FEASIBILITY OF USING COLD FORMED STEEL

FOR

MEDIUM SPAN ROOF STRUCTURES

IN SRI LANKA

B. N. C. FERNANDO

Degree of Master of Engineering in Structural Engineering Design

Department of Civil Engineering

University of Moratuwa
Sri Lanka
February 2014

Declaration

"I declare that the work contained in this thesis has not been previously submitted for a degree or diploma at any other higher education institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made.

Also, I hereby grant to university of Moratuwa the non exclusive right to reproduce and distribute my thesis, in whole or in part in print, electronic or other medium. I retain the right to use this content in whole or part in future works (such as articles or books)."

Signature Date

The above candidate has carried our research for the master under my supervision.

Signature Date

Abstract

Cold formed steel members have been widely used in industrial and commercial buildings over the world with increasing interest and even in the residential development. In the past it was used mostly in non-load bearing structural systems that is partition and architectural feature elements, but it is now used even in the structural systems, and are effective in reducing the self-weight of structure.

Cost of Construction around various parts of the world depends on various factors based on the structural category, availability of material, labour cost, material cost, technology available and use, serviceability limit requirements and standard structural design requirements, so on. Therefore light weight structure itself would not be an effective solution for every construction and structural system. Steel construction industry in Sri Lanka; mostly depends on hot rolled steel member / section for their structural solution. The other type of steel, that is cold formed sections / members available for construction, is very uncommon as a structural element, but it is still using as purlin, for steel roof structures. Feasibility of using cold formed steel in structural roof system has not been specifically studied yet, and construction industry is still waiting for such a detail study to overcome the excessive cost of steel construction in Sri Lanka.

This research is based on the 4-case studies, that were already completed using hot rolled steel members for its structural roof system, contain 4.0m, 8.0m, 10.0m, & 12.0m span parallel girder trusses and pitched trusses. Bay spacing for selected cases were pre-defined according to the column grid of the particular building and was 3.0m, 4.0m, 3.2m, and 6.0m respectively. Under this study, aforesaid roof structures were totally replaced by cold formed steel system (lipped channel sections), and checked the structural ability to reach the design requirements followed by ultimate limit state and serviceability limit state, under feasible limit of cost.

Detail comparison for roof structures were carried out and feasibility of using cold formed steel was studied. It was shown that, for medium span roof structures between ranges of 8.0m to 10.0m could gain a saving of 23% ~ 25% of total cost of roof construction cost. Therefore, uses of cold formed steel (CFS), for structural roof systems under medium scale construction is recommended with minimum saving of 20% of construction cost.

ACKNOWLEDGMENT

First and foremost I offer my sincerest gratitude to my supervisor Dr. Mrs. M T P Hettiarachchi, who has supported me throughout this research with her patience and knowledge whilst providing constructive suggestions encouragement and the valuable assistance in many ways over the past few years. This study would not have been a success to this level without such assistance.

I would like to thank all the staff members of Building and Structural Division of University of Moratuwa and special thank to the Post graduate coordinator, Dr. Mrs D Nanayakkara, and Post Graduate research Coordinator, Dr K Baskaran, for their time / guidance and evaluation of success of this effort.

Special thanks are due to my wife S S Nishani, for the support and encouragement in all the way over the past few years. It is my pleasure to thank my fellow post-graduate students for their support and contribution to this research.

I would like to express my sincere appreciation to my previous employees, Stems Consultant(pte)Ltd and Fong Consults (pte) Singapore Ltd, for their kind understanding and granting me a releases from my duties to continue my studies.

I wish to express my gratitude to all those who have not mentioned here but helped for the success of this effort in many ways.

Finally, I would like to express my heartfelt thanks to my mother who scarified her whole life for my all those success. Although she is not with me to share this pleasure at this moment, I wish her to attain great nibbana.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATION

A	Area or Gross area of a cross-section
A_e	Effective net area of a section
A_{eff}	Effective area
A_n	Net area of a section
A_{st}	Area of an intermediate stiffener
A_t	Tensile stress area of a bolt
a	Effective throat size of a fillet weld
a_1	Net sectional area of connected elements
a_2	Gross sectional area of unconnected elements
B	Overall width of an element
B_f	Half the overall flange width of an element
b	Flat width of an element
b_{eff}	Effective width of a compression element
b_{er}	Reduced effective width of a sub-element
b_{eu}	Effective width of an unstiffened compression element
C_W	Warping constant of a section

c	Distance from the end of a beam to the load or the reaction
d	Overall web depth of lip Channel
b_2	Overall width of lip channel
b_3	Depth of lip for lip channel
d	Diameter of a bolt
E	Modulus of elasticity of steel
F_t	Applied tensile load
F_c	Applied axial compressive load
g	Gauge, i.e. distance measured at right angles to the direction of stress in a member, centre-to-centre of holes in consecutive lines
h	Vertical distance between two rows of connections in channel sections
I	Second moment of area of a cross-section about its critical axis
I_{min}	Minimum required second moment of area of a stiffener
I_x, I_y	Second moment of area of a cross-section about the x and y axes respectively
J	St Venant torsion constant of a section
K	Buckling coefficient of an element
L	Length of a member between support points
L_E	Effective length of a member
M	Applied moment on a beam

M_b	Buckling resistance moment
M_c	Moment capacity of a cross-section
M_{cr}	Critical bending moment causing local buckling in a beam
M_{cx}	Moment capacity in bending about the x axis in the absence of Fc and My
M_{cy}	Moment capacity in bending about the y axis in the absence of Fc and Mx
M_x, M_y	Moment about x and y axes respectively
P_{bs}	Bearing capacity of a bolt
P_c	Buckling resistance under axial load
V	basic wind speed
V_s	design wind speed
h	height of building structure
w	width of building
q	dynamic pressure of wind (stagnation pressure)
S_1	topography factor
S_2	ground roughness, building size and height above ground factor
S_3	a statistical factor

CHAPTER 01

1.0 INTRODUCTION

In steel construction, there are two main families of structural members. One is the familiar group of hot rolled shapes and members built up of plates (HRS). The other, less familiar but of growing importance worldwide, is composed of sections cold formed from steel sheet, strip, or flat bars in roll-forming machine or by press brake or bending brake operation. These are cold formed steel structural members (CFS). The thickness of steel sheet or strip generally used in cold formed steel structural members ranges from 0.4mm to 6.4mm.

Construction industry in Sri Lanka has experienced a rapid increase in construction cost due to lack of traditional construction material. Therefore an increasing demand for alternative construction material has been seen in the last few years. Cold Formed Steel (CFS) is still a new concept to Sri Lankan construction industry. Developers, Contractors, and designers are also affected by the non availability of extensive studies on applicability of cold form steel and what / how CFS can be used so that it benefits construction industry. This study was aimed to fulfill above mentioned industry requirements in a detailed manner.

1.1. GENERAL ADVANTAGES OF COLD FORMED STEEL

In general, cold formed steel structural members provide the following advantages in building construction.

- As compared with thicker hot rolled shapes, cold formed light members can be manufactured for relatively light loads.
- Unusual section configuration can be produced economically by cold forming operation and consequently favorable strength to weight ratios can be obtained.
- Nestable sections can be produced, allowing for compact packaging and shipping.

- compared to other material such as timber and concrete, the following qualities such as
    - Lightness
    - High strength and stiffness
    - Fast and easy erection and installation
    - More accurate detailing
    - Uniform quality
    - Recyclable material; can be achieved from cold formed steel structural members.

1.2. MATERIAL PROPERTIES OF STEEL

Two basic yield strength of steel are generally used in cold formed sections, as defined in BSEN 10147

Fe E 280G - yield strength 280 N/mm^2 (Formerly Z28)

Fe E 350G - yield strength 350N/mm^2 (Formerly Z35)

Generally the cold formed steel is available as Galvanized structural members. Galvanized steel for cold formed sections are normally provided in a grade of zinc coating, G275, which corresponds to 275g/m^2 total coverage of zinc summed over both sides of the sheet. This is equivalent to 0.04mm approximate total thickness, which is normally included in the specified thickness of the sheet. Galvanizing gives additional corrosion protection for members in internal conditions or subjected to intermittent moisture, for example due to condensation.

1.3. PROBLEMS OF COLD FORMED STEEL MEMBERS

1.3.1 Buckling Failure

Since the thickness of the cold formed steel members is small, they are subjected to local buckling, distortional buckling, flexural buckling, flexural torsional buckling or their combinations. Generally short compression members fail due to local buckling and / or distortional buckling while long compression members fail due to flexural buckling, flexural torsional buckling or their combination. Lack of design rules is also a major issue with design of cold formed steel members as they are limited to general external conditions, such as general environmental condition, normal temperature etc.

1.3.2 Low Fire Resistance

One of the major issues of steel structures is that they can be subjected to fire which will cause loss of lives and properties, not only because of fire but also due to the structural failure of building due to deterioration of the mechanical properties of the steel at elevated temperatures. Therefore fire safety design of building structures has received greater attention in recent time. Series of researches have been conducted on cold form steel behavior at elevated temperature and recommendation have identified to adopt in both design and construction stage.

1.4. APPLICATIONS OF COLD FORM STEEL STRUCTURES

1.4.1 Deck / Cladding Applications

Cold formed steel is used for deck, roof cladding and wall cladding applications. In the case of roof and wall cladding usually cold formed steel with a thickness of 0.42mm or 0.48 mm is used. Cold formed steel is also used for composite decks. These deck systems should have sufficient strength to withstand the wet concrete load, construction load, reinforcement load etc. Usually trapezoidal profile is used for composite slabs.

1.4.2 Floor Systems

Cold formed steel members are used as floor beams with plywood boards or oriented strand-boards. The use of such floor systems leads to light weight structures.

1.4.3 Use as Truss Members

There is an increasing trend of cold formed steel in truss systems in industrial application. This is basically due to the better quality of cold formed steel members in terms of strength, various shapes, pleasing aesthetics, ease of connections etc.. compared to hot rolled steel or timber members.

1.4.4 Other Applications

Cold formed steel members are used as door and window frames, storage racks, frames for display boards, etc.

1.5. OBJECTIVE

The main objective of this research is to assess the benefit of using cold formed steel members on medium span roof trusses against the hot rolled steel members and encourage the innovative development of cold formed steel roof trusses for medium span roof structures in Sri Lanka.

1.6. SCOPE

This study is based on four case studies, which contain 4.0m, 8.0, 10.0m and 12.0m span roof trusses. All existing proposal for the aforesaid roof trusses were based on Hot Rolled Steel sections and now all structures are in operation stage. This Detailed study will assess and compare the exact benefits if CFS members are used for the same intended usage of above structure.

Further, the member connections for truss element were limited to plate and bolt connection only, for CFS design. The main reason for this was, lack of skilled labour available in CFS construction industry in Sri Lanka, as well as easy installation and fabrication. The required parameters have been considered in CFS design stage for member connections.

By considering the availability of CFS section in Sri Lanka, the minimum section of 100x1.2-C was considered in the design. Again this research was limited to lipped channel sections only, as they are the most common sections available for cold formed construction in Sri Lanka.

The selection of truss type for particular span was depending on requirement raised by other parties that is Architects / Clients etc...

1.7. METHODOLOGY

In order to fulfill the above objectives, the following methodology was adopted.

I. Carried out a literature review

II. Identified the typical truss forms already adopted in Sri Lanka for industrial & commercial buildings

III. Analyzed the truss for permanent and wind load using finite element design software, SAP2000 as full scale 3D model. Analysis results were used in design of members, using separate spreadsheet developed, as illustrate in Appendix C.

IV. Determined the capacity of steel sections, and identified the optimum sections that can serve the intended usage for particular structure by both CFS and HRS. several iteration was carried out between analysis and design to figure out the optimum section for both steel family.

V. Determined the steel quantity requirement in each truss type and determined the cost efficiency for each truss forms for medium span roof truss.

VI. Compared the unit cost of roof structure under the same configuration for both type of steel roof structure.

VII. Identified limitation of using CFS for medium scale construction in Sri Lanka and recommendations were figure out.

CHAPTER 02

2.0 LITERATURE REVIEW

The use of thin material and cold formed processes results in several design problems for cold formed steel construction. The following is a brief discussion of some problems and solutions developed by means of research / experiments.

Material properties of structural members play an important role to achieve the intended usage of the structure. Most significantly, almost all the mechanical properties are affected by the temperature, the extreme conditions; that is $T < -34^0C$ & $T > 93^0C$ (American Iron and steel institute specification and supplement [AISI],1996), have given prior attention in thin gauge member design.

Most important properties of steel that can be affected by temperature are identified as follows,

- Yield strength
- Tensile strength
- Strength and strain characteristic
- Modulus of Elasticity
- Ductility
- Weldability
- Fatigue Strength

2.1. LOCAL BUCKLING AND POST BUCKLING STRENGTH.

2.1.1. Flexural Buckling.

Flexural buckling is the deflection caused by bending or flexure and occurs about the axis with the largest slenderness ratio. This failure mode is common in long columns.

2.1.2. Torsional Buckling.

This type of buckling only occurs in the compression members that are doubly-symmetrical and have very slender cross sectional elements. It is caused by twisting of the cross section about the longitudinal axis. Torsional buckling occurs mostly in built-up sections.

2.1.3. Flexural-Torsional Buckling.

This type of buckling occurs in compression members that have a cross section with only one axis of symmetry. Flexural-Torsional buckling is the simultaneous bending and twisting of a member and mostly occurs in channel, structural tees, double angle shapes and equal-leg single angles.

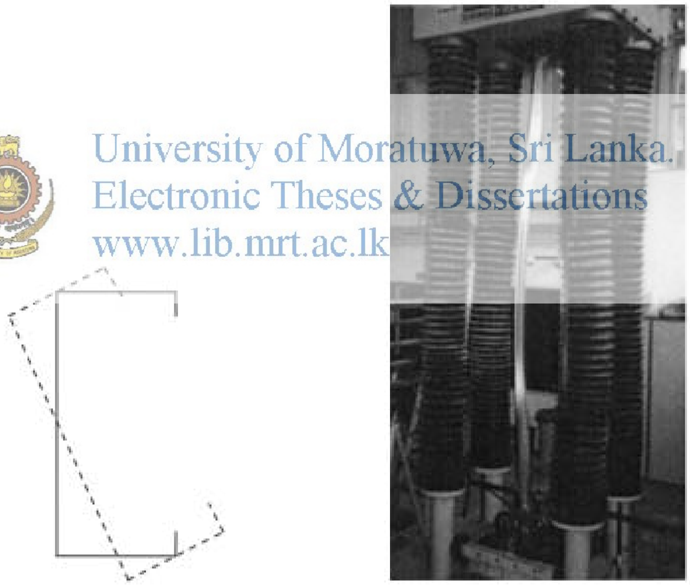


Figure 2-1 Flexural and Flexural-torsional Buckling (Young, 2005)

Elements that consist of thin walls when subjected to compression / bending or bearing, may buckle at stress levels less than the yield point of the material. When considering the CFS sections, the centroid and the shear center do not coincide with each other for most sections. Therefore torsional-flexural buckling be a critical factor for compression member design. Kwon and Hancock (1992) have carried out a research on thin walled channel section columns formed by brake processing. Two different sections, a simple lipped channel and a lipped channel with an intermediate stiffener in the web were tested between fixed-end boundary conditions. Distortional or mixed local-distortional buckling stresses obtained from testing are compared with theoretical buckling stresses. The result of compression tests showed that post buckling strength reserve occurred in the distortional mode and that local buckling occurred simultaneously at a shorter wave length.

2.2. GEOMETRIC IMPERFECTIONS AND RESIDUAL STRESSES.

Imperfection of steel member is very important as it reduces the capacity of the member. Two types of imperfections namely local imperfection and global imperfection, can be observed with cold-formed steel members. Local imperfection mainly affects the local or distortional buckling capacities while global imperfection mostly affected the member capacity. Local imperfections are expressed in terms of dimensions of the section and thickness while global imperfections are expressed in terms of the member length.

Finite element analysis needs imperfection to initiate the failure pattern. Finite element software assumes that imperfection follow a similar pattern of buckling wave with the amplitude of appropriate imperfection. Therefore accurate location of the maximum imperfection cannot be included in the analysis even if it measured. However the magnitude of imperfection is more important as the ultimate load is sensitive to that of imperfection magnitude.

2.2.1. Local Imperfections.

Local imperfection is the deformation of the plate element of the section. These deformations can occur during folding or handling. However, most accurate instrumentations are required to measure the actual initial local imperfection.

Walker (1975) derived an equation to predict the plate imperfection as given below. The recommended value for β is 0.3 by his study.

$$\Delta = \beta t \sqrt{\frac{p_y}{p_{cr}}}$$

Where; Δ - magnitude of initial imperfection.

t - thickness of the plate

β - constant that can be adjusted to fit experimental results.

p_y - yield or crushing load

p_c - elastic buckling load

Walker's (1975) proposal can be arranged by substituting the critical buckling load as given below.

$$\Delta = 0.3b\sqrt{\frac{12 f_y \left(1-\nu^2\right)}{k\pi^2 E}}$$

This equation is independent of the thickness of the plate element. Imperfections of cold-formed steel sections occur during the cold-forming process and handling. These deformations depend on the bending stiffness of the plate elements. Bending stiffness of the plate elements is a function of elastic modulus and second moment of inertia. Since the elastic modulus of steel is almost a constant, the imperfection will be a function of width, thickness and supporting type of plate elements.

Schafer and Pekoz (1998) developed the models for local imperfection after sorting available data from literature. These models consider the thickness, width and type of section. There are some limitations for the width to thickness ratio and for the thickness. The width to thickness ratio should be less than 200 for stiffened element and less than 100 for un-stiffened elements while thickness should also be less 3mm. Figure 2.2 below shows the type of imperfection and equation below give the value of imperfection.

$\Delta = 0.006w$ or $\Delta = 6te^{-2t}$ for stiffened elements

$\Delta = t$ for un-stiffened elements.

Δ - imperfection

w - width of the stiffened elements

t - plate thickness.

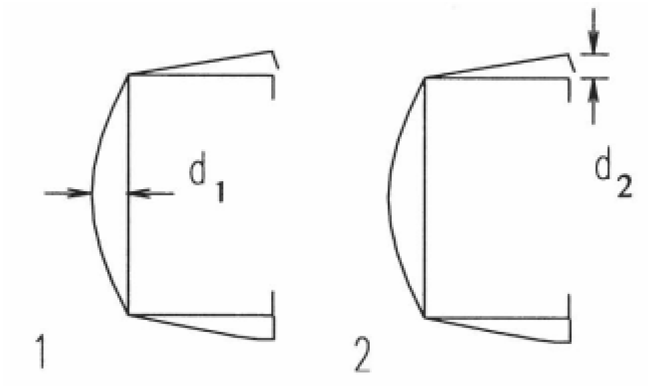


Figure 2-2 Imperfection Types Defined by Schafer and Pekoz (1998)

2.2.2. Residual Stresses.

There are two types of residual stresses known as flexural residual stress and membrane residual stress. Flexural residual stresses develop in the steel members due to the folding of section and are dominant in the cold-formed steel members. Cold forming induces flexural residual stresses in the cold formed steel members whereas membrane residual stresses develop due to the uneven cooling of the steel section during welding process. Usually cold-formed steel sections are not subjected to welding or heat treatment. Therefore flexural residual stresses are the most important for cold-formed steel members.

2.3. EFFECT OF ELEVATED TEMPERATURE

The effects of elevated temperature on the mechanical properties of steel and the structural strength of the steel have been subjected to a series of extensive investigation. Uddin and Culver (1975) presented the state of the art accompanied by an extensive list of reference. In addition, Klippstein (1978) has reported detailed studies of the strength of cold-form steel studs exposed to fire. The effect of the elevated temperature on the yield point, tensile strength, and modulus of elasticity of steel plate and sheet is shown graphically in Figure 2-3. It should be noted that when temperature are below zero, the yield point, tensile strength, and modulus of elasticity of steel are generally increased. However, the ductility and toughness are reduced. Therefore, great care must be exercised in designing cold form steel structure for extreme low temperature, particularly when subjected to dynamic load. (Hassinen, Helenius, Hieta & Westerlund, 1988)

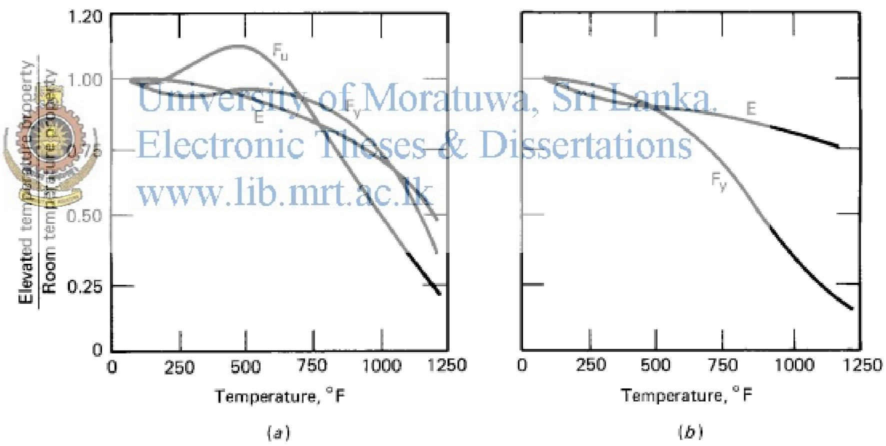


Figure 2-3 Effect of temperature on mechanical properties of low carbon steel (a)-steel plate. (b)-steel sheets

Chen & Young (2006) investigated the effect on the mechanical properties of corner region in cold-formed steel section at elevated temperature. They concluded that the reduction factors of yield strength, elastic modulus, and ultimate strength of corner region are similar to that of flat region. Further they concluded that elongation is also similar, provided steel temperature is greater than 180^0C. But below 1800C, total elongation of corner part is smaller than that of flat region. Their proposed equation for reduction factors for yield stress, elastic modulus, ultimate strength and strain corresponding to ultimate strength are given in Table 2.1

Table 2-1 : Reduction Factors of Mechanical Properties of Cold-Formed Steel at Elevated Temperatures (Chen & Young, 2006)

Reduction factors	Temperature, °C	a	b	c	n
For yield stress, $\frac{f_{y,T}}{f_{y,normal}} = a - \frac{(T-b)^n}{c}$	$22 \le T < 300$	1.0	22	$5.56\text{x}10^3$	1
	$300 \le T < 650$	0.95	300	$1.45\text{x}10^5$	2
	$650 \le T < 1000$	0.105	650	$5.00\text{x}10^3$	1
For elastic modulus, $\frac{E_T}{T_{normal}} = a - \frac{(T-b)^n}{c}$	$22 \le T < 450$	1.0	22	$1.25\text{x}10^3$	1
	$450 \le T < 650$	-0.11	860	-2.20x105	2
For ultimate strength, $\frac{f_{u,T}}{f_{u,normal}} = a - \frac{(T-b)^n}{c}$	$22 \le T < 450$	1.0	22	$5.6\text{x}10^8$	3
	$450 \le T < 1000$	0.043	1000	$-1.12\text{x}10^{11}$	4
For ultimate strain, $\frac{\varepsilon_{u,T}}{\varepsilon_{u,normal}} = a - \frac{(T-b)^n}{c}$	$22 \le T < 1000$	1.0	22	$1.0\text{x}10^6$	2

Mechanical and thermo-physical properties of hot rolled steel vary with the temperature. Mechanical properties do not vary linearly and their variation at elevated temperature is complex. Usually Mechanical properties of steel at elevated temperature are distributed in terms of reduction factors of the ambient temperature properties. Poisson’s ratio and density of steel are assumed to be constant at any temperature (Ranby, 1999 and Outinen & Myllymaki, 1995).

Eurocode 3 part 1.2 (ECS, 2005) provides the yield strength reduction factors for class 4 sections. But there is no difference in yield strength reduction factors between hot rolled and cold-formed steels. BS 5950: Part 8 (BSI, 1990) also provides yield strength reduction factors for cold-formed steel as given in Table 2.2. However, yield strength reduction factors are given at 0.5%, 1.5%, and 2.0% only.

Table 2-2 : Yield strength Reduction Factors for Cold-formed Steel (BSI, 1990)

Strain	Temperature (°C)								
%	200	250	300	350	400	450	500	550	600
0.5	0.945	0.890	0.834	0.758	0.680	0.575	0.471	0.370	0.269
1.5	1.000	0.985	0.949	0.883	0.815	0.685	0.556	0.453	0.349
2.0	1.000	1.000	1.000	0.935	0.867	0.730	0.590	0.490	0.390

2.4. THERMAL PERFORMANCE OF COLD-FORMED STELL UNDER FIRE.

When cold formed thin-walled steel sections are exposed to fire, they dissipate heat to the surrounding quickly, leading to rapid temperature increases. CFS sections which are employed on planar structural system are usually exposed to fire attack from one side, may cause a significant temperature gradient through the cross-section. Apart from that, a temperature gradient may also be present in the width direction because thin-wall members lose heat rapidly to their surroundings. As a result, temperature distribution in a thin-walled steel section can be highly non uniform under fire condition. Furthermore, the thermal performance of the steel section will depend on the presence of protective layer, usually made from gypsum boards.

Feng, Wang and Davies (2003) have carried out an extensive study on thermal performance of thin-walled steel channel sections in a planer system under fire attack from one side. The result from these investigations indicate that the thermal performance of cold-formed thin-walled steel channel wall panels are not significantly affected by the type of interior insulation and the shape of the cold-formed thin-walled steel cross-section. Temperature of the steel section of a Steel stud panel system, depend primarily on insulation panels on the fire exposed side. A cassette section, which has a wide web (300-400mm), two flanges and two narrow webs, is an alternative to conventional steel stud to eliminate cross-bracing. The thermal performance of this type of wall system is greatly affected by its layout, i.e. whether the continuous steel sheet is on the fire exposed side or the unexposed side. Temperatures in the steel cassette section are higher if the continuous sheet is on the fire exposed side.

