
*********************************
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
END DEFINE MATERIAL
CONSTANTS
MATERIAL STEEL MEMB 1 TO 28
MATERIAL STEEL MEMB 101 TO 104
MATERIAL STEEL MEMB 201 TO 204
MATERIAL STEEL MEMB 301 TO 304
MATERIAL STEEL MEMB 401 TO 404
MATERIAL STEEL MEMB 501 TO 504
MATERIAL STEEL MEMB 601 TO 604
MATERIAL STEEL MEMB 1001 1002
MATERIAL STEEL MEMB 2001 2002
MATERIAL STEEL MEMB 3001 3002
MATERIAL STEEL MEMB 4001 4002
MATERIAL STEEL MEMB 5001 5002
MATERIAL STEEL MEMB 6001 6002
SUPPORTS
1 PINNED
2 PINNED
3 PINNED
4 PINNED
*LOAD
LOAD 1 (AXIAL LOAD)
JOINT LOAD
25 FY -50
26 FY -50
27 FY -50
28 FY -50
PERFORM ANALYSIS
PRINT ANALYSIS RESULTS
FINISH
```

APPENDIX 2

Input code for triangular truss column

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 09-May-07
END JOB INFORMATION
INPUT WIDTH 79
*********************************************
*        STAAD.Pro Generated Comment        *
*********************************************
*********************************************
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0.5 0 0; 3 0.5 0 0.5; 4 0 0.75 0; 5 0.5 0.75 0; 6 0.5 0.75 0.5;
7 0 1.5 0; 8 0.5 1.5 0; 9 0.5 1.5 0.5; 10 0 2.25 0; 11 0.5 2.25 0;
12 0.5 2.25 0.5; 13 0 3 0; 14 0.5 3 0; 15 0.5 3 0.5; 16 0 3.75 0;
17 0.5 3.75 0; 18 0.5 3.75 0.5; 19 0 4.5 0; 20 0.5 4.5 0; 21 0.5 4.5 0.5;
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 1; 4 4 5; 5 5 6; 6 6 4; 7 7 8; 8 8 9; 9 9 7; 10 10 11;
11 11 12; 12 12 10; 13 13 14; 14 14 15; 15 15 13; 16 16 17; 17 17 18; 18 18 16;
19 19 20; 20 20 21; 21 19 21; 101 1 4; 102 2 5; 103 3 6; 201 4 7; 202 5 8;
203 6 9; 301 7 10; 302 8 11; 303 9 12; 401 10 13; 402 11 14; 403 12 15;
501 13 16; 502 14 17; 503 15 18; 504 16 19; 505 17 20; 506 18 21; 1001 5 3;
1002 4 3; 2001 7 6; 2002 8 6; 3001 10 9; 3002 11 9; 4001 13 12; 4002 14 12;
5001 16 15; 5002 17 15; 5003 19 18; 5004 20 18;
*Member from node to node
MEMBER PROPERTY BRITISH
1 TO 18 TABLE ST PIPE OD 0.04 ID 0.02
101 TO 103 TABLE ST PIPE OD 0.04 ID 0.02
201 TO 203 TABLE ST PIPE OD 0.04 ID 0.02
301 TO 303 TABLE ST PIPE OD 0.04 ID 0.02
401 TO 403 TABLE ST PIPE OD 0.04 ID 0.02
501 TO 503 TABLE ST PIPE OD 0.04 ID 0.02
1001 1002 TABLE ST PIPE OD 0.02 ID 0.01
2001 2002 TABLE ST PIPE OD 0.02 ID 0.01
3001 3002 TABLE ST PIPE OD 0.02 ID 0.01
4001 4002 TABLE ST PIPE OD 0.02 ID 0.01
5001 5002 TABLE ST PIPE OD 0.02 ID 0.01
506 TABLE ST PIPE OD 0.04 ID 0.02
504 TABLE ST PIPE OD 0.04 ID 0.02
505 TABLE ST PIPE OD 0.04 ID 0.02
21 TABLE ST PIPE OD 0.04 ID 0.02
19 TABLE ST PIPE OD 0.04 ID 0.02
```

```
20 TABLE ST PIPE OD 0.04 ID 0.02
5004 TABLE ST PIPE OD 0.02 ID 0.01
5003 TABLE ST PIPE OD 0.02 ID 0.01
**********************************
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
END DEFINE MATERIAL
CONSTANTS
MATERIAL STEEL MEMB 1 TO 18
MATERIAL STEEL MEMB 101 TO 103
MATERIAL STEEL MEMB 201 TO 203
MATERIAL STEEL MEMB 301 TO 303
MATERIAL STEEL MEMB 401 TO 403
MATERIAL STEEL MEMB 501 TO 503
MATERIAL STEEL MEMB 1001 1002
MATERIAL STEEL MEMB 2001 2002
MATERIAL STEEL MEMB 3001 3002
MATERIAL STEEL MEMB 4001 4002
MATERIAL STEEL MEMB 19 TO 21 504 TO 506 5001 TO 5004
SUPPORTS
1 PINNED
2 PINNED
3 PINNED
*LOAD
LOAD 1 (AXIAL LOAD)
JOINT LOAD
19 FY -66.66
20 FY -66.66
21 FY -66.66
PERFORM ANALYSIS
PRINT ANALYSIS RESULTS
FINISH
```