Chen, Ye, Bai, and Zhao (2013) carried out extensive experiment in order to improve the fire performance of load-bearing CFS wall systems for applications in mid-rise building. Five types of protective layers were used in this experiment, gypsum plasterboard, bolivian magnesium board, oriented standard board (OSB), autoclaved lightweight concrete (ALC) board and rock wool boards. The result showed a noticeable reduction of heat transfer to the surface of steel stud and a considerable improvement of fire performance of CFS wall by using aluminum silicate wool as external insulation. Different load ratios may also result in different failure modes, and the fire resistant time can be more than 150min when the load ratio was less than 0.65. It was also demonstrated that the fire performance of CFS wall systems lined with bolivian magnesium board or ALC boards were superior to those lined with gypsum plaster board and OSB.

2.5. DIRECT STRENGTH METHOD

Series of researches have been sponsored by American Iron and steel institution on the direct strength method (DSM), a relatively new design method for CFS members validated for member without holes, predicts the ultimate strength of a general CFS column or beam with the elastic buckling properties of the member cross-section. Extended research project conducted by Moen and Schafer (2009) studied the appealing generality of DSM to cold-formed steel beam and column with perforations.

The elastic buckling properties of rectangle plates and cold formed steel beams and columns including the presence of holes, are studied with thin shell finite element analysis. Researches on cold form steel columns with holes are conducted to observe the interaction between elastic buckling load, load-deformation response, and ultimate strength. Parameter studies demonstrate that critical elastic buckling load either increase or decrease with the presence of holes, depending on the member geometry and hole size, spacing and location. The result from these experiments supplemented with existing beam and column data, guide the development of design equations relating to elastic buckling and ultimate strength for cold-formed steel members with holes. These equations and the simplified elastic buckling prediction methods will be presented as a proposed design procedure for an upcoming revision to the American Iron and Steel Institute's North American specification for the design of cold-formed steel structural members.

2.6. DESIGN RULES

2.6.1. BS5950 PART 5 (BSI, 1998)

Strength and Stiffness properties of both the gross section and effective section are required for design in order to overcome local buckling effect. Mid line idealization is the most acceptable method to evaluate the effective sectional properties. (Steel Construction Institute [SCI], 1993)

EFFECTIVE SECTION

Flat thin elements will buckle under compression due to their slenderness (as illustrated in Figure 2.4), while corners remain fully effective. The effective width of each flat element depends on the buckling coefficient, K, (Appendix B, BSI [1998]) which is a function of element type, section geometry and stress distribution. For simplicity, K can be taken as 4 for stiffened elements or 0.425 for un-stiffened elements. It is important to note that the mid-line idealized dimensions may be used to evaluate all the local buckling coefficients.

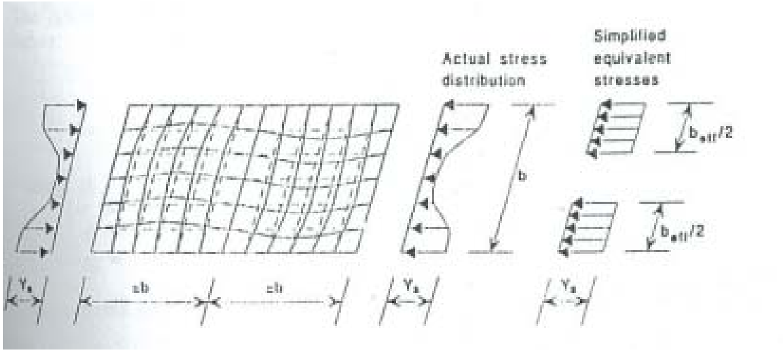


Figure 2-4 Illustration of effective width of a compression plate (SCI, 1993)

The basic effective width formula for all compression elements is given in BSI (1998), as below.

For $\frac{f_c}{p_{cr}} < 0.123$ $\qquad \frac{b_{eff}}{b} = 1$

(2a)

$$\frac{f_c}{p_{cr}} \geq 0.123 \qquad \frac{b_{eff}}{b} = \left[1 + 14\left\{\sqrt{\frac{f_c}{p_{cr}}} - 0.35\right\}^4\right]^{-0.2} \qquad (2b)$$

Where f_c - compression stress of the element.

Pcr - loacal buckling stress of the element, $185000K\left[\frac{t}{b}\right]^2$

b - flat width of the element

t - thickness of the element.

FLEXURAL BUCKLING CAPACITY.

BS 5950 Part 5 (BSI 1998) provides equations to calculate the flexural buckling capacity. In the case of flexural-torsional buckling, P_E in equation 2.1a needs to be replaced with the minimum value of elastic buckling load, P_E about the axis of symmetry and the elastic flexural torsional buckling load, P_{TF} from equation 2.2a.

$$P_C = \frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{CS}}} \qquad (2.1a)$$

where $$\phi = \frac{P_{CS} + (1+\eta)P_E}{2} \qquad (2.1b)$$

$$P_{CS} = A_{eff} P_Y \qquad (2.1c)$$

A_{eff} is effective cross sectional area

P_y is the design strength

E is the modulus of elasticity

I is the second moment of area of the cross-section about the critical axis

L_e is the effective length of the member about the critical axis

For $L/r \le 20$ $\qquad \eta = 0$ $\qquad$ (2.1e)

For $L/r > 20$ $\qquad \eta = 0.002(L_E/r - 20)$ $\qquad$ (2.1f)

Where,

r is the radius of gyration of the gross cross-section corresponding to P_E

Further

The buckling resistance capacity of singly symmetrical sections $P_c^{'}$ is given as below

$$P_c^{'} = \frac{M_C P_C}{(M_C + P_C e_s)} \qquad (2.1g)$$

Where

M_c is the moment capacity determined in accordance with Cl 5.2.2 (BSI,[1998]) having due regard to the direction of moment application as indicated in Figure 2.5

P_c buckling resistance calculated under equation (2.1a)

e_s is the distance between the geometric neutral axis of the gross cross-section and that of the effective cross-section as indicated in Figure 2.5

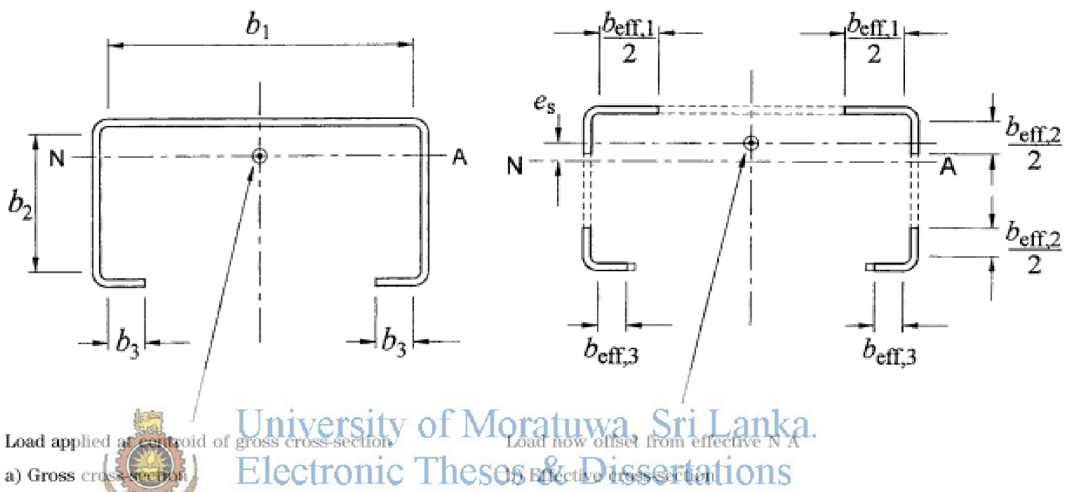


Figure 2-5 Compression of singly symmetrical section (BSI, [1998])

And,

$$P_{TF} = \frac{1}{2\beta}\left[(P_{EX} + P_T) - \{(P_{EX} + P_T)^2 - 4\beta P_{EX} P_T\}^{1/2}\right] \qquad (2.2a)$$

P_{EX} is the elastic flexural buckling load for a column about the x axis given by

$$\frac{\pi^2 EI}{L_E^2}$$

P_T is the torsional buckling load of a column given by,

$$P_T = \frac{1}{r_0^2}\left(GJ + 2\frac{\pi^2 EC_w}{L_E^2}\right) \quad (2.2b)$$

β is the constant given by $\beta = 1 - \left(\frac{x_0}{r_0}\right)^2$

r_0 is the polar radius of gyration about the shear centre given by

$$r_0 = (r_x^2 + r_y^2 + x_0^2)^{1/2}$$

r_x, r_y are the radii of gyration about the x and y axes;

G s the shear modulus;

x_0 is the distance from the shear centre to the centroid measured along the x axis,

J is the St Venant torsion constant which may be taken as the summation of $(bt^3)/3$ for all elements, where b is the element flat width and t is the thickness.

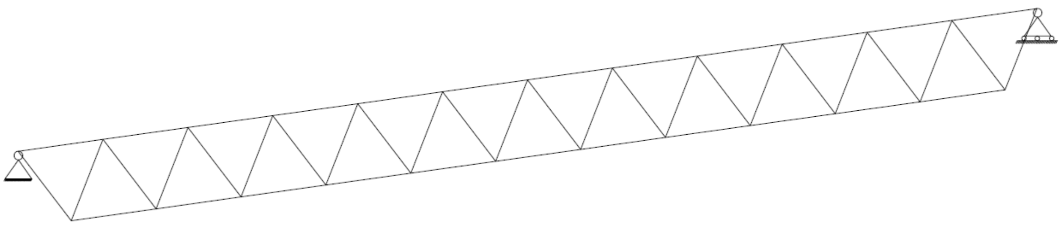
I_x is the second moment of area about the x-axis,

C_w is the warping constant for the cross section.

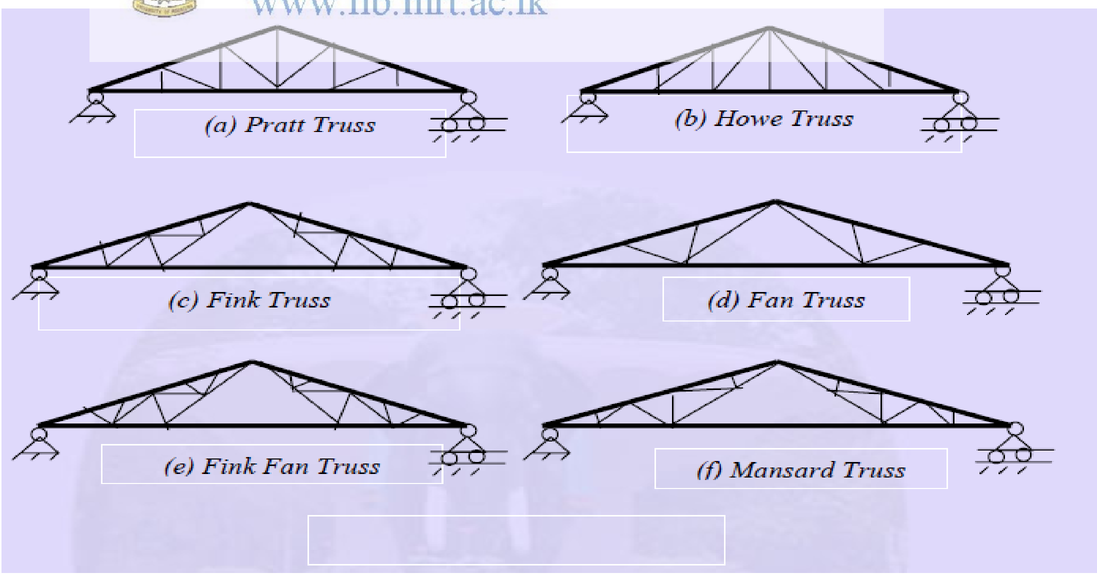
CHAPTER 03

3.0 TYPE OF TRUSSES AND STRUCTURAL IDEALIZATION

There are several types of trusses that have been using in roof structures in Sri Lanka. Figure 3.1 illustrate the most common truss types, identified after a survey of past-projects of which details are available in leading structural design offices in Sri Lanka.



a) Parallel Chord Truss



b) Pitched Roof Trusses

Figure 3-1 Type of Roof Trusses; a) Parallel Chord Truss b) Pitched Roof Trusses

Three buildings were selected for this study, which are situated in different part of Sri-Lanka, included a post disaster structure, Commercial Structure and normal structure. These structures are spread over the two different wind zones and comprise of parallel chord trusses and Pratt truss, having varied span from 4.0m to 12.0m.

The selected spans for this detail study were 4.0m, 8.0m, 10.0m, and 12.0m only. Bay spacing for above spans were 3.0m, 4.0m, 3.2m, and 6.0m respectively.

The Geometry of roof trusses for both steel families was kept exactly same for this study. That includes the bay spacing, purlin and tie rod spacing. Therefore, the design requirement such as direct stress and local buckling failure were addressed by selected design sections.

3.1. STRUCTURAL IDEALIZATION

Pratt truss system was use for 4.0m, 8.0m, and 10.0m span roof structure while parallel chord truss system was used for 12.0m span roof structure. The selected truss types for particular roof structures were analyzed for permanent and wind loads using finite element analysis software, SAP 2000-14. Full 3D analysis was carried out to study the exact structural response under static and wind loads & it's combinations, for both hot rolled and cold-formed steel sections.

3.1.1 COMPUTER MODELING

The finite Element software SAP2000-14 was used to model the roof structure. Eight basic trusses under four categories of span and two steel families were modeled. The bay spacing were defined according to the building geometry and column grid, as shown in Annex - A. Minimum head room requirement and roof pitch were maintained according to initial building requirement.

The top and bottom chord of each truss type were defined as continuous member, while bracing members are defined as individual section. The support conditions at one end, of each truss were defined as a pinned support while other end defined as roller support.

3.1.2 LOAD EVALUATION

The design loading for this study was selected to comply with BS6399-I; code of practice for dead and imposed loads, and other super imposed load requirement were defined to comply with building services and M & E requirements. The gravity loads evaluation according to roofing requirement is shown in Annex-B.2

The wind load was evaluated based on design wind speed in particular wind zone and building category as shown in Annex-B.1. The structures for this study were selected, such that it covers most common building category in Sri Lanka. Following building categories were covered, under this study.

- Normal structure
- Commercial structure
- Post disaster structure

The load combination used for 3D analysis is shown in Table 3.1

Table 3-1 : Combination Definitions

Combo Name	Combo Type	Auto Design	Case Type	Case Name	Scale Factor
SDL (Service Dead-Live Load)	Linear Add	No	Linear Static	DEAD	1.0
			Linear Static	LIVE	1.0
			Linear Static	SUPER DEAD	1.0
UDL Ultimate Dead-Live load)	Linear Add	No	Linear Static	DEAD	1.4
			Linear Static	SUPER DEAD	1.4
			Linear Static	LIVE	1.6
ULDW(max) (Ultimate Dead-Live-Wind [Inward] load)	Linear Add	No	Linear Static	DEAD	1.2
			Linear Static	LIVE	1.2
			Linear Static	SUPER DEAD	1.2
			Linear Static	Wind(max)	1.2
UDLW(min) (Ultimate Dead-Live-Wind [outward] load)	Linear Add	No	Linear Static	DEAD	1.2
			Linear Static	LIVE	1.2
			Linear Static	SUPER DEAD	1.2
			Linear Static	Wind(min)	1.2
ENV	Envelope	No	Linear Static	DEAD	1.0
			Linear Static	LIVE	1.0
			Response Combo	SDL	1.0
			Linear Static	SUPER DEAD	1.0
			Response Combo	UDL	1.0
			Response Combo	UDLW(min)	1.0
			Response Combo	ULDW(max)	1.0

CHAPTER 04

4.0 STRUCTURAL ANALYSIS

Iteration between analysis and design was carried out such that design sections and analysis sections are tally on each other. Then structural analysis results were compared for each case to figure out the exact difference of structural behavior between two steel families of CFS & HRS under this chapter. The Deflection and lateral stability was given high important in terms of stability and serviceability.

4.1. ANALYSIS RESULT COMPARISON

Following Sign Convention will be used hereafter to interpret the Analysis result in proper way.

- Compression - Negative(-)
- Tension - Positive(+)

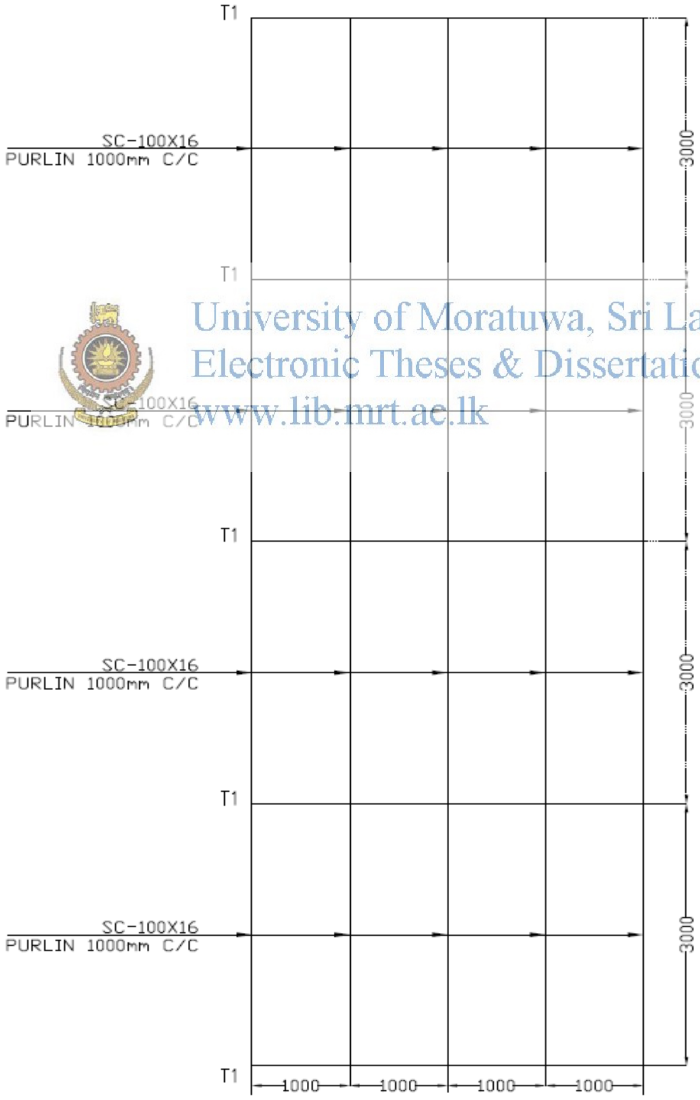
4.1.1 4.0m SPAN ROOF STRUCTURE

The general geometry of 4.0m span structure is shown in *Figure 4-1* below. Exactly same geometry was implemented on both CFS & HRS roof structures. The bay spacing for roof trusses were 3.0m according to column grid of the building and four bays were included in 3D analysis.

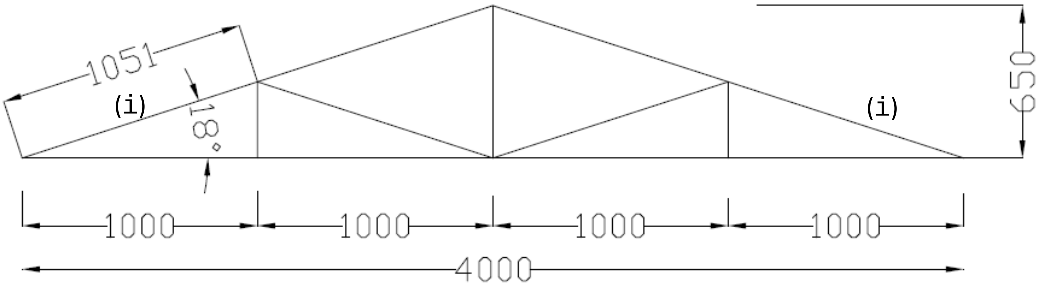
Design section and analysis sections were verified that they converge on each other after few iteration between design and analysis. The following *Table 4.1,* shows the selected optimum design sections for both CFS and HRS roof structure.

Table 4-1 : Optimum Design section for 4.0m Span Roof Truss

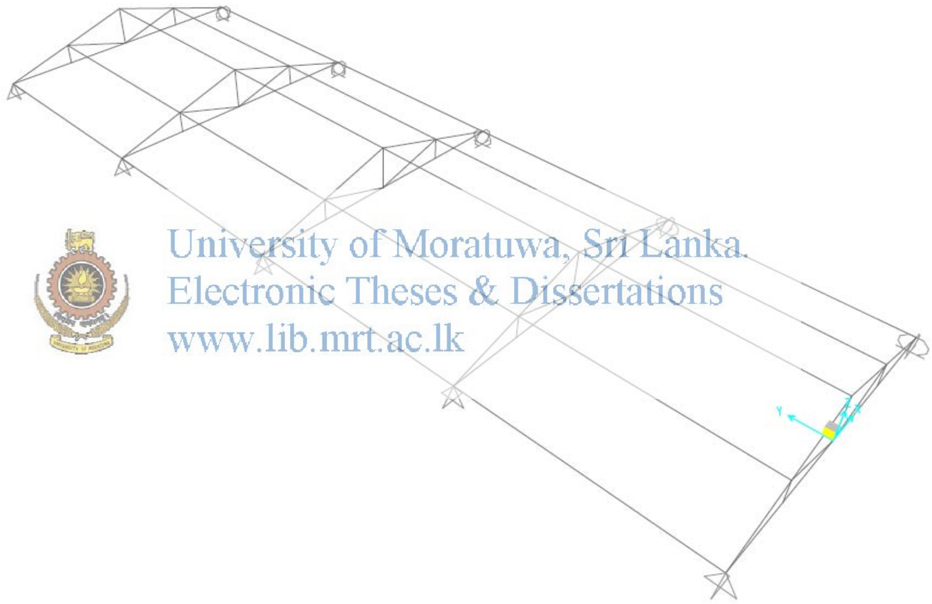
Design Sections for CFS & HRS Roof Trusses		
Roof Truss Member	**HRS Roof Structure**	**CFS Roof Structure**
Top Member	2/50x50x5mm	C 100x50x20mm
Bottom Member	2/50x50x5mm	C 100x50x20mm
Bracing Member	50x50x5mm	C 100x50x20mm



a) - Plan View



b) - Sectional View



c) 3D View of Roof Structure

Figure 4-1: 4.0m Span Truss - a) Plan View - b) Sectional View - c) 3D View of Roof Structure

The main structural system in the above roof structure was steel truss system. All structural limit requirements depend on the capacity and ability of resistance to design loads by truss system and its member. Detail discussion and comparison will be carried out hereinafter on truss members for much better understanding of differences of structural behaviour between two structural roof systems, CFS & HRS. Further, the highly stressed members in any roof truss are top and bottom members of that truss. In addition, internal members also checked and presented in Annex C, since it would be also critical due to its slenderness.

The forces in the bottom members of the truss, are given in *Table 4-2,* were compared under the envelope Load case (ENV) which was explained in *Table 3-1.*

Table 4-2 : 4.0 m Span Truss; Element Forces-Bottom Chord

	Element Forces - Frames -bottom chord (Combo - ENV)			
		Maximum Force(Tension)		
	Element Label	Cold formed (C)	Hot Rolled (H)	Ratio
		kN	kN	C/H
Outer most truss	16	12.5	15.1	0.83
	17	12.4	14.9	0.83
	18	12.4	14.9	0.83
	19	12.5	15.1	0.83
1st interior truss	23	21.1	24.8	0.85
	24	21	24.5	0.86
	25	21	24.5	0.86
	26	21.1	24.8	0.85
interior truss	62	21.1	24.8	0.85
	63	21	24.5	0.86

Each truss having four members at bottom chord and 5-trusses have been employed over the four bays. The variation of bottom member forces is shown in *table 4-2* above. By observing the load transfer behavior, it is obvious that the outer most truss would be loaded approximately half than interior truss. Assessing on the analysis result, it is shown that difference of the member forces between two steel families are getting lower when trusses are physically exposed to high level of external forces. Apart from that there is no much variation based on trust location whether it is outermost or interior truss. CFS roof system always shows less member forces which leads to "less stressed structural member" than HRS system and vulnerability of localized member failure hence reduce in CFS system.

Table 4-3 show the variation of member forces at Top chord for two families. Member forces under the envelope load case (ENV) were compared each other.

Table 4-3 : 4.0 m Span Truss; Element Forces-Top chord

	Element Forces - Frames -Top chord (Combo - ENV)			
		Minimum Force (Compression)		
	Element Label	Cold formed (C)	Hot Rolled (H)	Ratio
		kN	kN	C/H
Outer most truss	6	-13.5	-16.1	0.84
	12	-10.8	-11.3	0.96
	13	-10.8	-11.3	0.96
	14	-13.5	-16.1	0.84
1st interior truss	15	-22.7	-26.4	0.86
	20	-18	-18.3	0.98
	21	-18	-18.3	0.98
	22	-22.7	-26.4	0.86
interior truss	58	-22.7	-26.4	0.86
	59	-18	-18.3	0.98

Above member forces variation shows that top chord of each trusses subjected to high compression forces. The pattern distributions of forces are much differs from the bottom members. This pattern distribution of top members shows that compression forces at the support members [Member (i) shows in figure 4-1; b] are higher than the middle members. But in bottom members which are mainly subjected to tension forces shows flat distribution all the way support to support.

Here again compression forces in CFS top members are lesser than that of HRS, the absolute difference between forces of two steel families is lesser than that of bottom members. Force envelope for CFS members is closely follows the HRS force envelope in top members. But it is still providing a "less-stressed" structural system with CFS members, under exactly same imposed and super imposed load.

4.1.2 8.0m & 10.0m SPAN ROOF STRUCTURE

The general geometry of 8.0m & 10.0m span truss, for both CFS & HRS is shown in *Figure 4-2* below. The bay spacing for these trusses was 4.0m & 3.6m respectively, according to column grid of the building and four bays were included in 3D analysis.

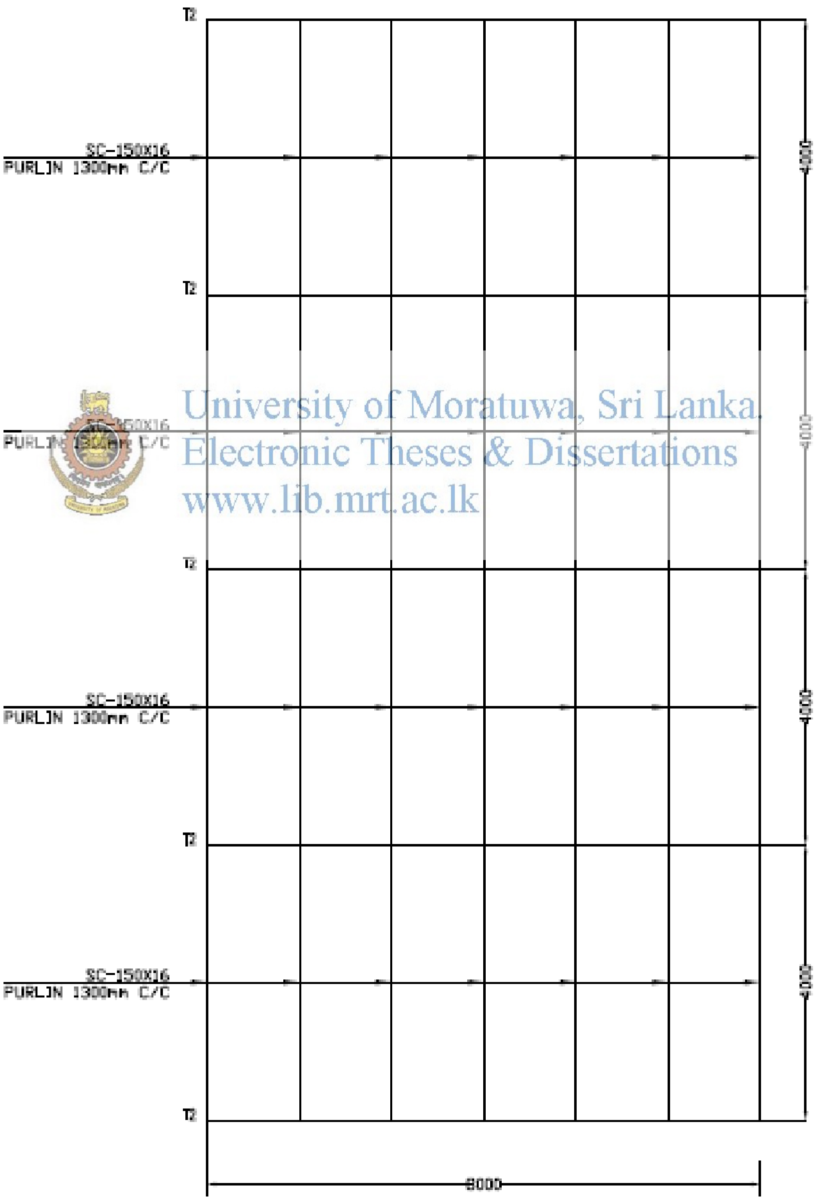
Design section and analysis sections were verified that they converge on each other after few iteration between design & analysis. The following Table 4-4& Table 4-5 , shows the selected optimum design sections for both CFS and HRS roof structure.

Table 4-4 : Optimum Design section for 8.0m Span Roof Truss

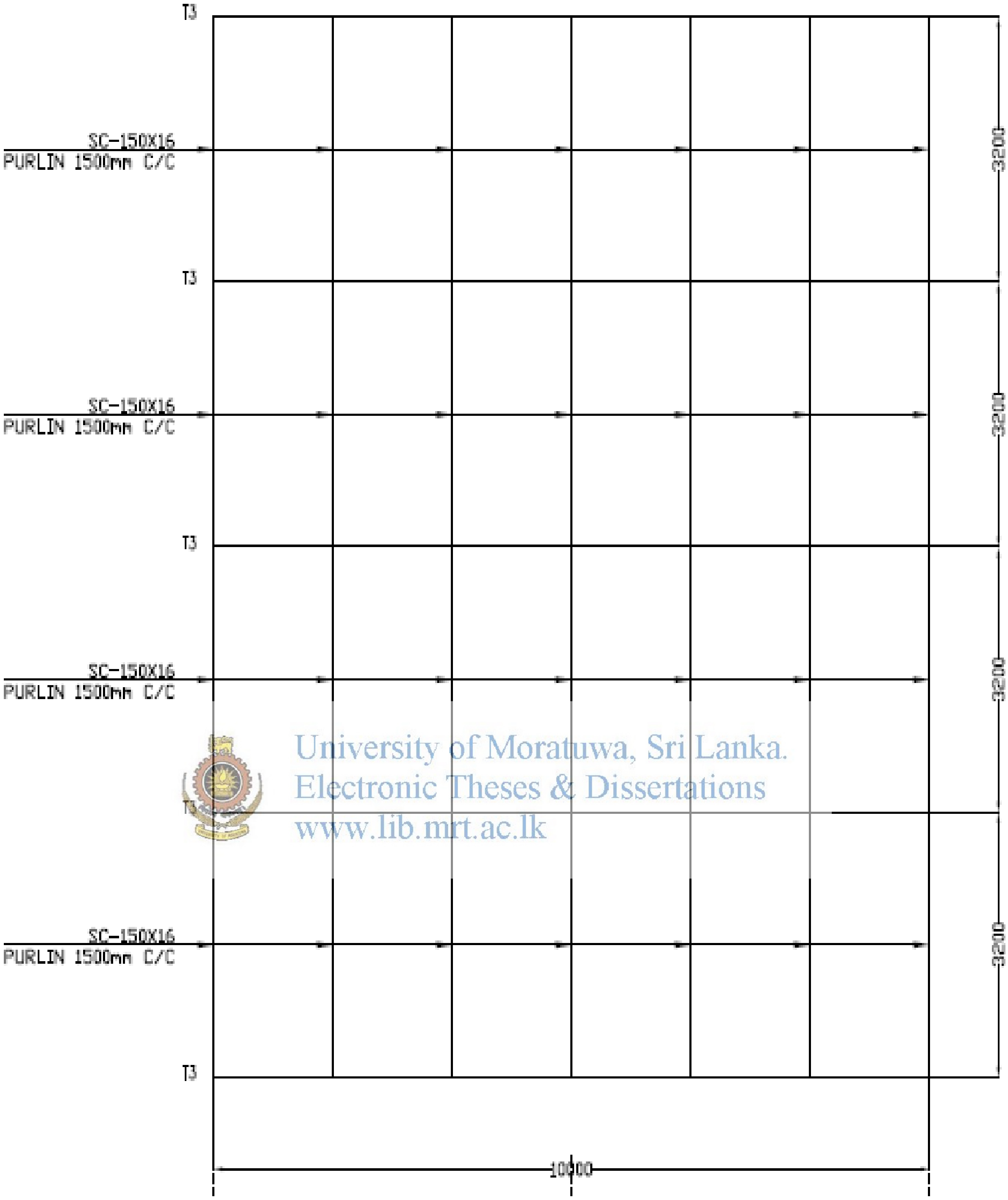
Design Sections for CFS & HRS Roof Trusses		
Roof Truss Member	HRS Roof Structure	CFS Roof Structure
Top Member	2/75x50x8mm	C 150x65x20mm
Bottom Member	2/75x50x8mm	C 150x65x20mm
Bracing Member	75x50x6mm	C 100x50x20mm

Table 4-5 : Optimum Design section for 10.0m Span Roof Truss

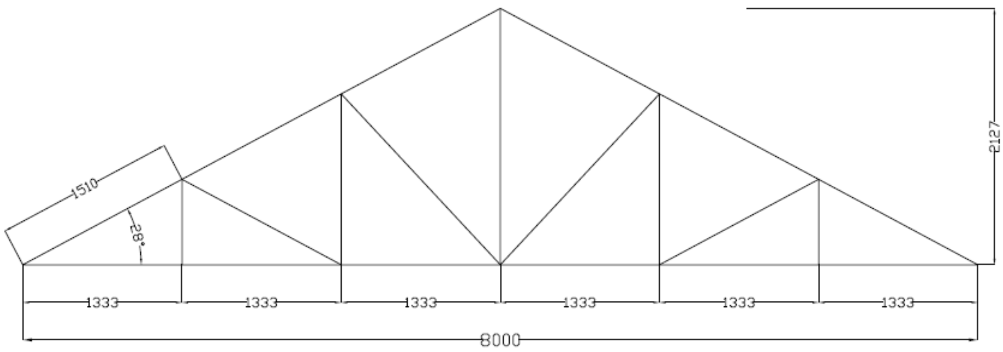
Design Sections for CFS & HRS Roof Trusses		
Roof Truss Member	HRS Roof Structure	HRS Roof Structure
Top Member	2/75x75x6mm	C 150x65x25mm (1.8 mm thck)
Bottom Member	2/75x75x6mm	C 150x65x25mm (1.8 mm thck)
Bracing Member	2/50x50x6mm	C 100x50x20mm (1.2 mm thck)



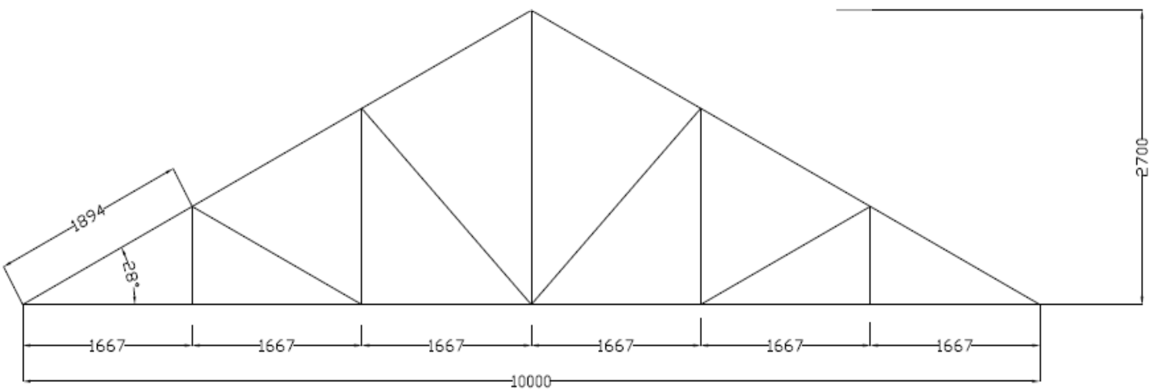
a) Plan View -8.0m Span Roof Structure



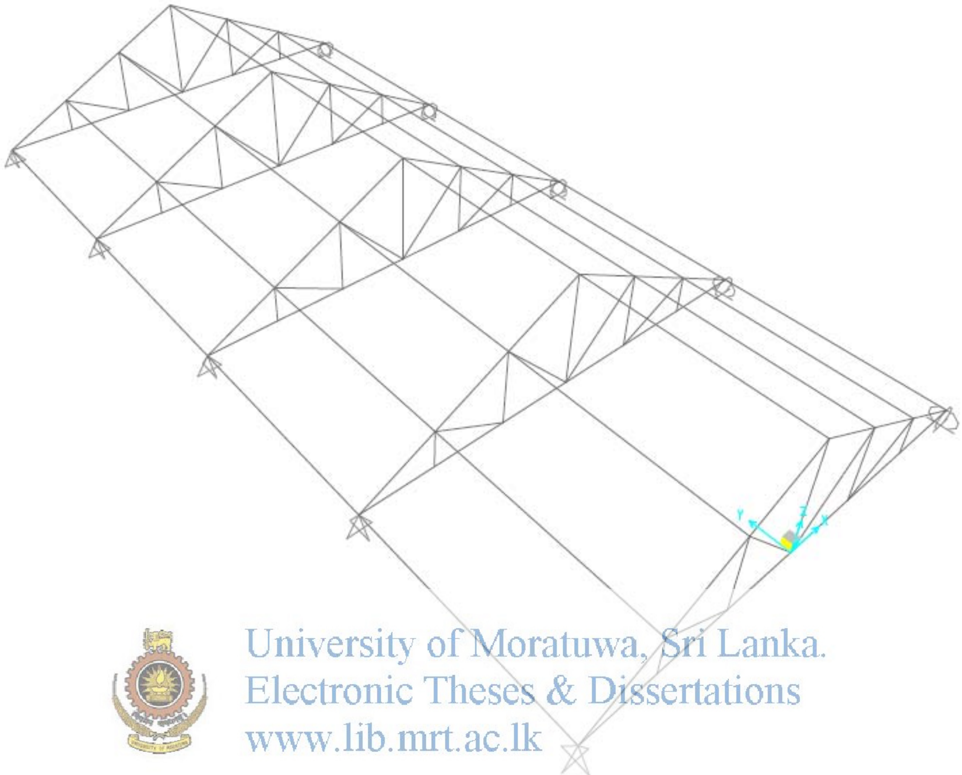
b) Plan View -10.0m Span Roof Structure



c) Sectional View - 8.0m Span roof Structure



d) Sectional View - 10.0m Span roof Structure



e) 3D View of Roof Structure

Figure 4-2: a) Plan View-8.0m Span Roof Structure - b) Plan View-10.0m Span Roof Structure - c) Sectional View-8.0m Span Roof Structure - d) Sectional View-10.0m Span Roof Structure

e) 3D View of Roof Structure

Detail discussion and comparison on truss member foces will be carried out for much better understanding of differences of structural behaviour between two structural roof systems, CFS & HRS.

The Table 4-6 show the variation of member forces at bottom chord of roof truss, for two steel families.

Table 4-6 : 8.0 m & 10.0m Span Truss; Element Forces-Bottom chord

Member Forces - Frames -bottom chord-(COMBO-ENV)							
		8.0m (Tension)			**10.0m** (Tension)		
	Element Label	**Cold formed**	**Hot Rolled**	**Ratio**	**Cold formed**	**Hot Rolled**	**Ratio**
		kN	kN	C/H	kN	kN	C/H
Outer most truss	1	28	32.4	0.86	29.3	34.9	0.84
	2	28.2	32.3	0.87	29.4	34.8	0.84
	3	22.1	24.6	0.90	22.7	26.4	0.86
	4	22.1	24.6	0.90	22.7	26.4	0.86
	5	28.2	32.3	0.87	29.4	34.8	0.84
	6	28	32.4	0.86	29.3	34.9	0.84
1st interior truss	7	44.8	49.7	0.90	44.8	48.6	0.92
	8	45.2	49.5	0.91	44.9	48.5	0.93
	9	36.1	38.2	0.95	35.4	37.4	0.95
	10	36.1	38.2	0.95	35.4	37.4	0.95
	11	45.2	49.5	0.91	44.9	48.5	0.93
	12	44.8	49.7	0.90	44.8	48.6	0.92
Interior truss	13	41	46.1	0.89	42.9	47.9	0.90
	14	41.4	45.9	0.90	43	47.9	0.90
	15	33.1	35.6	0.93	34	37	0.92

Each truss having six members at bottom chord and 5-trusses have been employed over the four bays. Assess on the analysis result, is shown that bottom members of each truss exposed to tensile forces with approximately half value at outer most truss than that of interior. Force pattern (maximum-Tension) within the members for two steel families are shown much similar distribution each other than 4.0m span truss. when compare the absolute level of different of tension forces between two steel families, for 4.0m, 8.0m & 10.0m trusses, it shows that force difference is higher in 8.0m, & 10.0m span than 4.0m span, and CFS members always follows the lower force intensity than HRS. This is similar advantage but in higher order, which observed under 4.0m span roof trusses.

Table 4-7 and show the variation of member forces at top chord for both steel families.

Table 4-7 : 8.0 m & 10.0m Span Truss; Element Forces-Top chord

Member Forces - Frames -Top chord-(COMBO-ENV)							
		8.0m (compression)			**10.0m** (compression)		
	Element Label	**Cold formed**	**Hot Rolled**	**Ratio**	**Cold formed**	**Hot Rolled**	**Ratio**
		kN	kN	C/H	kN	kN	C/H
Outer most truss	1	-33	-37.8	0.87	-34.6	-40.8	0.85
	2	-28.7	-31.4	0.91	-29.5	-33.8	0.87
	3	-23.2	-25.1	0.92	-23.8	-26.9	0.88
	4	-23.2	-25	0.93	-23.7	-26.8	0.88
	5	-28.7	-31.3	0.92	-29.5	-33.6	0.88
	6	-33	-37.7	0.88	-34.6	-40.6	0.85
1st interior truss	7	-52.1	-57.5	0.91	-52.2	-56.4	0.93
	8	-44.7	-47.2	0.95	-44.1	-46.3	0.95
	9	-35.4	-37	0.96	-34.8	-36.3	0.96
	10	-35.4	-36.8	0.96	-34.7	-36.2	0.96
	11	-44.7	-47.1	0.95	-44	-46.2	0.95
	12	-52.1	-57.4	0.91	-52.2	-56.2	0.93
Interior truss	13	-47.7	-53.3	0.89	-50.1	-55.7	0.90
	14	-41.1	-44	0.93	-42.4	-45.7	0.93
	15	-32.7	-34.6	0.95	-33.7	-35.9	0.94

This pattern distribution of top members shows that compression forces at truss support members are higher than the middle members, similarly as 4.0m truss member forces in member (i) [Figure 4-1; b]

The compression forces in all CFS top members, are lesser than that of HRS, the absolute difference between forces of two steel families is much closer to that of bottom member and in higher order than 4.0m span roof structure. This is mainly due to the high load resisting area on 8.m & 10.0m trusses than 4.0m span truss. Member force distribution pattern for CFS members is closely follows the HRS force distribution pattern in top member. But it is still providing a "less-stressed" structural system with CFS members, under exactly same imposed and super imposed load.

4.1.3 Deflection Check for 4.0m to 10.0m Span roof structure

Any roof structure must satisfy the strength limit state, in which each member is proportioned to carry the design load to resist buckling, yielding, etc.; and serviceability limit state which define functional performance and behavior under load and include such items as deflection, vibration, etc. Therefore Steel roof trusses under normal loading should be checked for the deflection. Since deflection at the mid span of the truss would prone to have higher value than expected value for individual members, a serviceability performance satisfactorily without causing any discomfort for the occupants of the structure. Therefore, the selection of truss members should be directly addressed by deflection requirement. Deflection was considered as governing criteria and especially CFS members were identified weak in deflection.

According to the International Building code (ICB2000) some of the typical deflection limits for steel roof structures are as follow.

Table 4-8 : Deflection Limits

Members	Max. Live Load defl.	Max. dead+live load defl.
Roof Beam:		
Supporting plaster ceiling	L / 360	L / 240
Supporting non-plaster ceiling	L / 240	L / 180
Not supporting a ceiling	L / 180	L / 120
Floor Beam	L / 360	L / 240

Note: L = Span Length

The deflection was considered under main two conditions which pulins are continuous over the trusses & simply supported at each truss location. Those conditions were adopted to ensure the roof structure to be at its serviceability limit, under any conditions, which could be used at construction stage.

Table 4-9 & Figure 4-3 below shows the vertical deflection of each truss for both steel families under the condition of purlins are continuous.

Table 4-9 : Maximum Vertical Deflection at Mid Span – 4.0m, 8.0m & 10.0m Span Roof Structures - Purlin continuous over trusses

TABLE: Vertical Deflection at the Mid Span of truss							
		4.0m Span Roof Truss		8.0m Span Roof Truss		10.0m Span Roof Truss	
Truss Location	**Output Case**	**CFS (Z-Axis)**	**HRS (Z-Axis)**	**CFS (Z-Axis)**	**HRS (Z-Axis)**	**CFS (Z-Axis)**	**HRS (Z-Axis)**
	Text	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**
Outer Most Truss	SDL	-1.037	-0.381	-1.503	-0.493	-1.752	-0.741
1st Interior Truss	SDL	-2.333	-0.798	-3.794	-1.095	-3.997	-1.345
Interior Truss	SDL	-2.099	-0.914	-3.287	-0.971	-3.808	-1.318
1st Interior Truss	SDL	-2.333	-0.798	-3.794	-1.095	-3.997	-1.345
Outer Most Truss	SDL	-1.037	-0.381	-1.503	-0.493	-1.752	-0.741

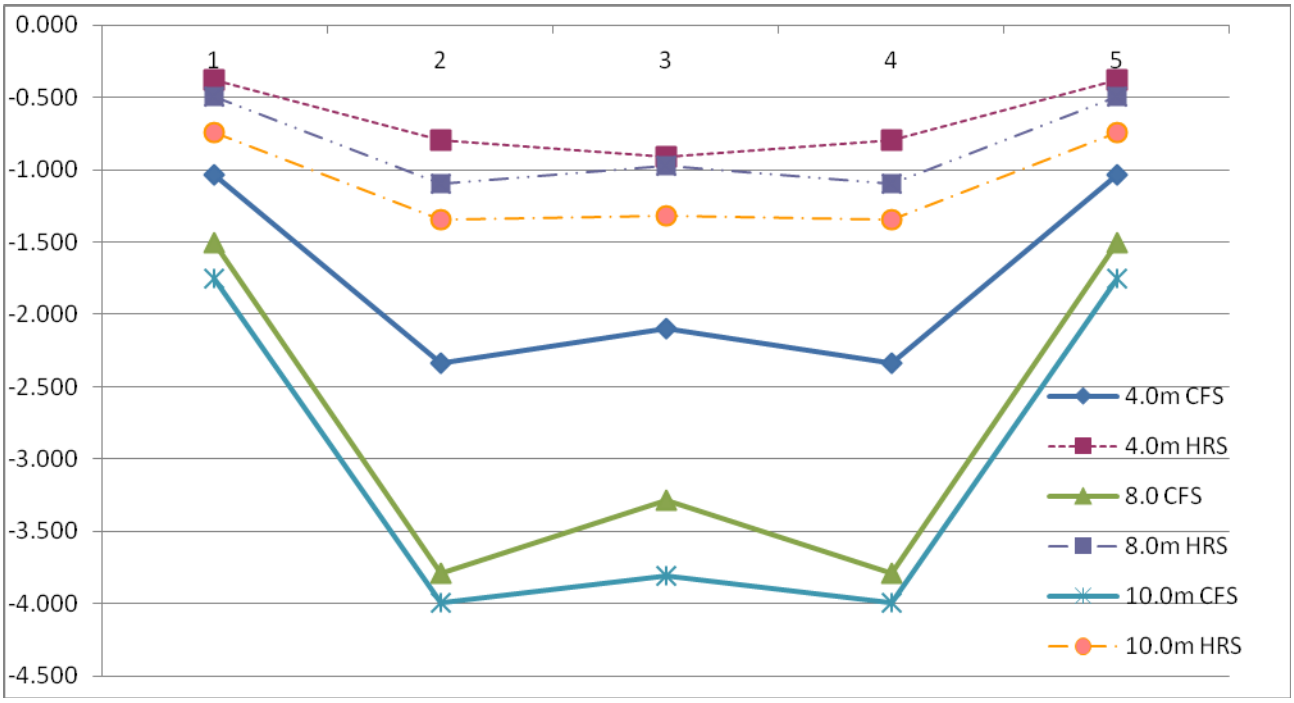


Figure 4-3: Variation of Deflection at Mid Span of Trusses

Table 4-10 & Figure 4-4 below shows the vertical deflection of each truss for both steel families under the condition of purlins are discontinuous at each truss locations.

Table 4-10 : Maximum Vertical Deflection at Mid Span - 4.0m, 8.0m & 10.0m Span Roof Structures - Purlin discontinuous over trusses

TABLE: Vertical Deflection at the Mid Span of truss							
		4.0m Span Roof Truss		8.0m Span Roof Truss		10.0m Span Roof Truss	
Truss Location	**Combi-nation**	**CFS (Z-Axis)**	**HRS (Z-Axis)**	**CFS (Z-Axis)**	**HRS (Z-Axis)**	**CFS (Z-Axis)**	**HRS (Z-Axis)**
	Text	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**	**U3 mm**
Outer Most Truss	SDL	-1.249	-0.445	-1.780	-0.561	-1.964	-0.763
1st Interior Truss	SDL	-2.436	-0.832	-3.441	-1.008	-3.736	-1.321
Interior Truss	SDL	-2.436	-0.832	-3.441	-1.008	-3.736	-1.321
1st Interior Truss	SDL	-2.436	-0.832	-3.441	-1.008	-3.736	-1.321
Outer Most Truss	SDL	-1.249	-0.445	-1.780	-0.561	-1.964	-0.763

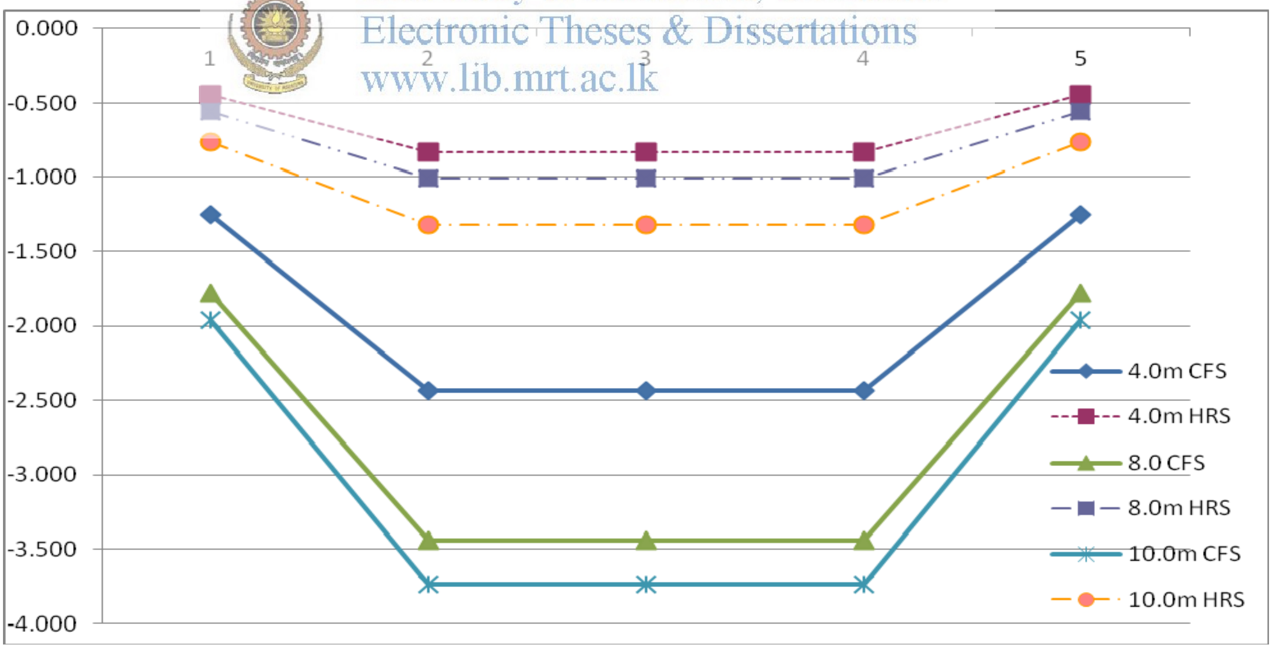


Figure 4-4: Variation of Deflection at Mid Span of Trusses

The purpose of compare the deflection, for both purlin continuous over trusses and discontinuous over trusses were identify the most critical occurrence of deflection. Although the most practical way of fixing purlins would be discontinuous over the truss supports, in the event of having closer bay spacing would give an opportunity to make purlins continuous over few truss support. When compare the results for above two cases, it was obvious that the most critical deflection has been occurred at the first interior truss, when purlins are continuous over its supports. Since it is essential requirement to satisfy the serviceability requirement by both steel families, Figure 4-3 & Figure 4-4 shows the variation of maximum deflection for each roof trusses at its mid span, over the whole roof structure. Although the CFS roof system shows a larger deflection than HRS roof system the maximum deflection value in CFS structure is well within the allowable limit (Table 4-8).

$$y_{max} = \Delta_{max} \leq \Delta_{allowable} = \frac{l}{value}$$

Comparing the both 8.0m & 10.0m truss deflection, it shows that the deflection for both spans is in similar order. Although the value of deflection is well bellow the requirement given in Table 4-8, further comparison will be carried out for 12.0m span roof systems, in terms of serviceability requirement.

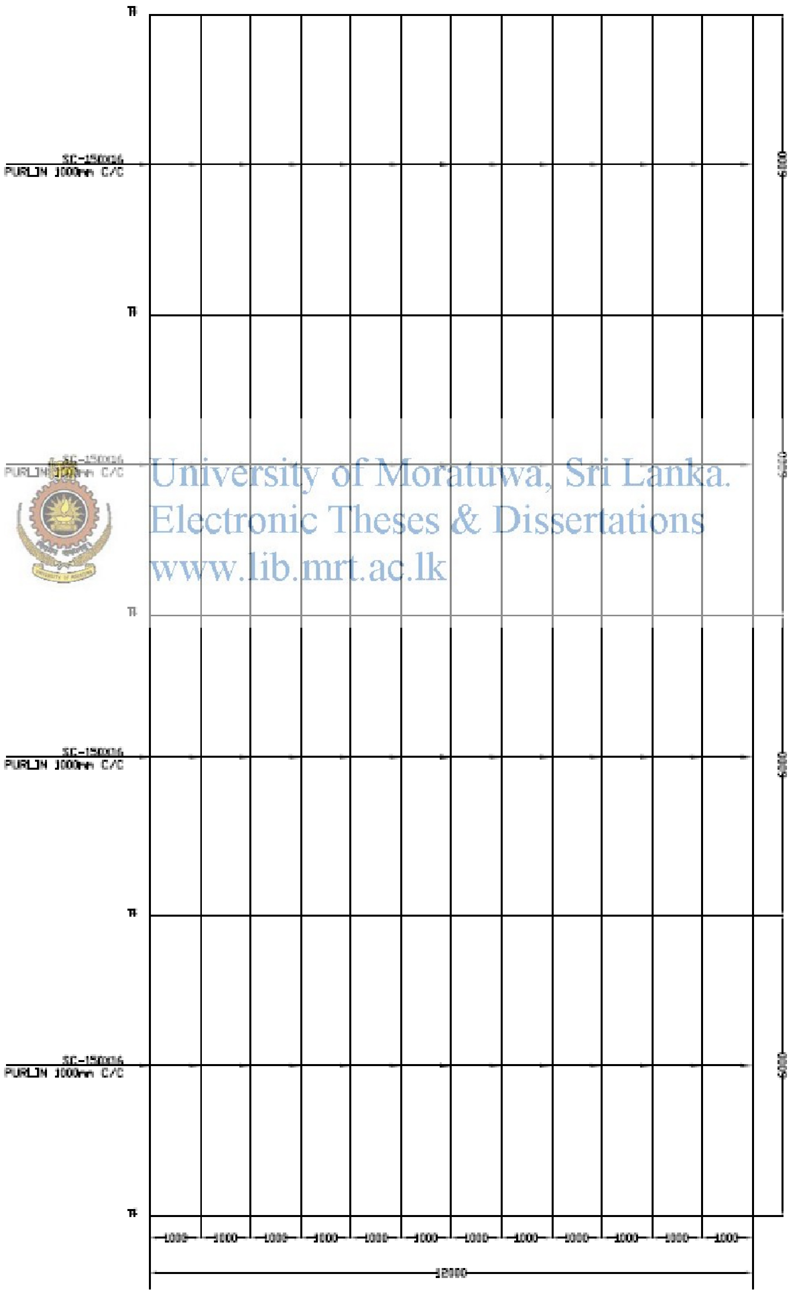
4.1.4 12.0m SPAN ROOF STRUCTURE

The general sectional Geometry of 12.0m span truss, for both CFS & HRS is shown in Figure 4-5 below. The bay spacing of the trusses were govern by the column grid of the building and was set as 6.0m.

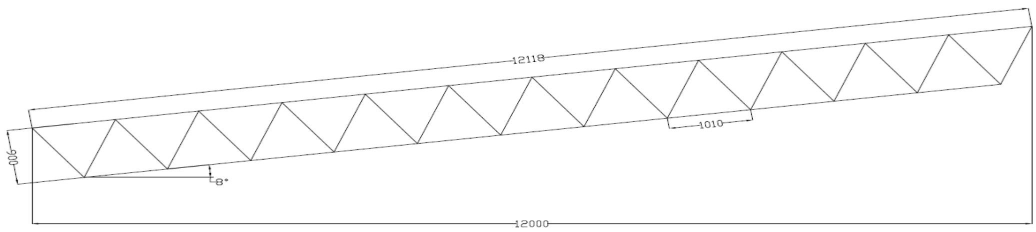
Design sections and analysis sections were verified that they converge on each other after few iterations between design and analysis. The following Table 4-11, shows the selected optimum design sections for both CFS and HRS roof structure.

Table 4-11 : Optimum Design section for 12.0m Span Roof Truss

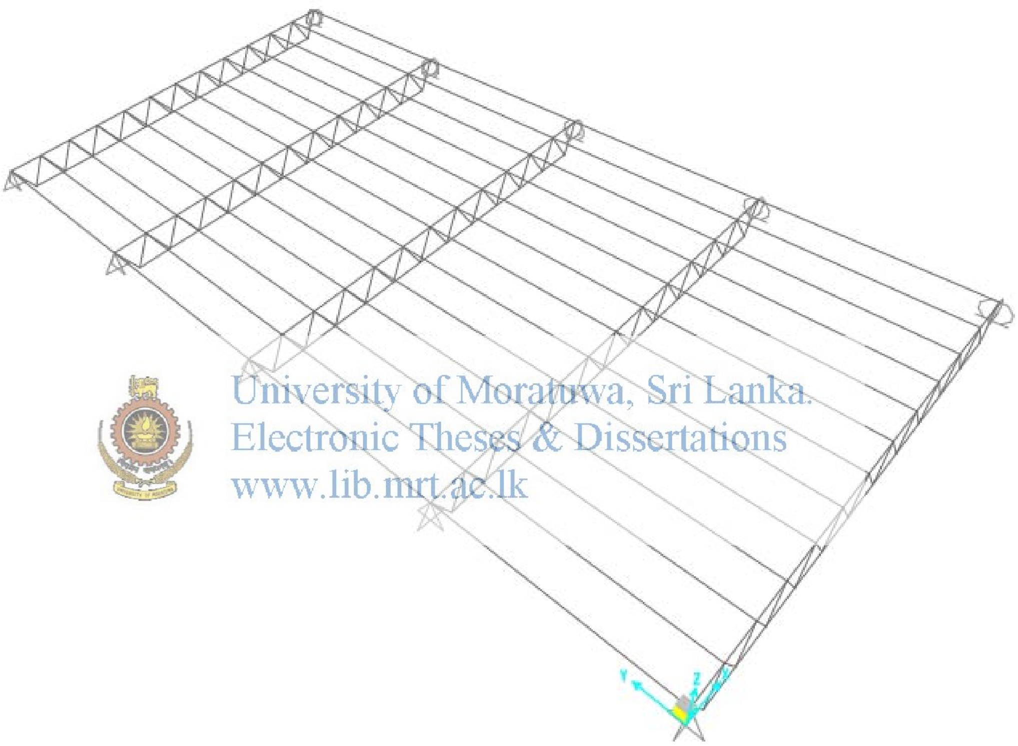
Design Sections for CFS & HRS Roof Trusses		
Roof Truss Member	**HRS Roof Structure**	**HRS Roof Structure**
Top Member	2/75x75x8mm	C 200x75x20mm (Back to Back-Double section)
Bottom Member	2/75x75x8mm	C 200x75x20mm (Back to Back-Double section)
Bracin Member	75x50x6mm	C 200x75x20mm



a) Plan View



b) Sectional View



c) 3D View of Roof Structure.

Figure 4-5: 12.0m Span Truss - a) Plan View - b) Sectional View -

c) 3D View of Roof Structure

The Table 4-12 show the variation of member forces at bottom chord for two families of steel members.

Table 4-12 : 12.0 m Span Truss; Element Forces-Bottom chord

Element Forces - Frames -bottom chord (COMBO-ENV)				
	Maximum Force (Tension)			
	Element Label	Cold formed	Hot Rolled	Ratio
		KN	KN	C/H
Outer most truss	25	23.3	25.2	0.92
	26	42.3	45.8	0.92
	27	57.2	61.8	0.93
	28	67.9	73.3	0.93
	29	74.3	80.2	0.93
	30	76.5	82.5	0.93
	31	74.3	80.2	0.93
	32	67.9	73.3	0.93
	33	57.3	61.9	0.93
	34	42.4	45.8	0.93
	35	23.4	25.3	0.92
1st interior truss	635	53.7	57.1	0.94
	636	97.8	103.9	0.94
	637	132.2	140.3	0.94
	638	156.9	166.3	0.94
	639	171.6	181.9	0.94
	640	176.6	187.1	0.94
	641	171.7	181.9	0.94
	642	156.9	166.3	0.94
	643	132.3	140.3	0.94
	644	97.9	103.9	0.94
	645	53.9	57.2	0.94
Interior truss	682	44.6	47.5	0.94
	683	81.2	86.4	0.94
	684	109.9	116.8	0.94
	685	130.4	138.4	0.94
	686	142.7	151.4	0.94
	687	146.9	155.8	0.94

Each truss having eleven members at bottom chord and 5-trusses have been employed over the four bays. The variation of bottom member forces on these five trusses is shown in Table 4-12 above.

Assessing on the analysis result is shown that the load pattern distribution on each truss is much regular and having elliptical variation. But this tension forces distribution is much differ from 8.0m span truss. This concludes that there is significant deference in load pattern distribution for Pratt truss and Parallel girder truss. The other significant observation, there is no compression forces occurred at bottom members of parallel girder truss despite of the truss location. For each truss, the middle members are the highly stress members than the members close to support, which was totally different observation from 8.0m, and 10.0m truss systems. When compare the absolute difference of the tension forces between CFS and HRS system it still show in same order with 10.0m span roof structure. CFS members always shows lower force intensity than HRS members, concluded that CFS members always provide "less-stress" bottom members than HRS. This is similar advantage discussed under 10.0m span roof trusses but in less order.

The Table 4-13 show the variation of member forces at top chord of two families.

The force distribution pattern in top member of 12.0m truss system is much likely to mirror image of bottom member force distribution. This concludes that both top and bottom members are expose to same level of force intensity. This pattern distribution of top members forces shows, the middle members are highly stressed than the members close to support, which was totally different observation from 8.0m, and 10.0m truss systems again. Mainly it concluded that the member forces distribution is totally opposite for parallel girder trusses and pitched trusses.

But still CFS members shows lower force intensity than HRS members.

Table 4-13 : 12.0 m Span Truss; Element Forces-Top chord

Element Forces - Frames -Top chord (COMBO-ENV)				
	Minimum Force(Compression)			
	Element Label	Cold formed	Hot Rolled	Ratio
		KN	KN	C/H
Outer most truss	1	-9.4	-9.6	0.98
	2	-30.2	-32.8	0.92
	3	-50.6	-54.7	0.93
	4	-63	-68	0.93
	5	-71.1	-76.8	0.93
	6	-74.9	-80.9	0.93
	7	-74.5	-80.5	0.93
	8	-69.8	-75.4	0.93
	9	-60.9	-65.8	0.93
	10	-47.7	-51.6	0.92
	11	-30.2	-32.8	0.92
	12	-9.4	-9.6	0.98
1st interior truss	13	-21.5	-21.7	0.99
	14	-69.9	-74.7	0.94
	15	-119.2	-126.4	0.94
	16	-147.5	-156.4	0.94
	17	-166	-175.9	0.94
	18	-174.6	-185	0.94
	19	-173.4	-183.7	0.94
	20	-162.4	-172.1	0.94
	21	-141.5	-150	0.94
	22	-110.7	-117.6	0.94
	23	-69.9	-74.7	0.94
	24	-21.5	-21.7	0.99
Interior truss	25	-35	-35.7	0.98
	26	-80.8	-86.1	0.94
	27	-99.5	-105.7	0.94
	28	-123.1	-130.6	0.94
	29	-138.4	-146.8	0.94
	30	-145.6	-154.4	0.94

The member forces of the roof trusses, are showing a difference depending whether its purlins are continuous or discontinuous over the trusses. If the spacing of trusses is much smaller, it could be possible to provide a continuous purlins over few trusses. It gives the most critical member forces at 1st interior truss, but simultaneously reduce the interior truss member forces. In other case, the member forces for all 1st interior and interior truss members are identical, but lesser than the value of previously discussed 1st interior truss. This is accurately demonstrate in above Table 4-12 & Table 4-13, showing the less forces for interior truss and high member forces for 1st interior truss. But most practical situation is purlins are discontinuous over each trusses, which gives less critical member forces, and hence this study was compared the member forces for continuous pulins over the truss.

The deflection was considered again under main two conditions, which pulins are continuous over the trusses & simply supported at each truss location. Those conditions were adopted to ensure the roof structure to be at its serviceability limit, under any conditions, which could be used at construction stage.

Table 4-14 & Figure 4-6 below shows the vertical deflection of each truss at its mid span, for both steel families under both conditions describe above.

Although the CFS roof system show a larger deflection than HRS roof system the maximum deflection value in CFS structure is 15.1mm which is well within the allowable limit (Table 4-8) of 66.7mm.

$$y_{max} = \Delta_{max} \leq \Delta_{allowable} = \frac{l}{value}$$

Comparing the both 8.0m & 10.0m truss deflection, it shows that the rate of increase of deflection is higher in 12.0m span CFS roof system.

Table 4-14 : 12.0 m Span Truss - Maximum Vertical Deflection at Mid Span;

TABLE: Vertical Deflection at the Mid Span of truss				
Truss Location	**Joint**	**Output Case**	**CFS**	**HRS**
	Text	Text	**U3(mm)**	**U3(mm)**
Outer Most Truss	13	SDL (purlin discontinuous)	-7.097	-5.492
	13	SDL (purlin continuous)	-5.784	-4.519
1st Interior Truss	261	SDL (purlin discontinuous)	-13.389	-10.137
	261	SDL (purlin continuous)	-15.106	-11.427
Interior Truss	286	SDL (purlin discontinuous)	-13.389	-10.137
	286	SDL (purlin continuous)	-12.579	-9.530
1st Interior Truss	311	SDL (purlin discontinuous)	-13.389	-10.137
	311	SDL (purlin continuous)	-15.106	-11.427
Outer Most Truss	336	SDL (purlin discontinuous)	-7.097	-5.492
	336	SDL (purlin continuous)	-5.784	-4.519

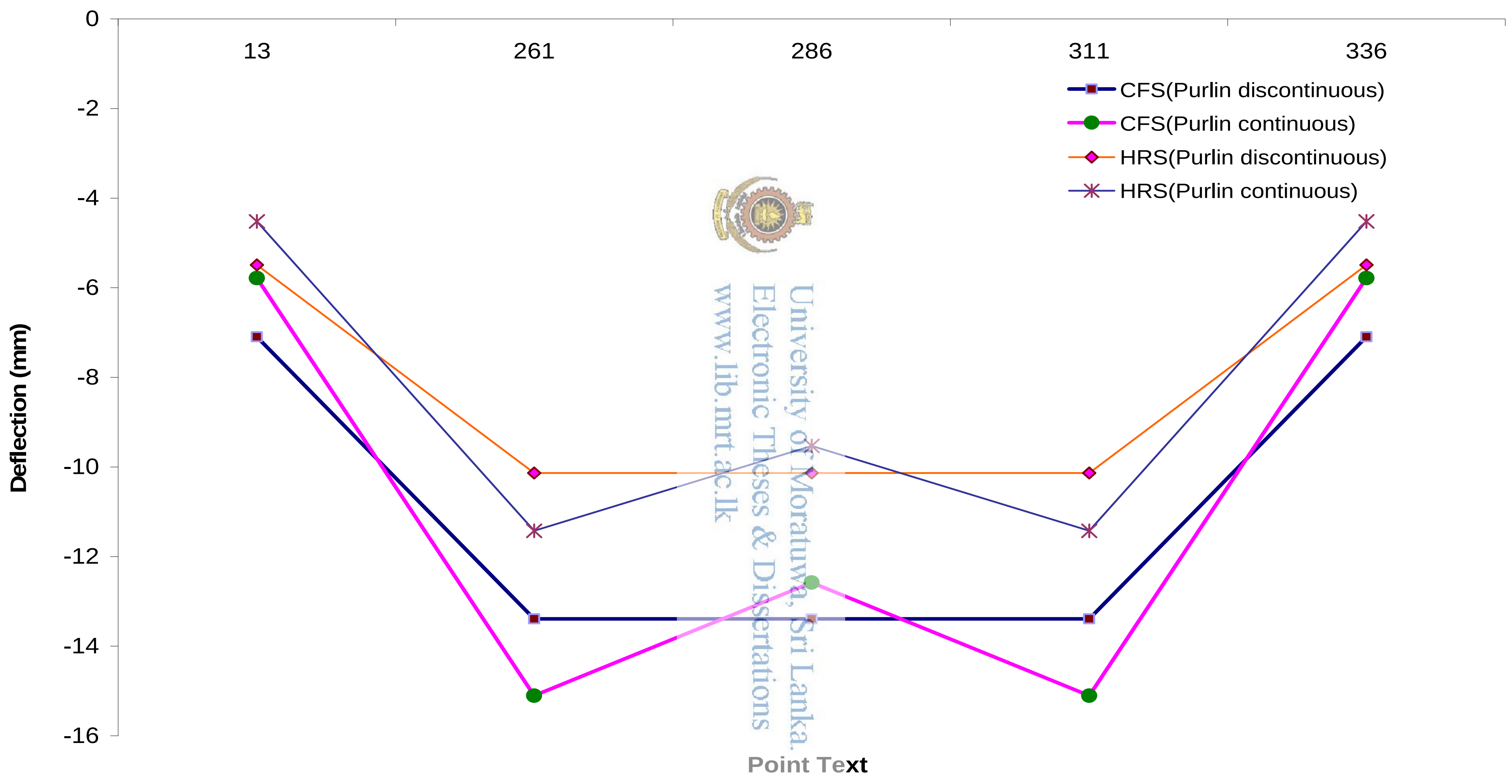


Figure 4-6: Variation of Deflection at Bottom Mid Span of Truss

4.2. STRUCTURAL DESIGN TO BS5950-I AND BS5950-V

The Hot rolled steel trusses were design according to the British Standards BS5950-I while the Cold form trusses were design to BS5950-V. The main deference in design criteria for both CFS and HRS is, CFS design is always govern by the local buckling requirement even for the non-flexural members, while HRS design criteria govern by direct stress requirement for non-flexural members. Therefore the buckling failure of the CFS members are much critical than direct stress failure. The demonstration design calculation for both CFS and HRS members are given in ANNEX-C.

5.0 RESULTS

5.1. STEEL QUANTITY

All individual trusses were compared to its steel quantity against the span of truss. Table 5-1 and figure 5-1 shows the variation of weight Vs Span.

Table 5-1 : Weight of Steel

STEEL QUANTITY COMPARISON			
Span of Roof Structure	Hot Rolled	Cold form	%
	Ton	Ton	Difference
4.0m	0.069	0.03	56.5
8.0m	0.336	0.118	64.8
10.0m	0.45	0.178	60.4
12.0m	0.56	0.45	19.6

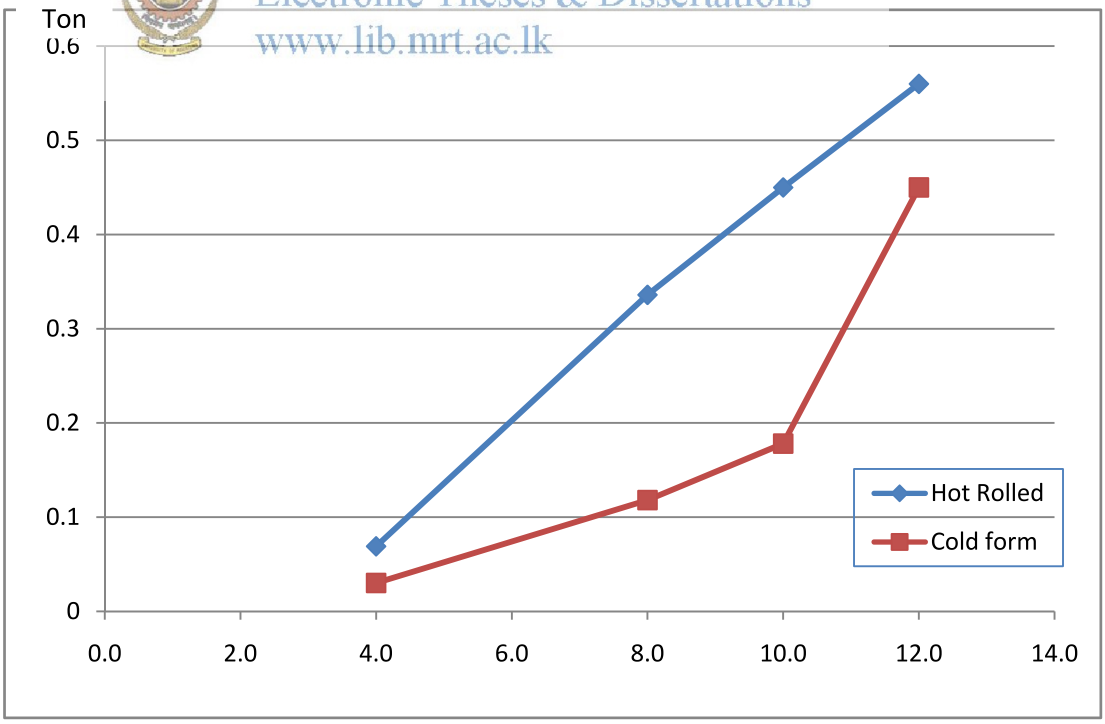


Figure 5-1: Steel Quantity Comparison for Individual Truss

5.2. TOTAL COST OF ROOF STRUCTURE

The total cost for complete roof structures were compared under this section. This included the cost of galvanizing for the HRS section as a precaution for corrosion. Since CFS itself comes as galvanized members, there is no need of additional percussion on this regard. Cost of the purlins and the roofing sheet were also included. Evaluation of the cost of structures are shown in *Annex - F*. Comparison of the total cost of roof structures for two steel families shown in *Table – 5-2*, and *Figure – 5-2*

Table 5-2 : Total Cost of Structure

TOTAL COST (Rs)			
Truss Span	**Cold Form**	**Hot Rolled**	**Net Saving**
4m	482,750.00	559,250.00	76,500.00
8m	1,180,500.00	1,574,250.00	393,750.00
10m	1,294,750.00	1,700,150.00	405,400.00
12m	2,939,250.00	3,160,500.00	221,250.00

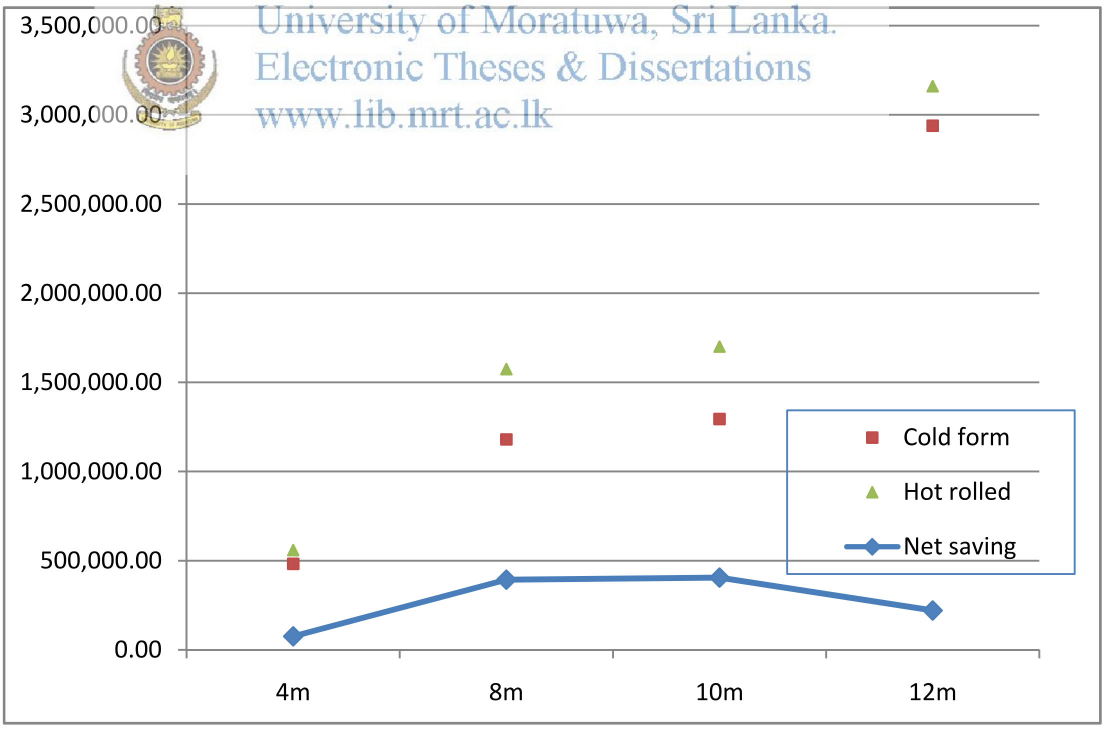


Figure 5-2: Total Cost of Roof Structure - Comparison

5.3. UNIT COST OF ROOF STRUCTURE

Extensive study of cost evaluation for 1m^2 of roof structure has been carried out under this section and summarized results are tabulated in *Table 5-3*. Graphical interpretation of cost variation for unite area of roof structure is shown in *Figure 5-3*.

Table 5-3 : Unit Cost of Roof Structure

UNIT COST OF ROOF (Rs)				
Truss Span	**Roof cover area m^2**	**Cold Form**	**Hot Rolled**	**Net Saving**
4m	48	10,057.29	11,651.04	1,593.75
8m	128	9,222.66	12,298.83	3,076.17
10m	128	10,115.23	13,282.42	3,167.19
12m	288	10,205.73	10,973.96	768.23

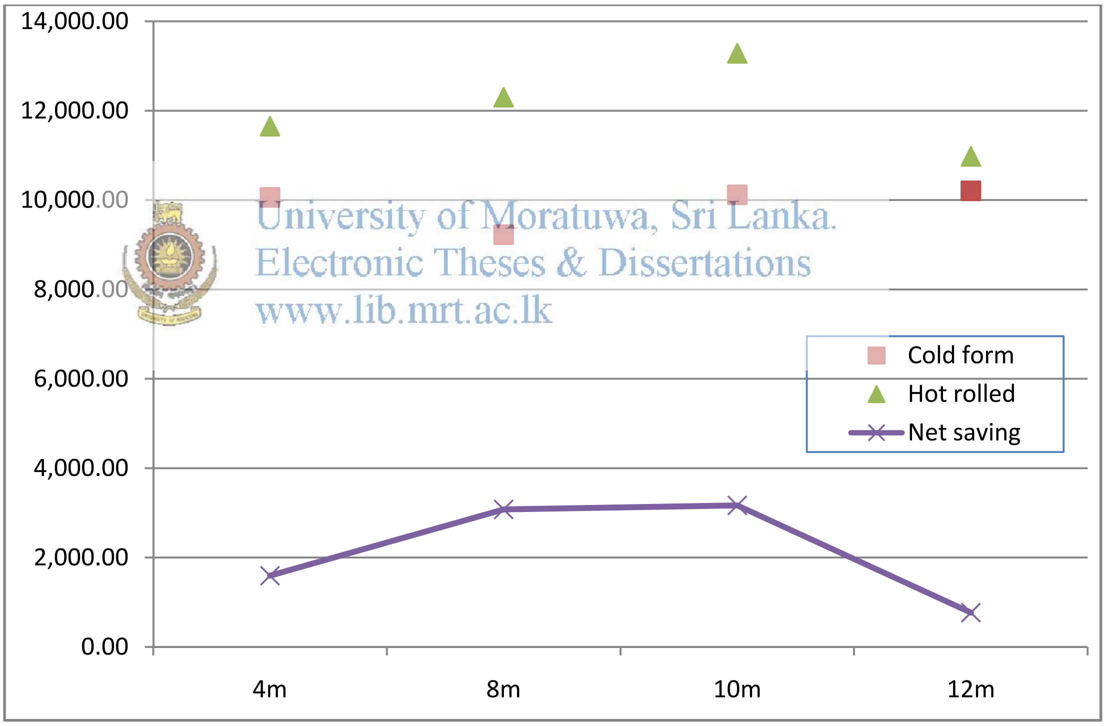


Figure 5-3: Unit Cost of Roof Structure - Comparison

6.0 CONCLUSION AND RECOMMENDATION

6.1 CONCLUSION

The extensive study on comparison of steel quantity required for medium span roof structure, between two steel families, under same imposed and super imposed loading, shows CFS required lesser steel quantity than HRS. These findings will directly effects to the construction cost of roof structure.

When compared the cost of roof structure, under the terms of total cost and unite cost, it was obvious that a great saving for medium span roof construction can be achieved. According to the results obtained, maximum cost saving has been occurred between the rang of 8.0m to 10.0m span. The percentage saving within theses identified range varies between, 24% to 25 %, which generates the significant reduction on the medium scale construction cost.

By considering the availability of the CFS section in Sri Lanka, Minimum section was considered as 100x1.2-C, for this study. If further smaller sections are available for construction, the optimum cost saving range could be expanded toward the lower spans range and future studies are required to be carryout to identify the exact benefits.

According to the current market price of material in Sri Lanka, RS 3000/- to RS 3200/- per unit area of ($1.0m^2$) can be saved for a medium span roof structure.

6.2 RECOMMENDATION

According to the finding of this study, cold form steel members for medium span roof trusses up to 12.0 m span, can be recomended with a total cost saving of 25% for whole roof structure. The recommended optimum beneficiary span range is lying, between 8.0m to 10.0m span.

Therefore cold form steel will provide a sound solution as a alternative construction material for rapidly increasing construction cost in Sri Lanka construction industry.

6.2.1. Recommendation for future work.

This study was limited to actual truss forms and bay spacing. Check the optimum cost of roof structure by varying the truss forms and its spacing, and for its combinations would give the most finest result in future study.

REFERENCES

- AISI(1996) Cold Form Steel Design Manual, American Iron and Steel Institute, Washington D.C.
- BS 5950-8 (1990) Structural use of steel work in building, Code of practice for fire resistant design.
- BS 5950-5 (1998) Structural use of steel work in building, Code of practice for design of cold formed thin gauge section.
- Chen, J. and Young, B. (2004), Mechanical Properties of Cold-Formed Steel at Elevated Temperatures, Seventh International Specialty Conference on Cold-Formed Steel Structures, Oriando, Florida, USA.
- Chen, J. and Young, B. (2006), Corner Properties of Cold-formed Steel Sections at Elevated Temperatures, Thin-Walled Structures, Vol. 44, Issue 2.
- Chen W., Ye, J., Bai, Y. and Zhao, X.L. (2013), Improved fire resistant performance of load bearing cold formed steel interior and exterior wall systems, Thin-Walled Structures, Vol.73, Pages 145-157.

- Culver, C. G. (1972) ''Steel Column Buckling under Thermal Gradients,'' Journal of the Structural Division, ASCE Proceedings, vol. 98.
- Eurocode 3 (2005) EN 1993-1-2, Design of steel structures, General rules, Structural fire design.
- Feng, M., Wang, Y.C., Davies J.M. (2003), Thermal Performance of cold-formed thin-walled steel panel systems in fire, Fire safety journal, Vol 38, Issue 4, pages 365-394.
- Hassinen, P., Helenius, A., Hieta, J., and Westerlund, A. (1988) "Structural Sandwich Panels at Low Temperature," Prereport of the 13th Congress, IABSE.
- Klippstein, K.H. (1978) "Strength of Cold form Steel Studs Exposed to Fire," Proceeding of the 4th International Specialty Conference on the Cold-Formed Steel Structures, University of Missouri-Rolla.

- Klippstein, K.H. (1980), "Behaviour of Cold-Formed Steel Studs in Fire Test," Proceeding of the 5th International Specialty conference on Cold-Formed steel Structures, University of Missouri-Rolla.

- Kwon, Y. and Hancock, G. (1992), Test of Cold Formed Channels with Local and Distortional Buckling, Journal of Structural Engineering, Vol. 118.

- Moen, C.D. and Schafer, B.W., Direct Strength Design of Cold Form Steel Members With Perforations, The Johns Hopkins University, Department of Civil Engineering, Baltimore, USA, March (2009).

- Outinen, T.A and Myllymaki, J. (1995), "The Local Buckling of RHS member at Elevated Temperatures", VTT research notes 1672, Technical Research Center of Finland, ESPOO

- Ranby, A. (1999), "Structural Fire Design of Thin walled Steel Sections", Report 188:5, Swedish Institute of Steel Construction and Division of Steel Construction, Lulea University of Technology.

- Schafer, B.W. and Pekoz, T. (1998), "Computational Modelling of cold-formed steel: Characterising Geometric Imperfection and Residual Stresses", Journal of Constructional Steel Research, Vol47.

- Uddin, T. and Culver, C.G. (1975), "Effects of Elevated Temperature on Structural Members." Journal of the Structural Division, ASCE Proceeding, Vol 101.

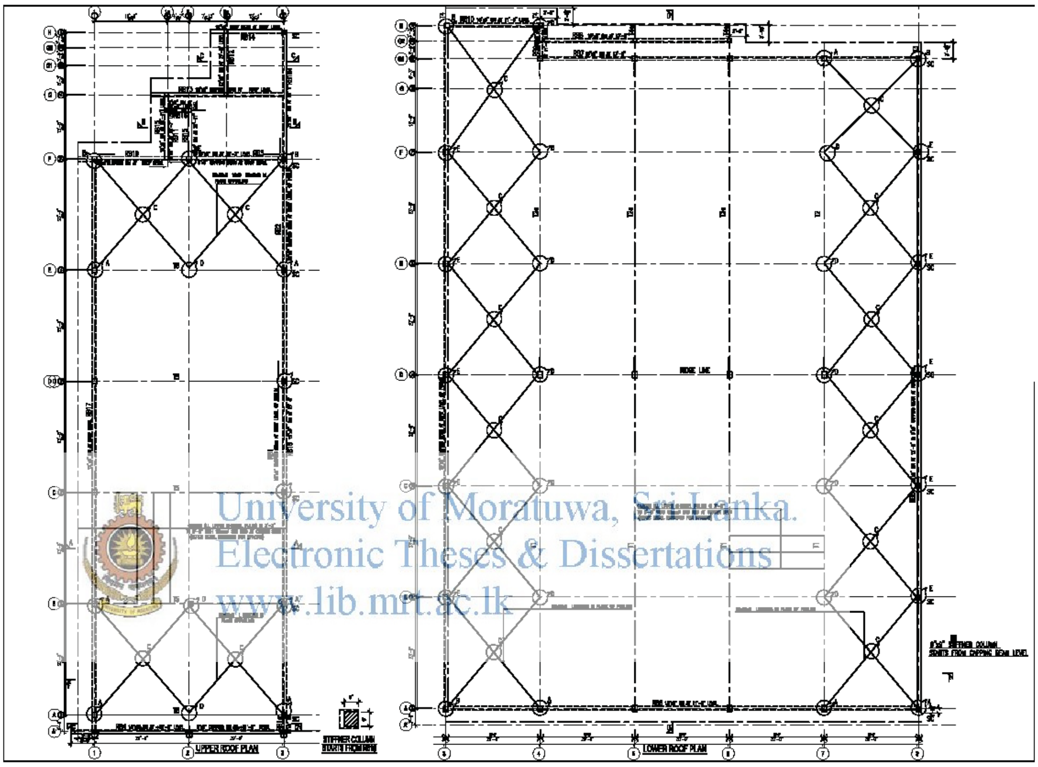
- Walker, A.C. (1975), "Design and Analysis of Cold-formed section", Halsted Press, New York

- Worked Examples to BS5950 Part 5 (1993) The Steel Construction Institute – 1993 edition.

- Young, B. (2005), Experimental Investigation of Cold-formed Steel Lipped Channel Concentrically Loaded Compression Members, Journal of Structural Engineering, Vol.131, Issue 9

ANNEX : A - GENERAL LAYOUT & FEATURES OF SELECTED CASES

1) CARGILLS BIG CITY AT MAHARAGAMA



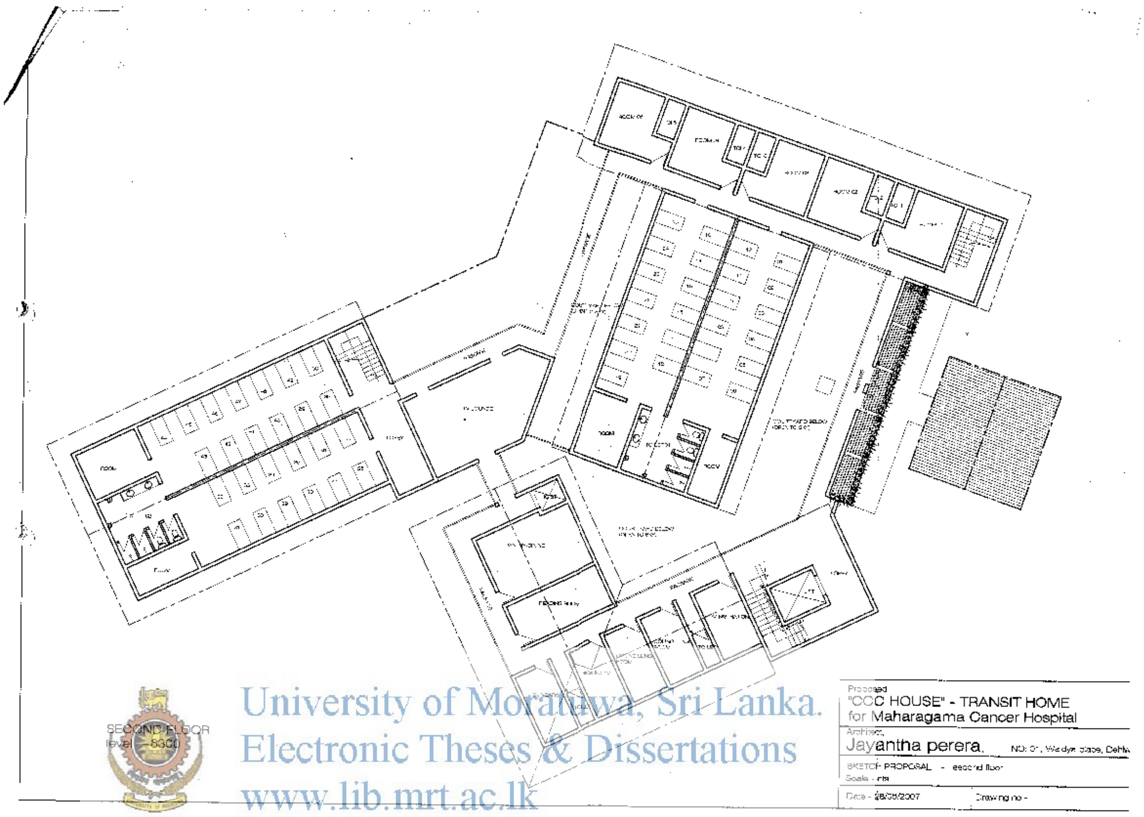
PLAN VIEW OF ROOF STRUCTURE.

DETAIL OF BUILDING AND ROOF GEOMETRY.

- TRUSS SPANS AVAILABLE - 7m,12m, 15m,
- TRUSS SPANS SELECTED FOR THE STUDY - 12.0m
- ROOF SLOPE - 8^{O} Degree
- MINIMUM EAVE HEIGHT - 3.6m
- BAY SPACING - 6.0m

- SUPPORTING ELEMENT TYPE - RC Columns & Beams
- WIND EXPOSURE CATEGORY - Normal structure (wind zone 3)
- HORIZONTAL CEILING AT EAVE HEIGHT - Yes
- SUPER IMPOSED DEAD LOAD -Thermal insulating and service lines
- ROOF COVERING -light weight Al roofing sheet
- PROPOSED TRUSS FORMS
  - FOR HOT ROLLED STEEL - parallel girder truss
  - FOR COLD FORMED STEEL - parallel girder truss

2) PROPOSED CANCER HOSPITAL BUILDING



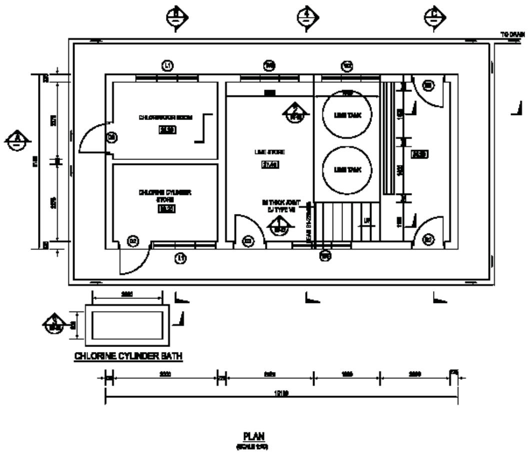
PLAN VIEW OF STRUCTURE.

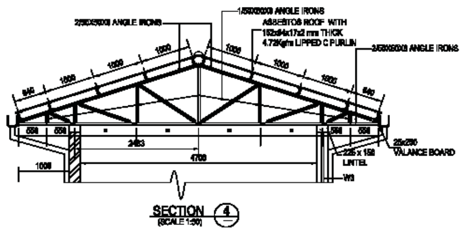
DETAIL OF BUILDING AND ROOF GEOMETRY:

- TRUSS SPANS AVAILABLE - 5m, 6.5m,10m
- TRUSS SPANS SELECTED FOR STUDY - 10.0m
- ROOF SLOPE - 28^{O},
- MINIMUM EAVE HEIGHT - From top most storey – 3.1m
- BAY SPACING - 3.2m
- SUPPORTING ELEMENT TYPE - RC Columns & Beams, Steel columns
- WIND EXPOSURE CATEGORY - Post disaster structure (wind zone 03)

- HORIZONTAL CEILING AT EAVE HEIGHT - yes
- SUPER IMPOSED DEAD LOAD - Ceiling
- ROOF COVERING - Colour cone tile / Coloured Asbestos
- PROPOSED TRUSS FORMS
    - FOR HOT ROLLED STEEL - Triangular or Parallel girder Truss
    - FOR COLD FORMED STEEL - Triangular Truss

3) PUMP HOUSE AT AMPARA





REQUIRMENT BY CLIENT / ARCHITECT

- SPANS - 5m
- ROOF PITCH - 18°
- MINIMUM EAVE HEIGHT - 3.45m
- SUPPORTING ELEMENT TYPE - RC Columns
- WIND EXPOSURE CATEGORY - Normal structure (Wind zone 1)
- HORIZONTAL CEILING AT EAVE HEIGHT - Not specified
- ROOF COVERING - Asbestos sheet
- PROPOSED TRUSS FORMS
    - FOR HOT ROLLED STEEL - Triangular or Parallel girder truss
    - FOR COLD FORMED STEEL - Triangular Truss

ANNEX : B - LOAD CALCULATION

B.1 WIND LOAD

WIND LOAD CALCULATION TO CP 3: Chapter V

Pump House @ Ampara (wind Zone : 01-normal structure)

$Vs = S_1 x S_2 x S_3 \text{ x } V$

S_1	- 1.00	Level terrain
S_2	- 0.74	Open country with scatter wind break
S_3	- 1.00	H-5.0m

$V = 49 \text{ ms}^{-1}$

$V_S = 36.26$

$.q = k \, V_s^2$ k = 0.613

$= 805.9 \text{ N/m}^2$

Table 13

h/w – 3.6 / 5.15 = 0.699

Roof angle - 18^0

Φ=0.2

Overall Coefficient

Maximum +ve pressure coefficient : $C_p = +0.6$

Minimum -ve pressure coefficient : $C_p = -0.98$

$P = C_P \cdot q$

$P = 483.54\ N/m^2$ (Maximum +ve pressure) downward

$P = -789.78\ N/m^2$ (Maximum -ve pressure) upward

For 3.0m Bay spacing - 0.48354 x 3 = +1.45 kN/m

- 0.78978 x 3 = -2.37 kN/m

WIND LOAD CALCULATION TO CP 3: Chapter V

<u>CCC House @ Maharagama (Wind zone – 03 Post Disaster Structre)</u>

$V_s = S_1 \times S_2 \times S_3 \times V$

S_1 - 1.00 Level terrain

S_2 - 0.88 Open country with scatter wind break

(Building Hight = 9.0m, L=30.58m W=11.5m)

S_3 - 1.00

$V = 38\ ms^{-1}$

$V_S =$ 33.44

$.q = k\ V_s^2$ k – 0.613

$= 685.5\ N/m^2$

Table 13

h/w – 9 / 11.5 = 0.78

Roof angle - 28^0

$\Phi=0.2$

Overall Coefficient

Maximum +ve pressure coefficient : $C_p = +0.9$

Minimum -ve pressure coefficient : $C_p = -1.06$

$P = C_P . q$

$P = 616.95\ N/m^2$ (Maximum +ve pressure) down ward

$P = -726.63\ N/m^2$ (Maximum -ve pressure) upward

For 4.0m Bay spacing - 0.61695 x 4 = +2.47 kN/m

- 0.72663 x 4 = -2.91 kN/m

For 3.2m Bay spacing - 0.61695 x 3.2 = +1.974 kN/m

- 0.72663 x 3.2 = -2.325 kN/m

WIND LOAD CALCULATION TO CP 3: Chapter V

<u>Cargills BigCity @ Maharagama</u>

$Vs = S_1 x S_2 x S_3 \ x \ V$

S_1 - 1.00 Level terrain

S_2 - 0.74 Open country with scatter wind break

S_3 - 1.00 H-5.0m

$V = 33\ ms^{-1}$

$V_S = 24.42$

$.q = k\ V_s^2$ k – 0.613

$= 365.6\ N/m^2$

Table 13

h/w – 3.3 / 42 = 0.0.078

Roof angle - 8^0

Φ=0.2

Overall Coefficient

Maximum +ve pressure coefficient : $C_p = +0.4$

Minimum -ve pressure coefficient : $C_p = -0.82$

$P = C_P \cdot q$

$P = 146.24\ N/m^2$ (Maximum +ve pressure) down ward

$P = -299.79\ N/m^2$ (Maximum -ve pressure) upward

For 6.0m truss spacing - 0.14624x 6.0 = +0.8774 kN/m

- 0.29979 x 6.0 = -1.799 kN/m

B.2 GRAVITY LOAD

GENARAL LOADINGS FOR ROOF TRUSS ANALYSIS

Live load - $0.25kN/m^2$

(Maximum possible impose load to calculated by analysis)

Purlin loads - NA kN/m^2 (Purlin have been modeled)

Roofing sheet - $0.5\ kN/m^2$

Foils & other SDL - $0.1\ kN/m^2$

LOAD EVALUATION ON ROOF STRUCTURE

<u>Pump House at Ampara - 4m spacing roof structure</u>

Purlin Spacing	-	1.0m
Therefore SDL UDL on perlin	-	(0.5 + 0.1) x 1.0
	-	0.600 kN/m
Live load UDL on perlin	-	0.250 kN/m

<u>CCC House @ Maharagama - 8.0m spacing roof structure</u>

Purlin Spacing	-	1.30m
Therefore SDL UDL on perlin	-	(0.5 + 0.1) x 1.30
	-	0.780 kN/m
Live load UDL on perlin	-	0.325 kN/m

<u>CCC House @ Maharagama - 10.0m spacing roof structure</u>

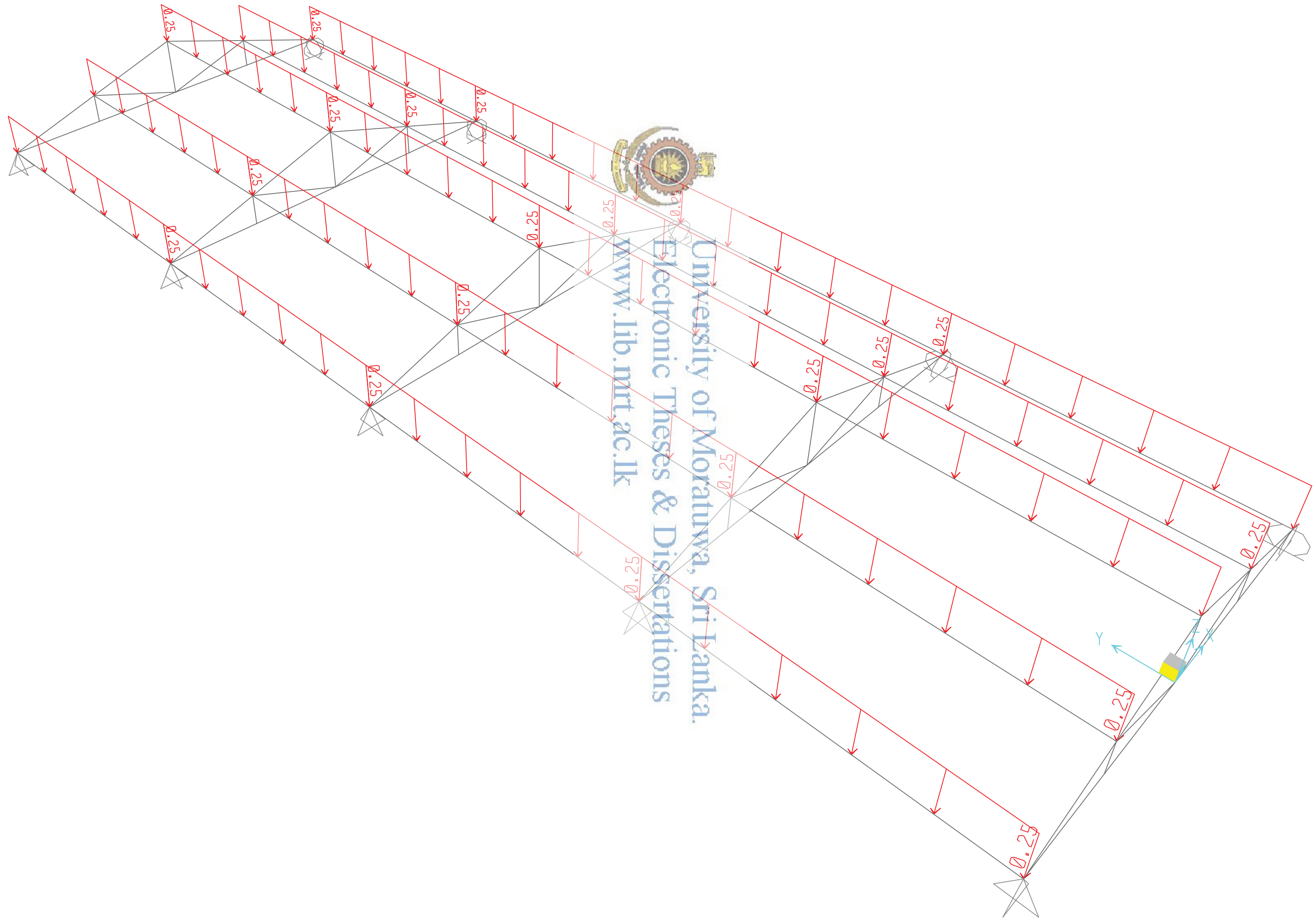
Purlin Spacing	-	1.60m
Therefore SDL UDL on perlin	-	(0.5 + 0.1) x 1.60
	-	0.960 kN/m
Live load UDL on perlin	-	0.400 kN/m

Cargill's Big City @ Maharagama - 12.0m spacing roof structure

Purlin Spacing	-	1.0m
Therefore SDL UDL on perlin	-	(0.5 + 0.1) x 1.0
	-	0.6 kN/m
Live load UDL on perlin	-	0.25 kN/m

SAP2000 11-25-13 3:51:55

LIVE LOAD ASSIGNMENT - 4.0m SPAN ROOF STRUCTURE

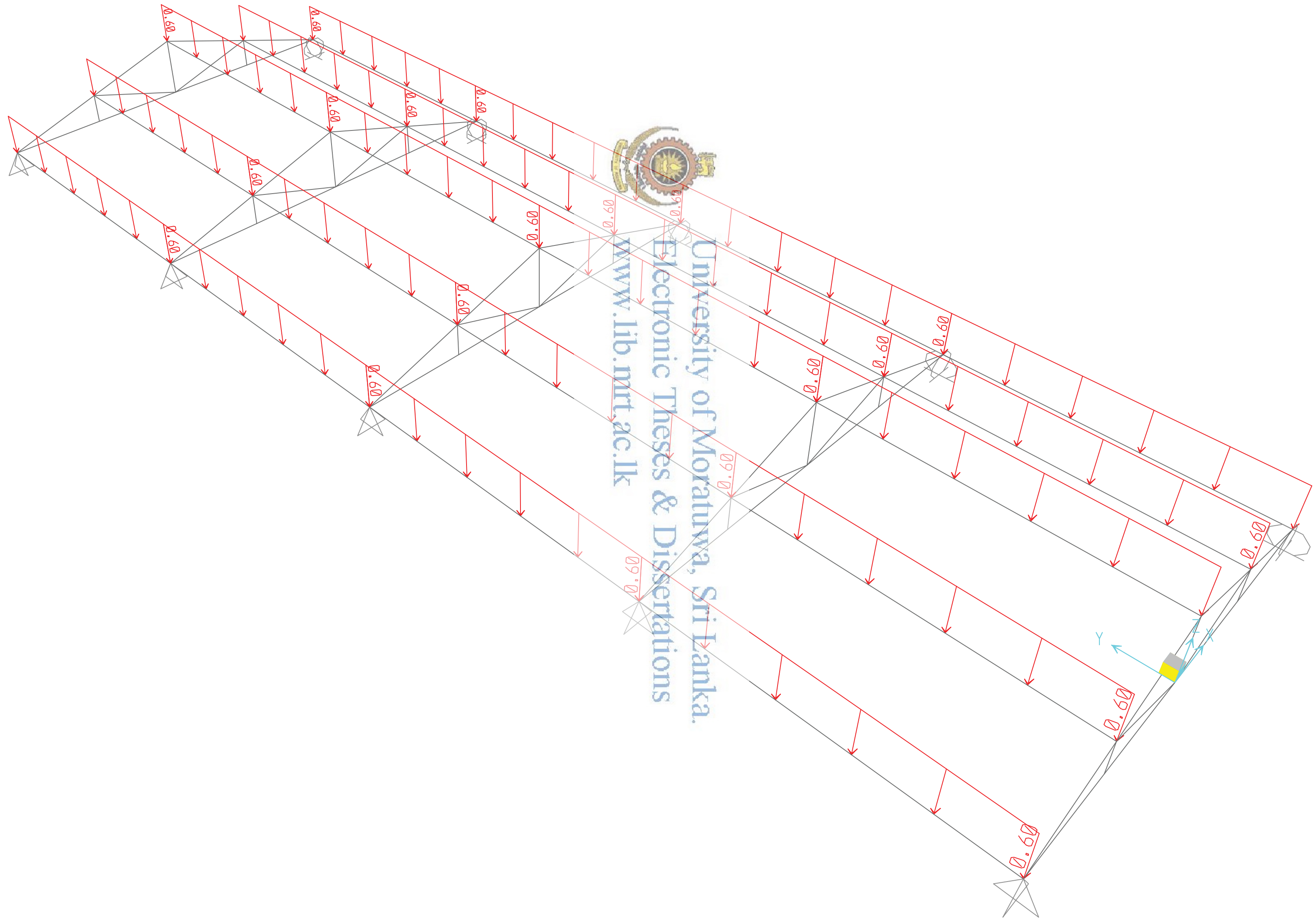


SAP2000 v14.0.0 - File:4m-span devided to - 4(3) - Frame Span Loads (LIVE) (As Defined) - KN, m, C Units

SAP2000

11-25-13 3:49:41

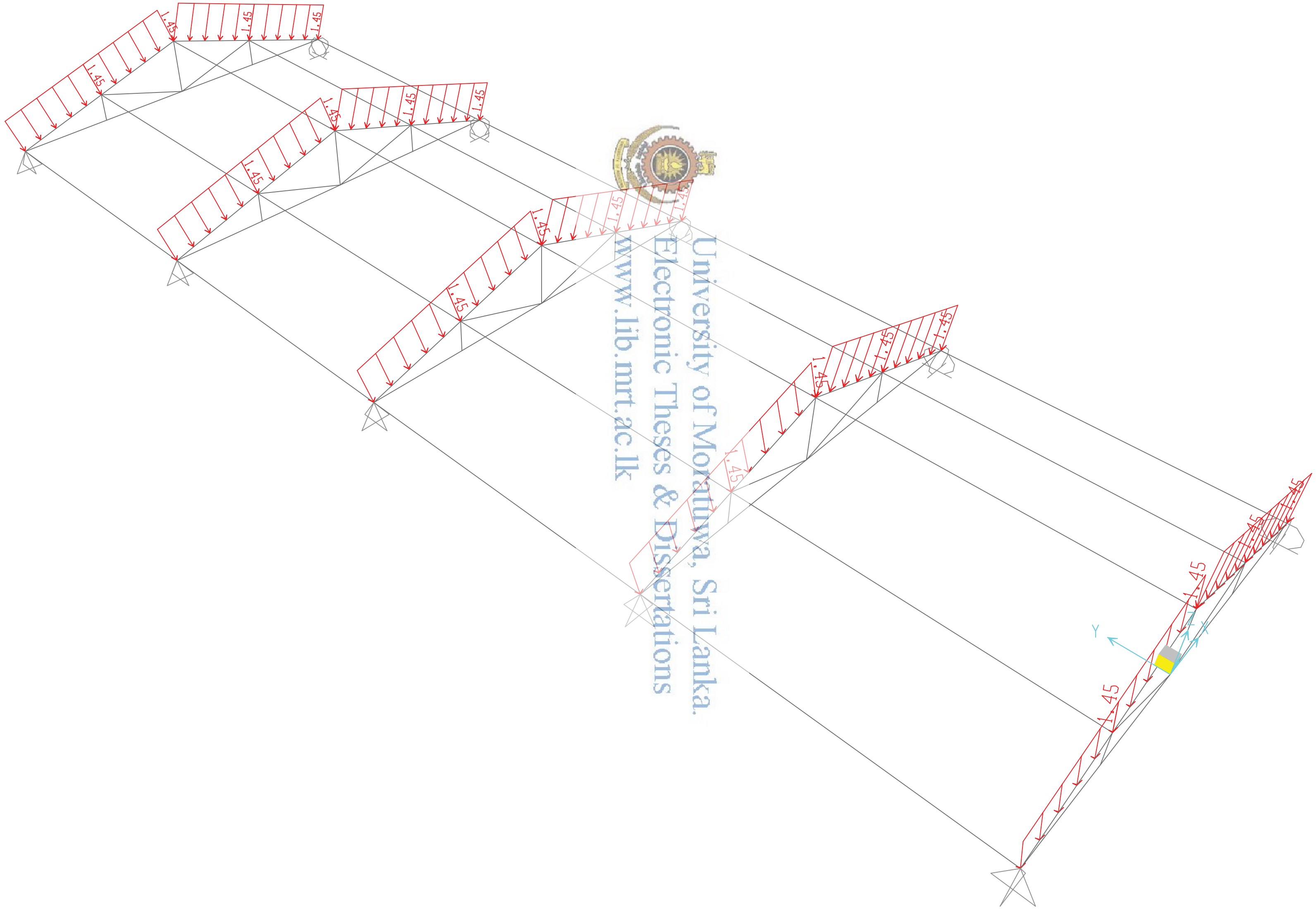
SUPER IMPOSED DEAD LOAD ASSIGNMENT - 4.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:4m-span devided to - 4(3) - Frame Span Loads (SUPER DEAD) (As Defined) - KN, m, C Units

SAP2000 11-25-13 3:53:57

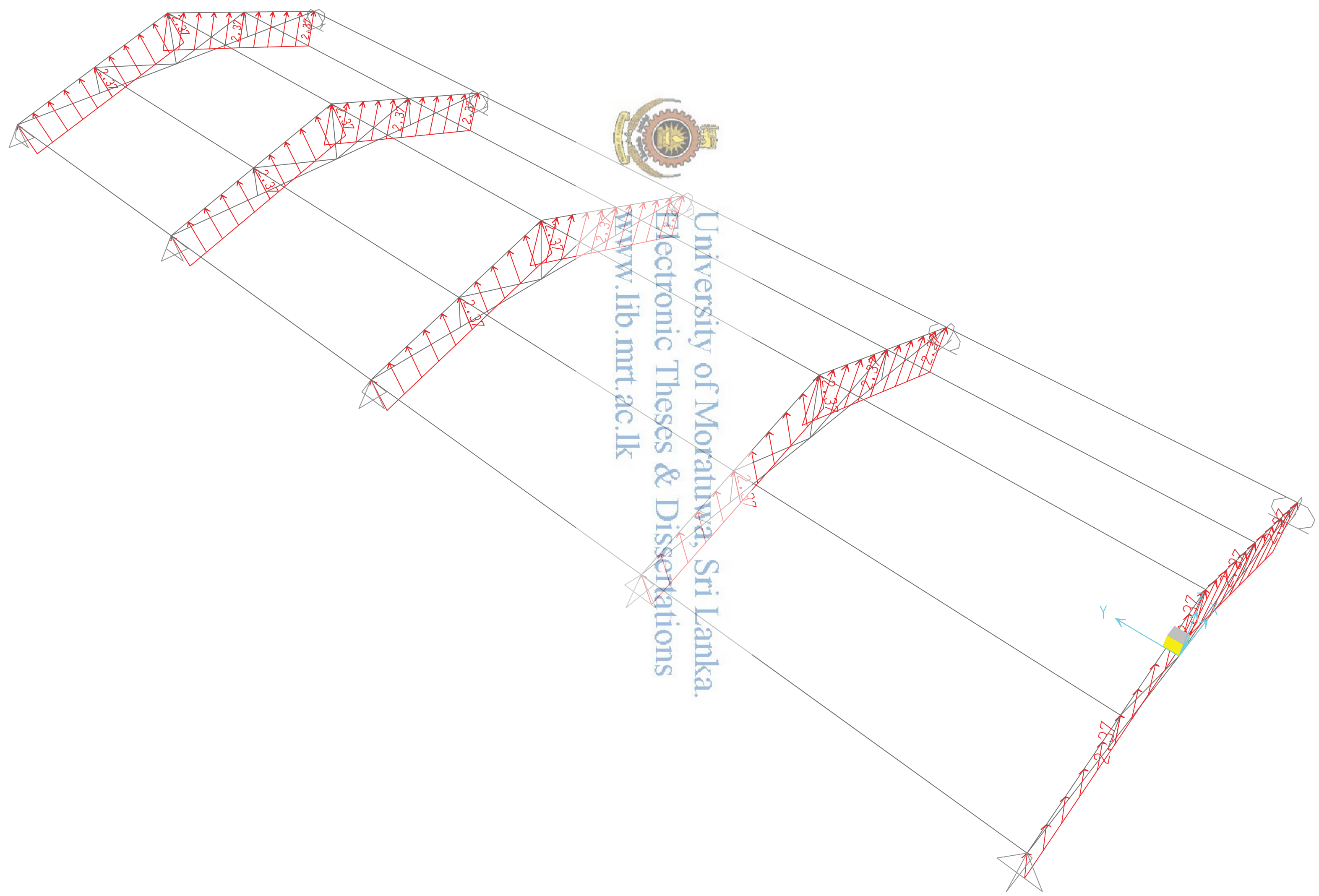
WIND LOAD ASSIGNMENT - 4.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:4m-span devided to - 4(3) - Frame Span Loads (Wind(max)) (As Defined) - KN, m, C Units

SAP2000 11-25-13 3:59:16

WIND LOAD ASSIGNMENT - 4.0m SPAN ROOF STRUCTURE

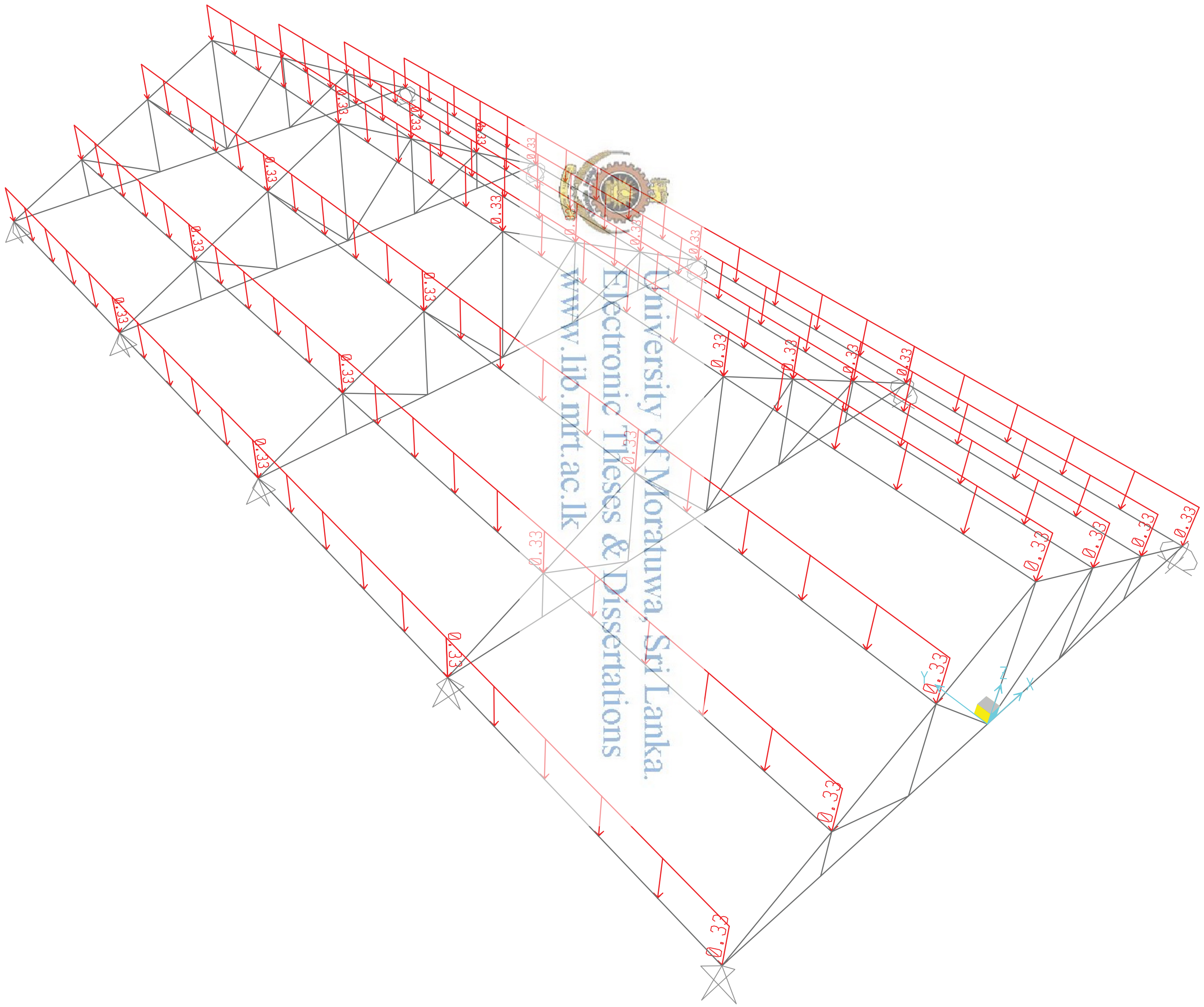


SAP2000 v14.0.0 - File:4m-span devided to - 4(3) - Frame Span Loads (Wind(min)) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:18:35

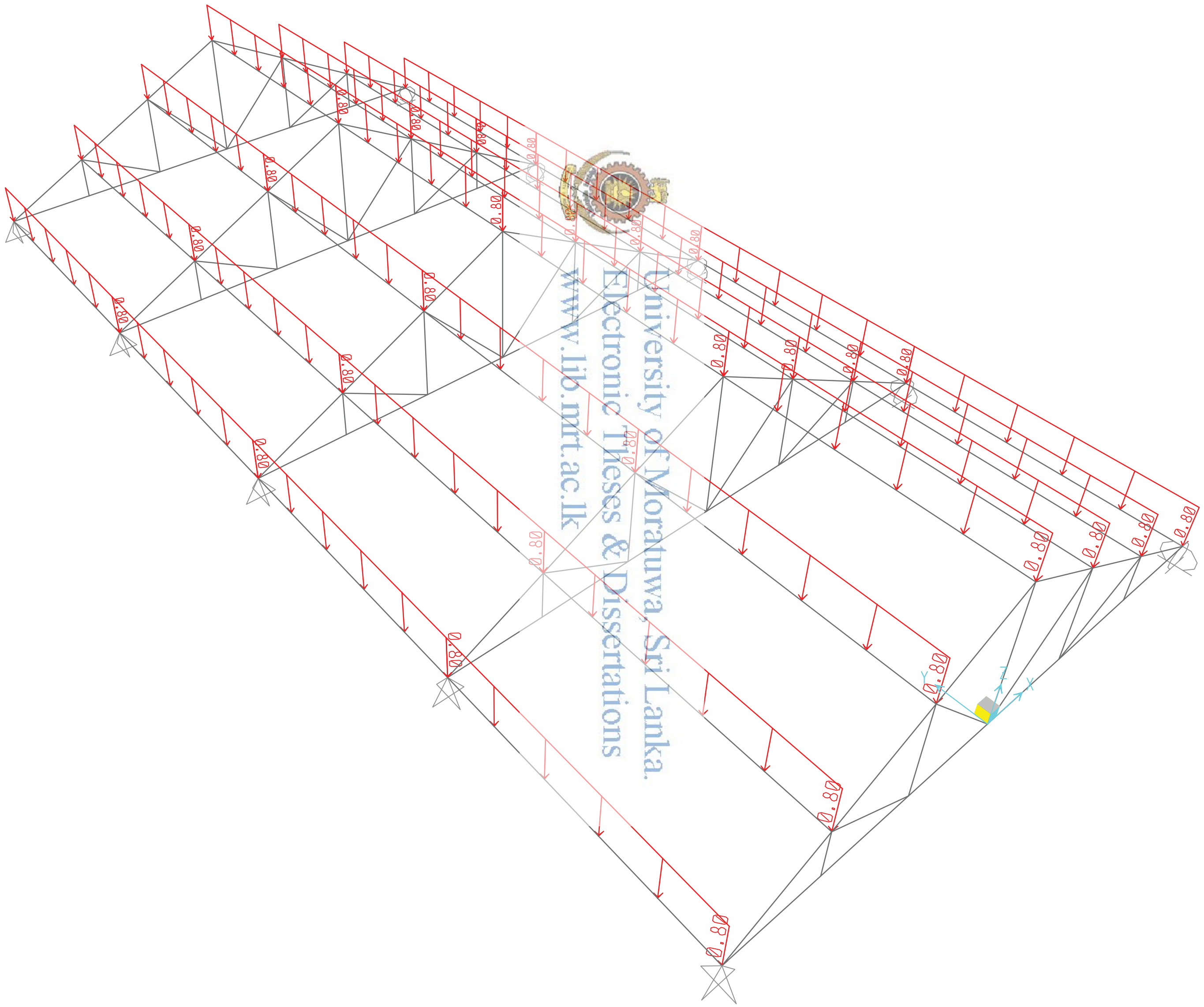
LIVE LOAD ASSIGNMENT - 8.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:8m span- devided in to 6(1) - Frame Span Loads (LIVE) (As Defined) - KN, m, C Units

SAP2000 11-25-13 4:20:15

SUPER IMPOSED DEAD LOAD ASSIGNMENT - 8.0m SPAN ROOF STRUCTURE

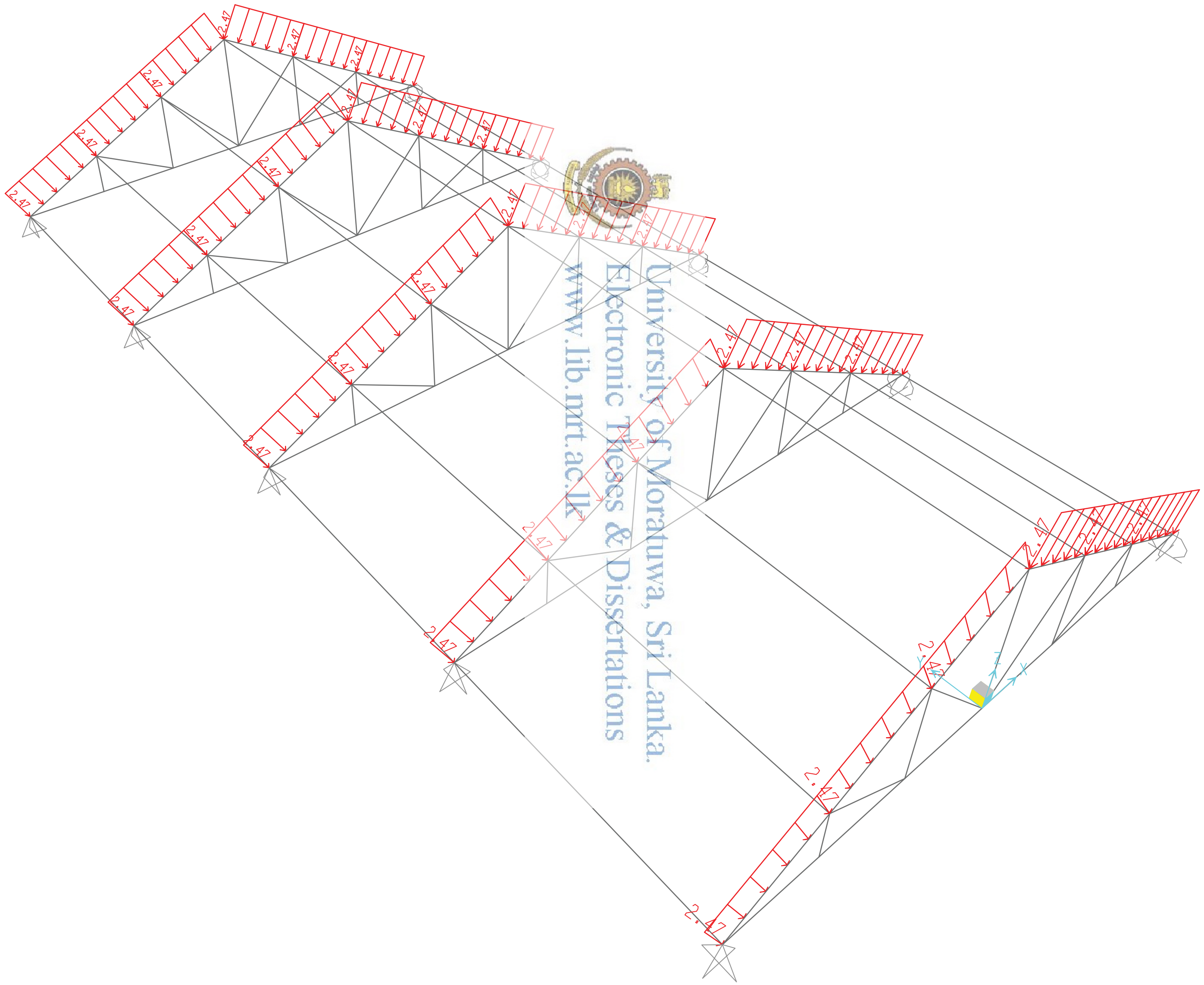


SAP2000 v14.0.0 - File:8m span- devided in to 6(1) - Frame Span Loads (SUPER DEAD) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:21:51

WIND LOAD ASSIGNMENT - 8.0m SPAN ROOF STRUCTURE

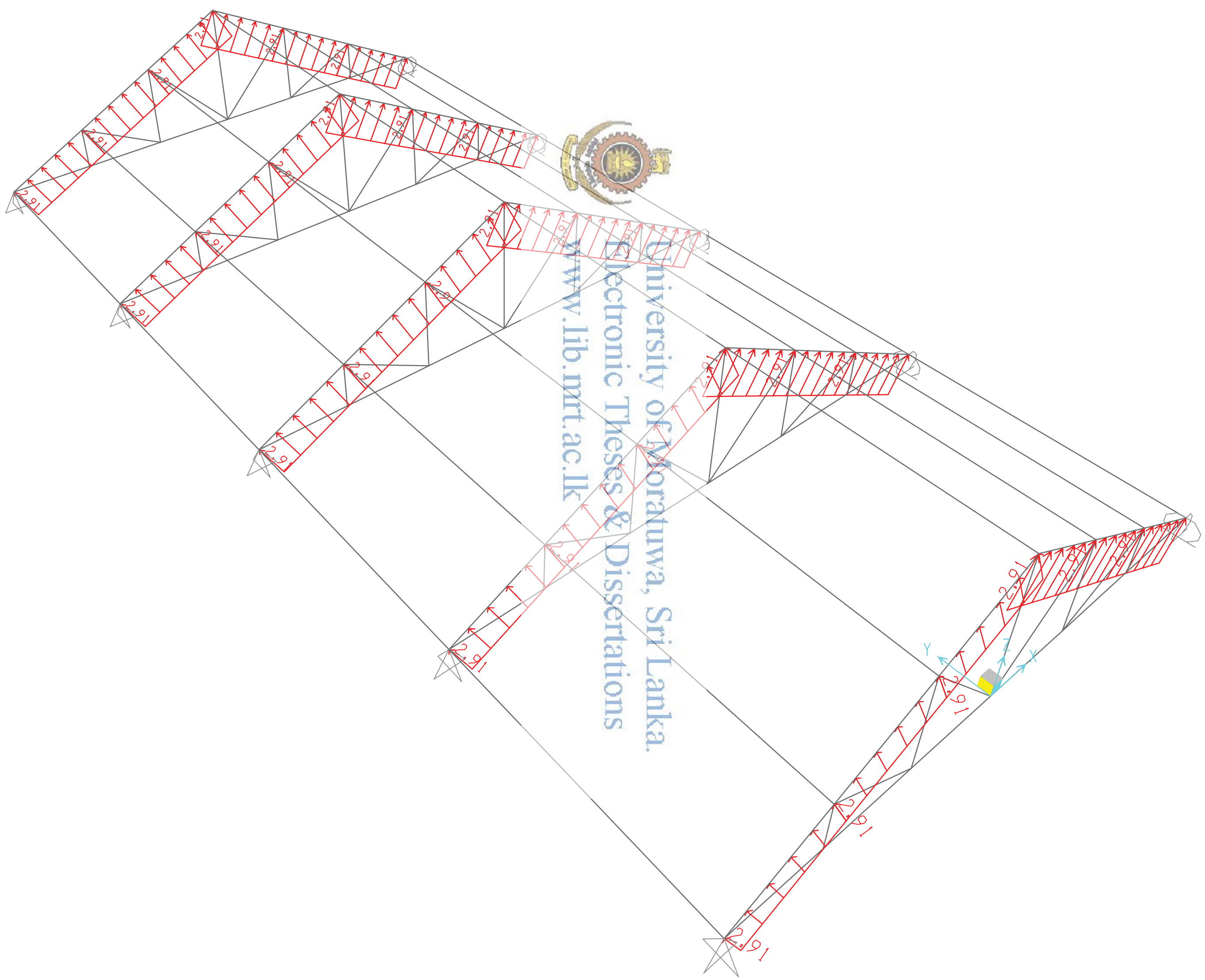


SAP2000 v14.0.0 - File:8m span- devided in to 6(1) - Frame Span Loads (WIND(max)) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:23:28

WIND LOAD ASSIGNMENT - 8.0m SPAN ROOF STRUCTURE

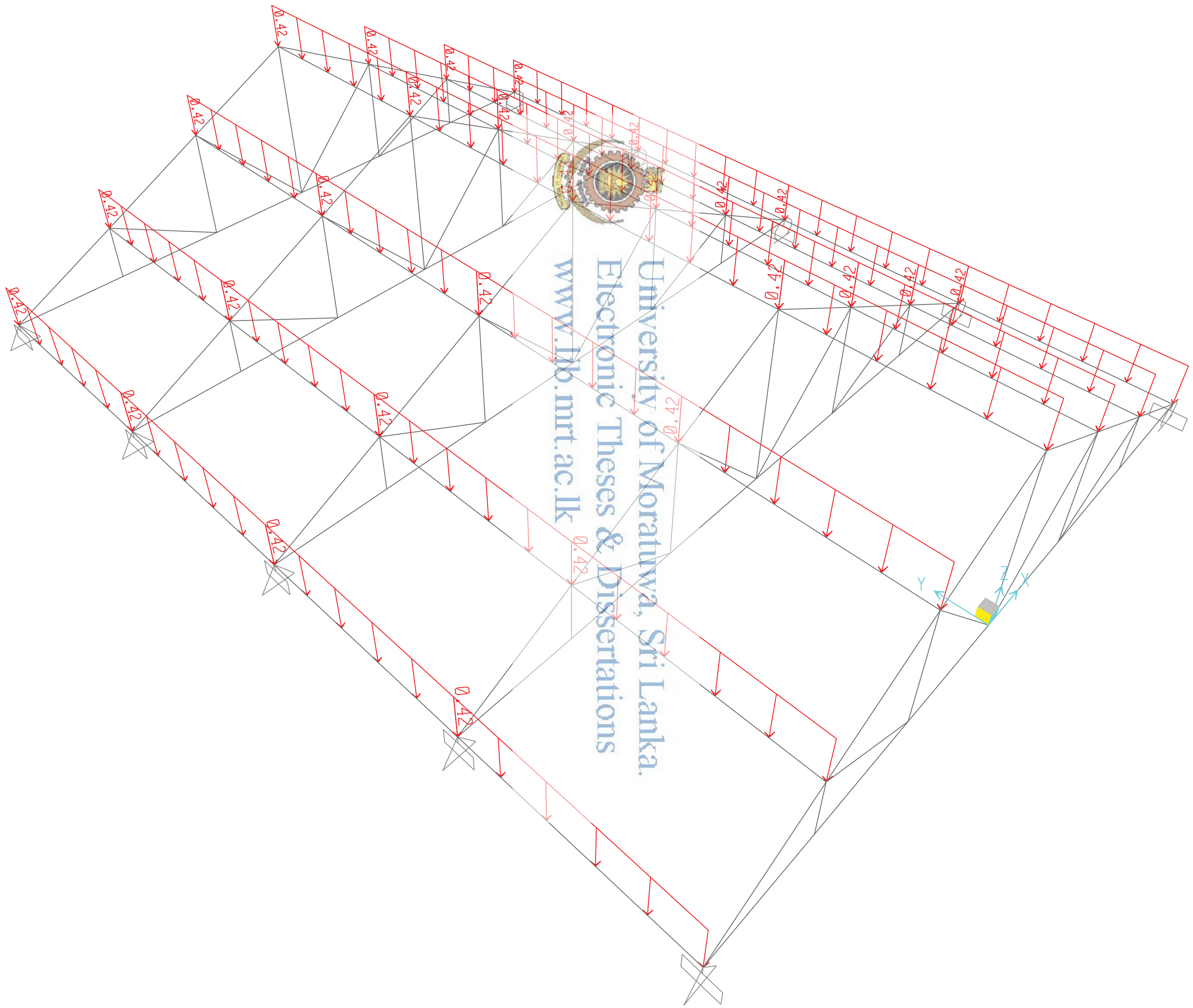


SAP2000 v14.0.0 - File:8m span- devided in to 6(1) - Frame Span Loads (WIND(min)) (As Defined) - KN, m, C Units

SAP2000

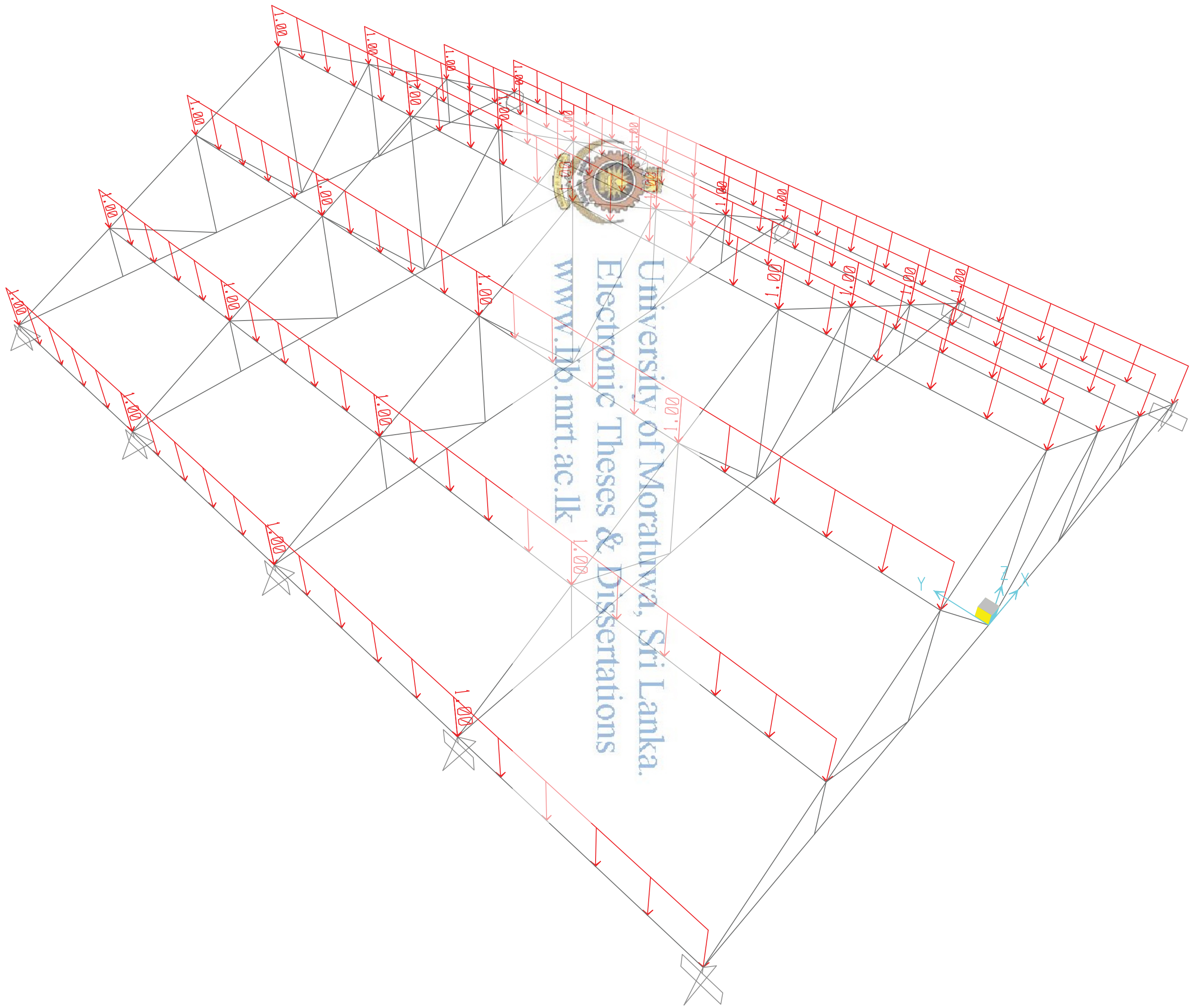
11-25-13 4:09:33

LIVE LOAD ASSIGNMENT - 10.0m SPAN ROOF STRUCTURE



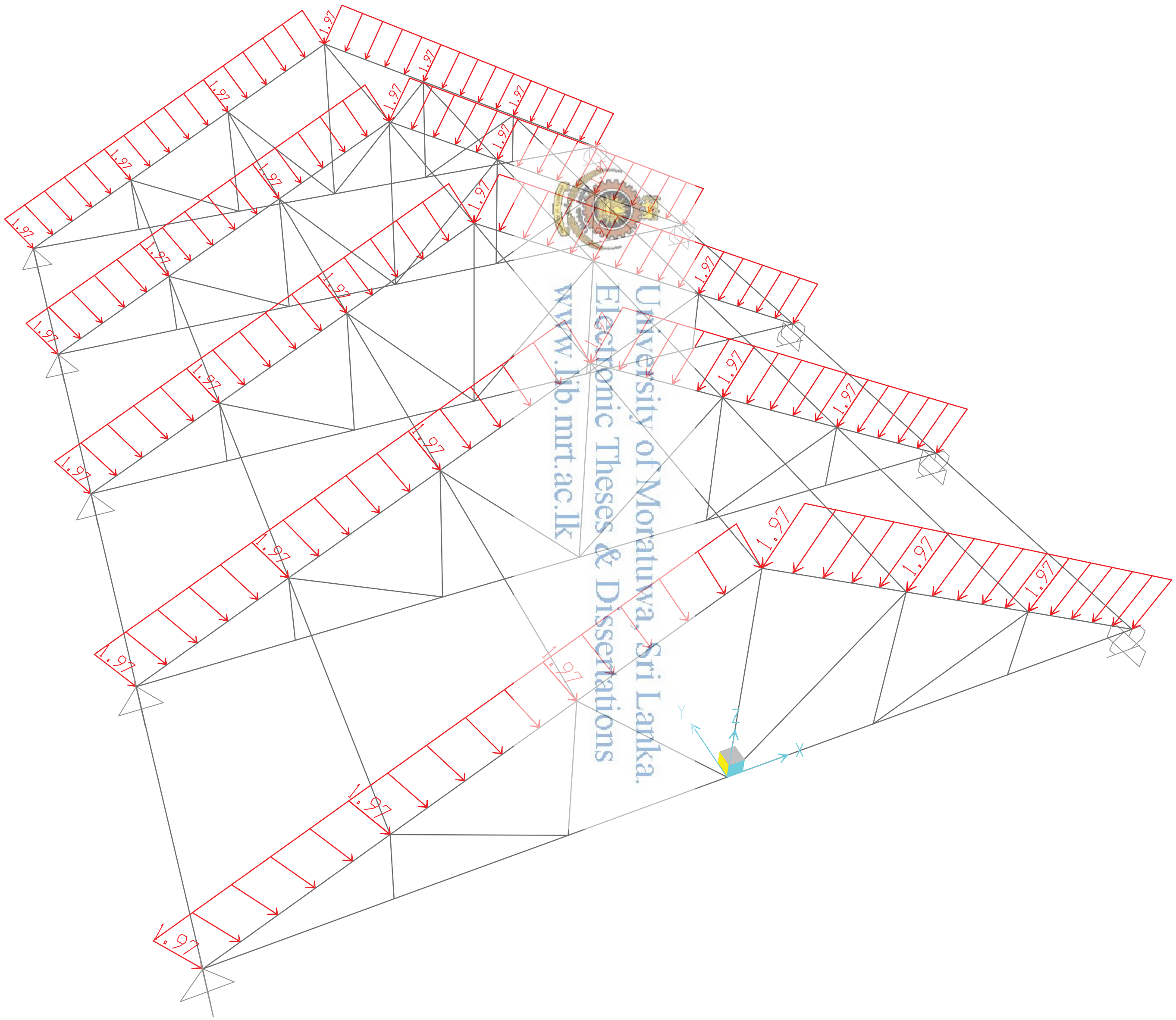
SAP2000 v14.0.0 - File:10m Span Truss - Frame Span Loads (LIVE) (As Defined) - KN, m, C Units

SUPER IMPOSED DEAD LOAD ASSIGNMENT - 10.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:10m Span Truss - Frame Distributed Loads (SUPER DEAD) - KN, m, C Units

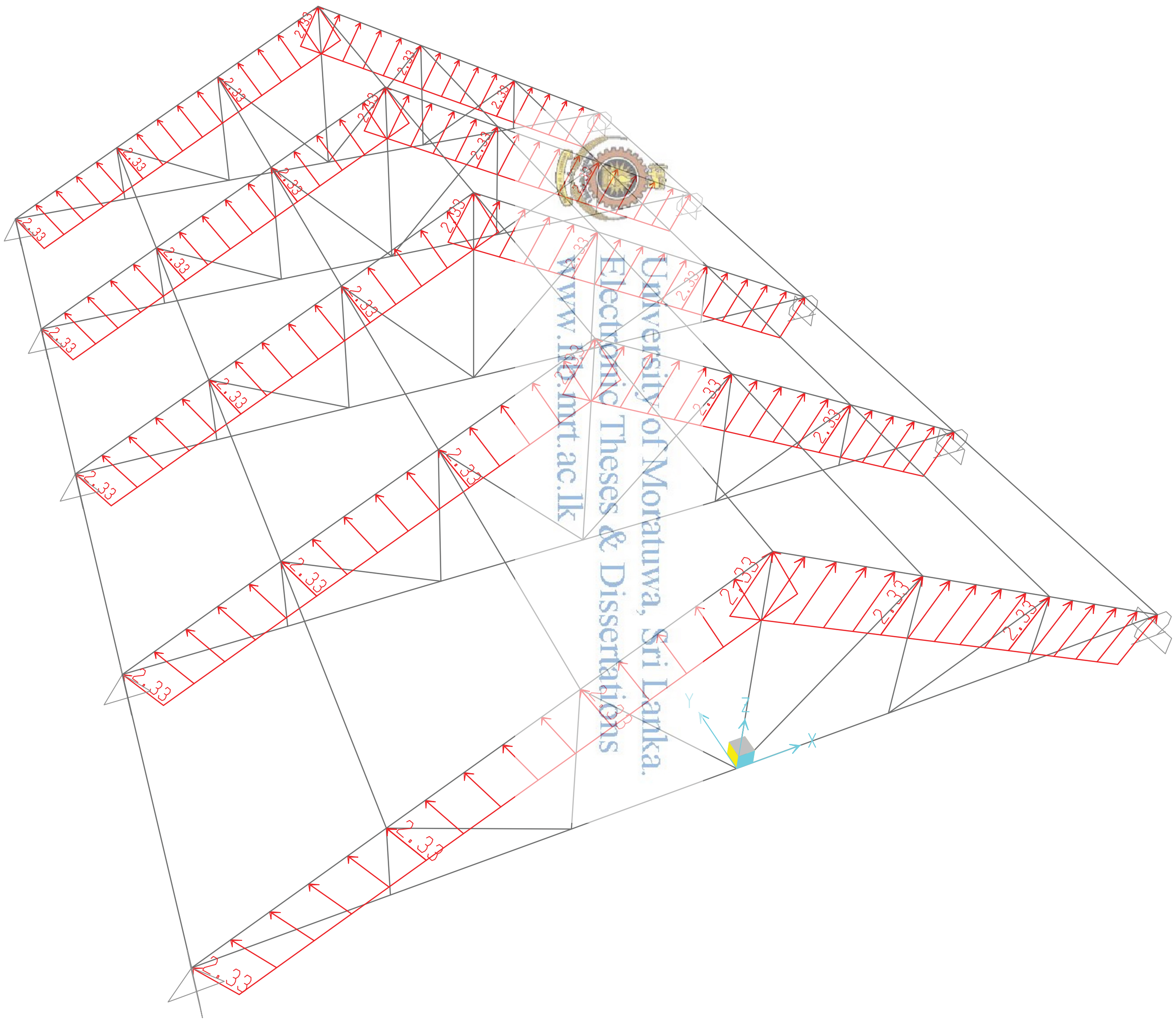
WIND LOAD ASSIGNMENT - 10.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:10m Span Truss - Frame Span Loads (WIND(max)) (As Defined) - KN, m, C Units

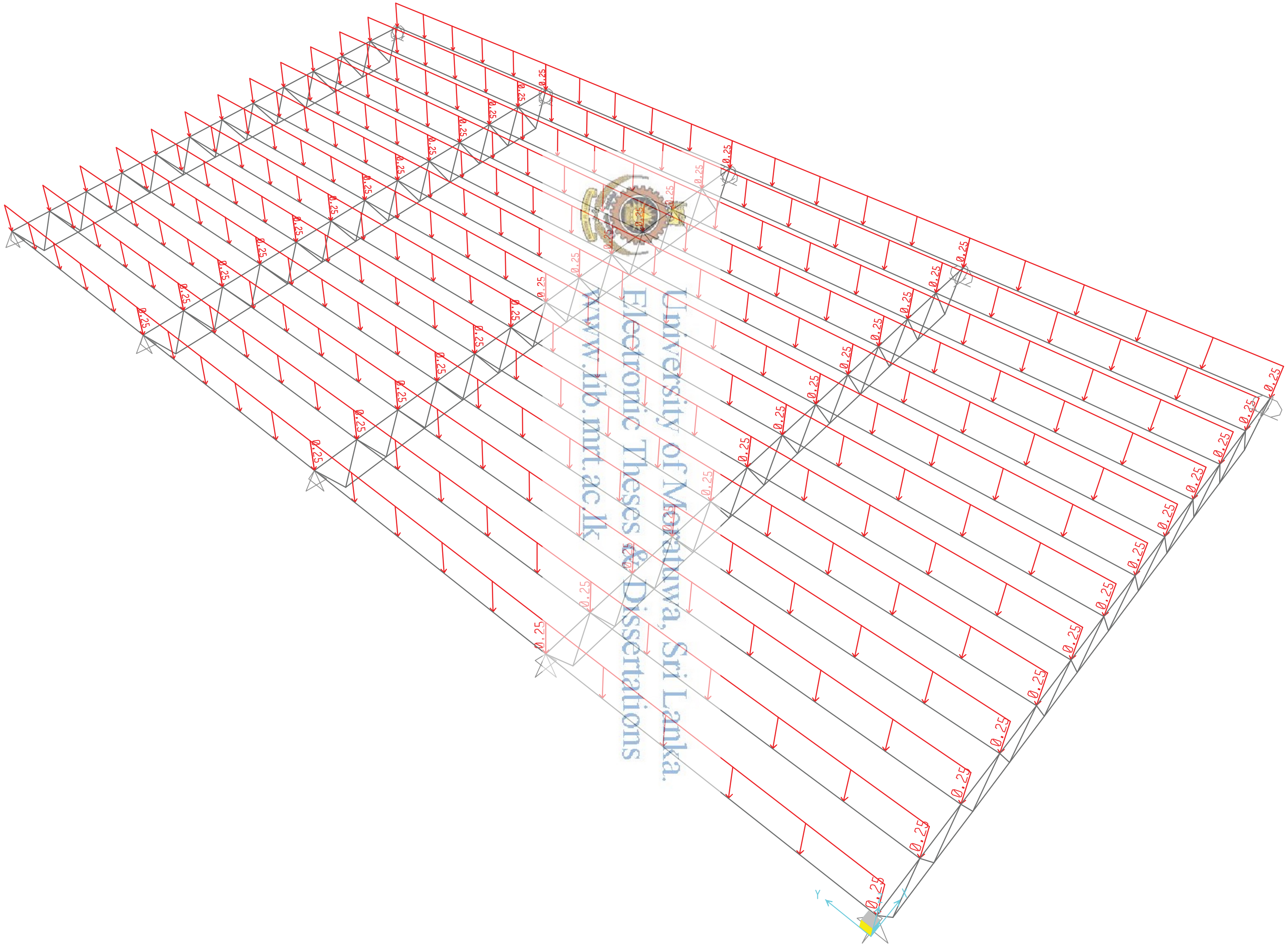
SAP2000 11-25-13 4:16:25

WIND LOAD ASSIGNMENT - 10.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:10m Span Truss - Frame Span Loads (WIND(min)) (As Defined) - KN, m, C Units

LIVE LOAD ASSIGNMENT - 12.0m SPAN ROOF STRUCTURE

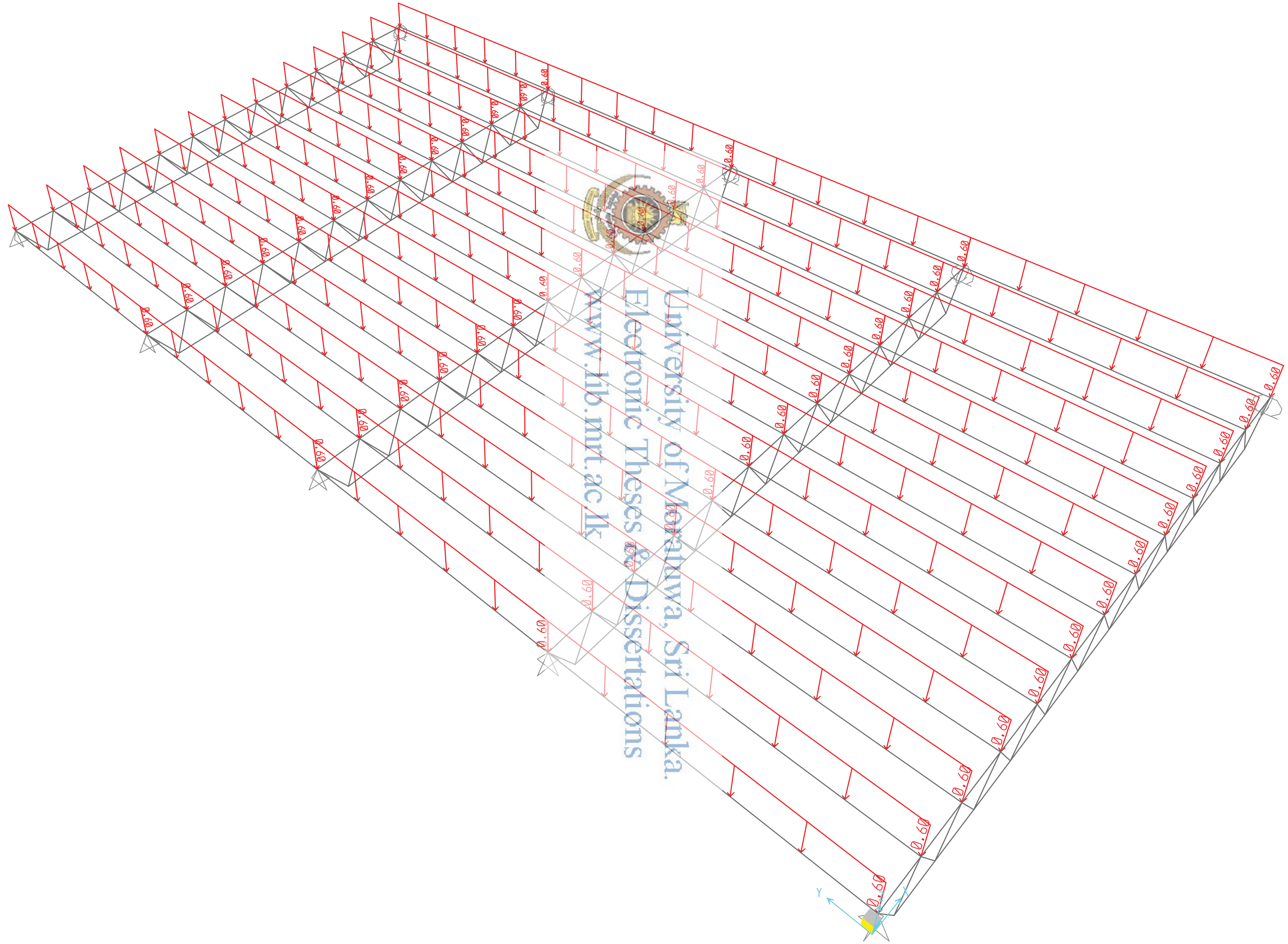


SAP2000 v14.0.0 - File:12m span- 0.8m pitch - Frame Span Loads (LIVE) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:27:08

SUPER IMPOSED DEAD LOAD ASSIGNMENT - 12.0m SPAN ROOF STRUCTURE

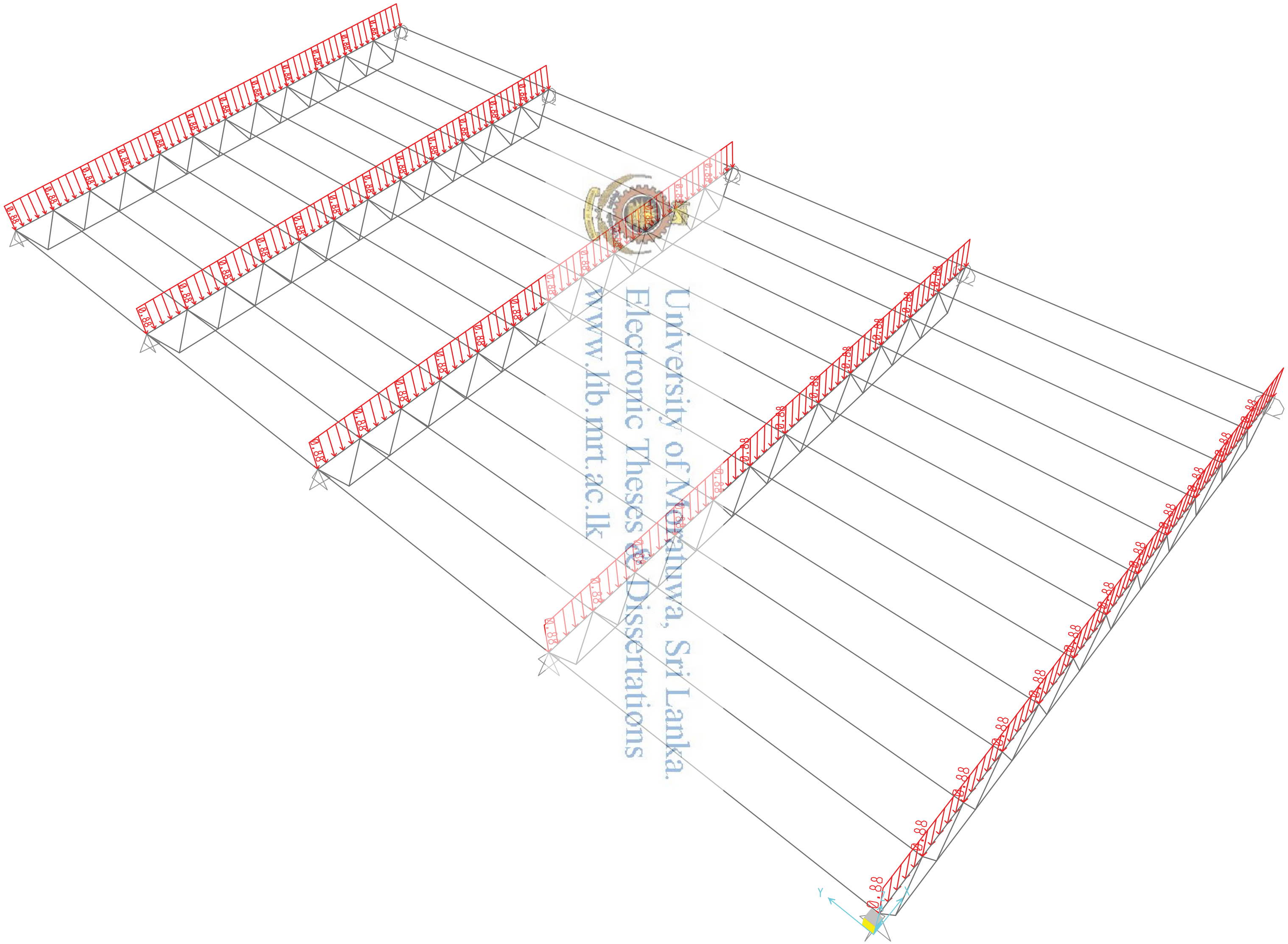


SAP2000 v14.0.0 - File:12m span- 0.8m pitch - Frame Span Loads (SUPER DEAD) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:28:10

WIND LOAD ASSIGNMENT - 12.0m SPAN ROOF STRUCTURE

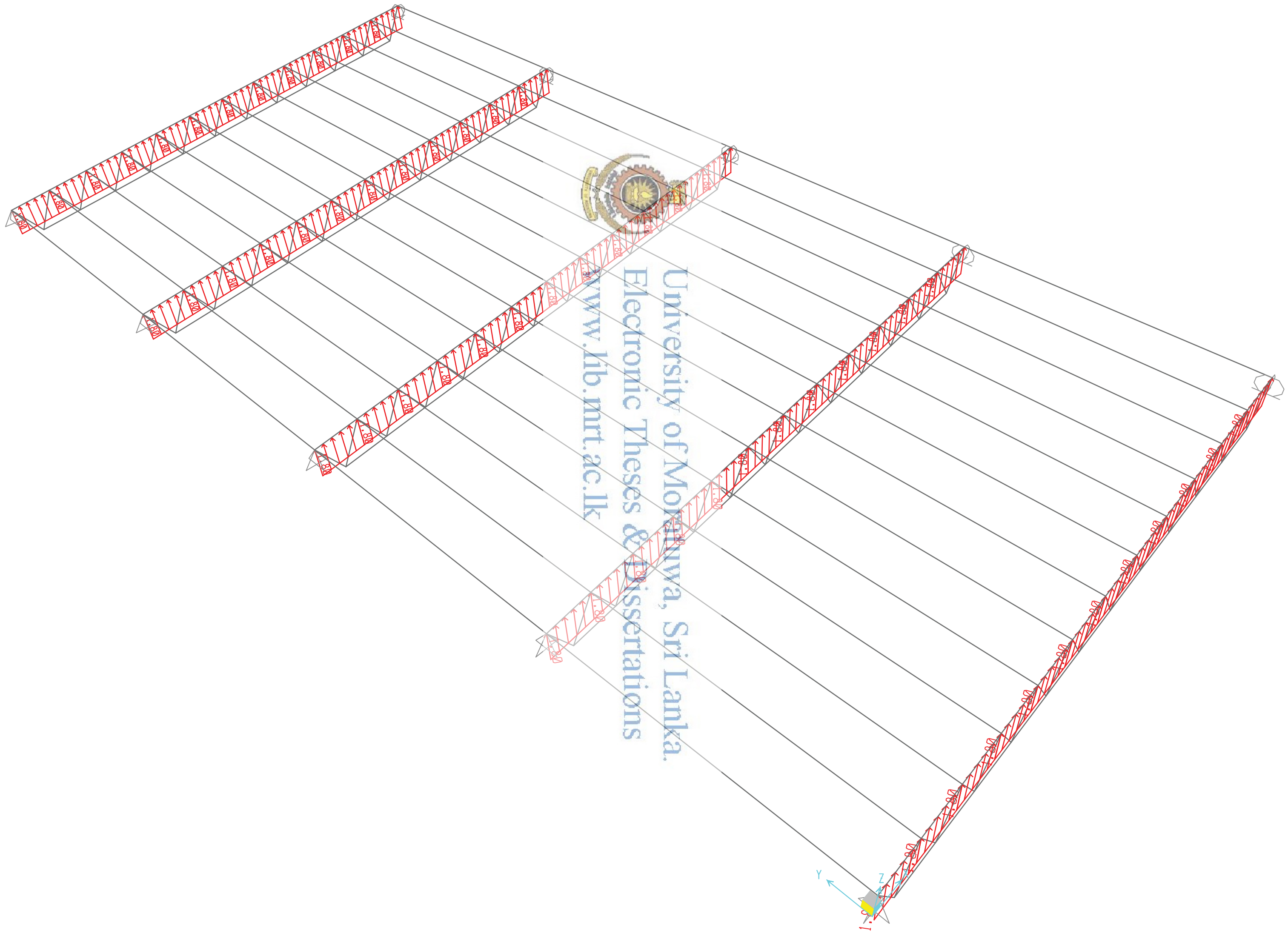


SAP2000 v14.0.0 - File:12m span- 0.8m pitch - Frame Span Loads (Wind(max)) (As Defined) - KN, m, C Units

SAP2000

11-25-13 4:29:29

WIND LOAD ASSIGNMENT - 12.0m SPAN ROOF STRUCTURE



SAP2000 v14.0.0 - File:12m span- 0.8m pitch - Frame Span Loads (Wind(min)) (As Defined) - KN, m, C Units

ANNEX : C - SECTION DESIGN

C.1 DESIGN OF COLD FORM SECTION

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Bot. chord	BNC	2014-03-24	

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	31	13	
Axial force	-	44.868	-8.442	kN
M_{xx}	-	0.089	-0.114	kNm
M_{yy}	-	0.098	0.000	kNm
Unrestrained length of section (L)	-	1.667		m

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	1.0*L*
Effectively held in position at both ends and restrained in direction at one end	0.85*L*
Effectively held in position and partially restrained in direction at both ends	0.85*L*
Effectively held in position and restrained in direction at both ends	0.7*L*
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	1.2*L*
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	1.5*L*
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	2.0*L*

Table. 09

Effective length Factor (L_E) - 1.00 L

Material Properties

E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Single C section
S 150 x 1.8
[C150x65x25(1.8)]

Gross Section Property

A	-	4.84	cm^2
I_{XX}	-	165.4	cm^4
I_{YY}	-	18.87	cm^4
d	-	150	mm
t	-	1.8	mm
r_{xx}	-	5.85	cm
r_{yy}	-	1.97	cm
b_2	-	65	mm
b_3	-	20	mm

Effective Length (L_E) - 1.667 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Bot. chord	BNC	2014-03-24	

Members in Compression

Cl 6.2.2 Slenderness ratio $\left(\dfrac{L_E}{r_{xx}}\right)$ - 28.51

$\left(\dfrac{L_E}{r_{yy}}\right)$ - 84.41

Maximum Slenderness limit

1) For members resisting loads other than wind load	-	180
2) For members resisting self weight and wind load	-	250
3) For members acting as ties with reversal stressing	-	350

load pattern with respect to Max.Slenderness limit - 2 — Max. slenderness OK

Cl 6.2.3 **Singly Symetrical Section**

Effective Cross-sectional area

a) Effective Breadth of Web Element

Annex B

f_c - 280 N/mm^2

K - 5.54

For Lipped Channel : $K_1 = 7 - \dfrac{1.8h}{0.15+h} - 1.43h^3$

$h = \dfrac{b_2}{b_1}$ - 0.44

b_1 - 148.2 mm

b_2 - 65.0 mm

t_1 - 1.76 mm

t_2 - 1.76 mm

$P_{cr} = 0.904EK_1\left(\dfrac{t}{b}\right)^2$ - 144.75 N/mm^2

$\dfrac{f_c}{P_{cr}}$ - 1.93 > 0.123

For $\dfrac{f_c}{P_{cr}} < 0.123;$ $\dfrac{b_{eff}}{b} = 1$

For $\dfrac{f_c}{P_{cr}} > 0.123$ $\dfrac{b_{eff}}{b} = \left[1+14\left\{\left(\sqrt{\dfrac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

DESIGN CALCULATION	By	Date
PART : Cold form -10.0m Span Truss-Bot. chord	BNC	2014-03-24

$\frac{b_{eff}}{b}$ - 0.565

b_{eff} - 83.67 mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ - 1.07

$P_{cr} = 0.904\, E K_2 \left(\frac{t}{b}\right)^2$ - 543.5 N/mm^2

$\frac{f_c}{P_{cr}}$ - 0.515 > 0.123

$\frac{b_{eff}}{b}$ - 0.955

b_{eff} - 62.10 mm

Q - Factore Representing Reduced Cross Section

Q - 0.801

A_{eff} - 387.68 mm^2

P_{cs} - 108.55 kN

P_E - 137.4 kN

perry coefficient η - 0.13

ϕ - 131.9

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}}\right)$$

P_C - 82.20 kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{1.13 - 0.0019 \frac{D_w}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\} p_y$$ 271.06 N/mm^2

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$M_{cx} = p_0 \times Z_{xr}$ - 5.784 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$M_{cy} = p_0 \times Z_{y1r}$ - 1.770 kNm

outstand in compression

$M_c = p_0 \times Z_{y2r}$ - 1.309 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4

Buckling Resistance Under Axial Load for Single symmetrical section

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 70.12 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 66.67 kN

Basic Requirment $F_c < (P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 8.44 kN

Buckling Resistance OK

Cl 6.4

Combined bending and Compression

Applied Bending Moment about X-X Axis

M_x - 0.114 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

Cl 6.4.2

Local Capacity Check $$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.078 + 0.020 + 0.000 - 0.098 — Local Capacity OK

Cl 3.5.4

Net corss sectional area - A_{net}

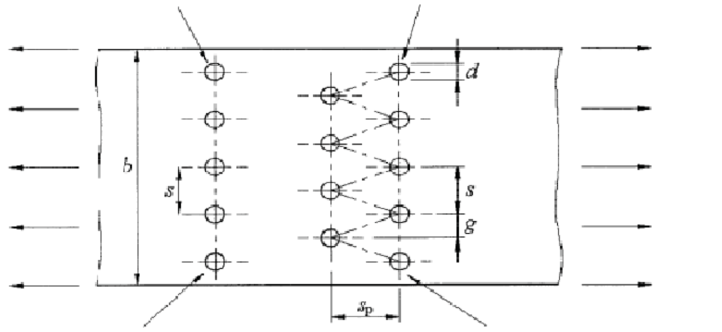


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes - 3
Holes Pattern - Zig-Zag
Hole Diameter - 8.00 mm
S_p - 20
g - 60
A_{net} - 232.8 mm²

Members in Tension

Cl 7.2.2

Tension Capcity

$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg
a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3

b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above
a_2 - as above

a_1 - 232.80 mm²
a_2 - 110.52 mm²

Connection type - a)

A_e - 328.22 mm^2

-

p_t - 91.90 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 44.868 kN

Applied Bending Moment about X-X Axis

M_x - 0.089 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.098 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.49 + 0.02 + 0.06 - 0.56 Local Capacity OK

DESIGN CALCULATION	By	Date
PART : Cold form -10.0m Span Truss-Bracing Member	BNC	2014-03-24

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	20	62	
Axial force	-	22.105	-14.673	kN
M_{xx}	-	0.000	-0.012	kNm
M_{yy}	-	0.024	0.000	kNm

Unrestrained length of section (L) - 2.453 m

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	1.0*L*
Effectively held in position at both ends and restrained in direction at one end	0.85*L*
Effectively held in position and partially restrained in direction at both ends	0.85*L*
Effectively held in position and restrained in direction at both ends	0.7*L*
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	1.2*L*
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	1.5*L*
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	2.0*L*

Table.09

Effective length Factor (L_E) 1.00 L

Material Properties E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Single C section

S 100 x 1.2

[C100x50x20]

Gross Section Property

A	-	2.67	cm^2
I_{XX}	-	44.31	cm^4
I_{YY}	-	11.49	cm^4
d	-	100	mm
t	-	1.2	mm
r_{xx}	-	4.07	cm
r_{yy}	-	2.07	cm
b_2	-	50	mm
b_3	-	20	mm

Effective Length (L_E) - 2.453 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Bracing Member	BNC	2014-03-24	

Members in Compression

Cl 6.2.2

Slenderness ratio $\left(\dfrac{L_E}{r_{xx}}\right)$ - 60.22

$\left(\dfrac{L_E}{r_{yy}}\right)$ - 118.25

Maximum Slenderness limit

1) For members resisting loads other than wind load	-	180
2) For members resisting self weight and wind load	-	250
3) For members acting as ties with reversal stressing	-	350

load pattern with respect to Max.Slenderness limit - 2 — Max. slenderness OK

Cl 6.2.3

Singly Symetrical Section

Effective Cross-sectional area

a) Effective Breadth of Web Element

Annex B

f_c - 280 N/mm^2

K_1 - 5.43

For Lipped Channel : K_1 = $7-\dfrac{1.8h}{0.15+h}-1.43h^3$

$h=\dfrac{b_2}{b_1}$ - 0.51

b_1 - 98.8 mm

b_2 - 50.0 mm

t_1 - 1.16 mm

t_2 - 1.16 mm

$P_{cr}=0.904EK_1\left(\dfrac{t}{b}\right)^2$ - 138.62 N/mm^2

$\dfrac{f_c}{P_{cr}}$ - 2.02 > 0.123

For $\dfrac{f_c}{P_{cr}}$ < 0.123; $\dfrac{b_{eff}}{b}$ = 1

For $\dfrac{f_c}{P_{cr}}$ > 0.123 $\dfrac{b_{eff}}{b}$ = $\left[1+14\left\{\left(\sqrt{\dfrac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

$\frac{b_{eff}}{b}$	-	0.552	
b_{eff}	-	54.58	mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\frac{t_1}{t_2} \right)^2$	-	1.39	
$P_{cr} = 0.904\, EK_2 \left(\frac{t}{b} \right)^2$	-	399.0	N/mm^2
$\frac{f_c}{P_{cr}}$	-	0.702	> 0.123
$\frac{b_{eff}}{b}$	-	0.890	
b_{eff}	-	44.49	mm

Q - Factore Representing Reduced Cross Section

	Q	-	0.796	
	A_{eff}	-	212.53	mm^2
	P_{cs}	-	59.51	kN
	P_E	-	38.6	kN
perry coefficient	η	-	0.20	
	ϕ	-	52.9	

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}} \right)$$

P_C	-	30.60	kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{ 1.13 - 0.0019 \frac{D_w}{t} \left(\frac{Y_s}{280} \right)^{1/2} \right\} p_y \quad 270.54 \text{ N/mm}^2$$

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$$M_{cx} = p_0 \times Z_{xr}$$ - 2.237 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$$M_{cy} = p_0 \times Z_{y1r}$$ - 1.023 kNm

outstand in compression

$$M_c = p_0 \times Z_{y2r}$$ - 0.871 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4 **Buckling Resistance Under Axial Load for Single symmetrical section**

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ 27.64 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 27.19 kN

Basic Requirment F_c < $(P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 14.67 kN — Buckling Resistance OK

Cl 6.4 **Combined bending and Compression**

Applied Bending Moment about X-X Axis

M_x - 0.012 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

Cl 6.4.2 **Local Capacity Check**

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

DESIGN CALCULATION	By	Date
PART : Cold form -10.0m Span Truss-Bracing Member	BNC	2014-03-24

0.247 + 0.005 + 0.000 - 0.252 — Local Capacity OK

Cl 3.5.4 Net corss sectional area - A_{net}

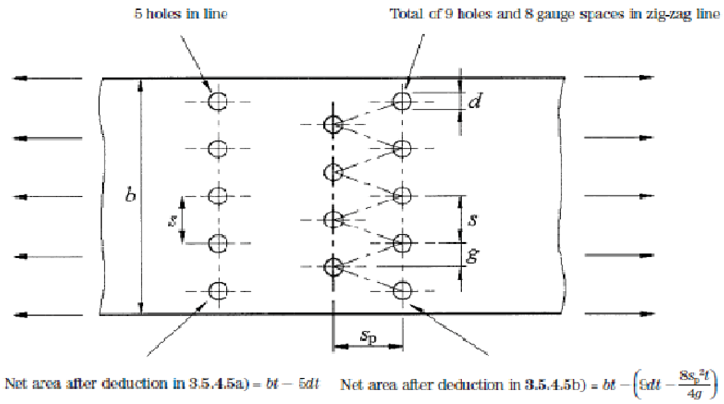


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes - 3
Holes Pattern - Zig-Zag
Hole Diameter - 8.00 mm
S_p - 20
g - 60
A_{net} - 95.2 mm²

Members in Tension

Cl 7.2.2 Tension Capcity

$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg
a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3

b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above
a_2 - as above

a_1 - 95.20 mm²
a_2 - 57.12 mm²

Connection type - a)

A_e - 142.80 mm^2

-

p_t - 39.98 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 22.105 kN

Applied Bending Moment about X-X Axis

M_x - 0.000 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.024 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.55 + 0.00 + 0.02 - 0.58 Local Capacity OK

DESIGN CALCULATION	By	Date
PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	14	33	
Axial force	-	10.957	-52.221	kN
M_{xx}	-	0.749	-0.292	kNm
M_{yy}	-	-0.001	-0.017	kNm

Unrestrained length of section (L) - 1.894 m

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	1.0*L*
Effectively held in position at both ends and restrained in direction at one end	0.85*L*
Effectively held in position and partially restrained in direction at both ends	0.85*L*
Effectively held in position and restrained in direction at both ends	0.7*L*
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	1.2*L*
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but nct held in	1.5*L*
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	2.0*L*

Table. 09

Effective length Factor (L_E) - 1.00 L

Material Properties E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Single C section
S 150 x 1.8
[C150x65x25(1.8)]

Gross Section Property

A	-	4.84	cm^2
I_{XX}	-	165.4	cm^4
I_{YY}	-	18.87	cm^4
d	-	150	mm
t	-	1.8	mm
r_{xx}	-	5.85	cm
r_{yy}	-	1.97	cm
b_2	-	65	mm
b_3	-	20	mm

Effective Length (L_E) - 1.894 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24	

Members in Compression

Cl 6.2.2

Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 32.40

$\left(\frac{L_E}{r_{yy}}\right)$ - 95.93

Maximum Slenderness limit

1) For members resisting loads other than wind load	-	180
2) For members resisting self weight and wind load	-	250
3) For members acting as ties with reversal stressing	-	350

load pattern with respect to Max.Slenderness limit - 2

Max. slenderness OK

Cl 6.2.3

Singly Symetrical Section

Effective Cross-sectional area

a) Effective Breadth of Web Element

Annex B

f_c - 280 N/mm^2

K_1 5.54

For Lipped Channel : K_1 = $7-\frac{1.8h}{0.15+h}-1.43h^3$

$h=\frac{b_2}{b_1}$ - 0.44

b_1 - 148.2 mm

b_2 - 65.0 mm

t_1 - 1.76 mm

t_2 - 1.76 mm

$P_{cr}=0.904EK_1\left(\frac{t}{b}\right)^2$ - 144.75 N/mm^2

$\frac{f_c}{P_{cr}}$ - 1.93 > 0.123

For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{eff}}{b}$ = 1

For $\frac{f_c}{P_{cr}}$ > 0.123 $\frac{b_{eff}}{b}$ = $\left[1+14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

DESIGN CALCULATION		By	Date
PART :	Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24

$\frac{b_{eff}}{b}$	-	0.565	
b_{eff}	-	83.67	mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$	-	1.07	
$P_{cr} = 0.904\, EK_2 \left(\frac{t}{b}\right)^2$	-	543.5	N/mm^2
$\frac{f_c}{P_{cr}}$	-	0.515	> 0.123
$\frac{b_{eff}}{b}$	-	0.955	
b_{eff}	-	62.10	mm

Q - Factore Representing Reduced Cross Section

Q	-	0.801	
A_{eff}	-	387.68	mm^2

	P_{cs}	-	108.55	kN
	P_E	-	106.4	kN
perry coefficient	η	-	0.15	
	ϕ	-	115.6	

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}}\right)$$

P_C	-	73.10	kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{1.13 - 0.0019 \frac{D_w}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\} p_y$$ 271.06 N/mm^2

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$M_{cx} = p_0 \times Z_{xr}$ - 5.784 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$M_{cy} = p_0 \times Z_{y1r}$ - 1.770 kNm

outstand in compression

$M_c = p_0 \times Z_{y2r}$ - 1.309 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4

Buckling Resistance Under Axial Load for Single symmetrical section

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 63.39 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 60.55 kN

Basic Requirment $F_c < (P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 52.22 kN | Buckling Resistance OK

Cl 6.4

Combined bending and Compression

Applied Bending Moment about X-X Axis

M_x - 0.292 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.017 kNm

Cl 6.4.2

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$$

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24	

0.481 + 0.050 + 0.013 - 0.544 Local Capacity OK

Cl 3.5.4 Net corss sectional area - A_{net}

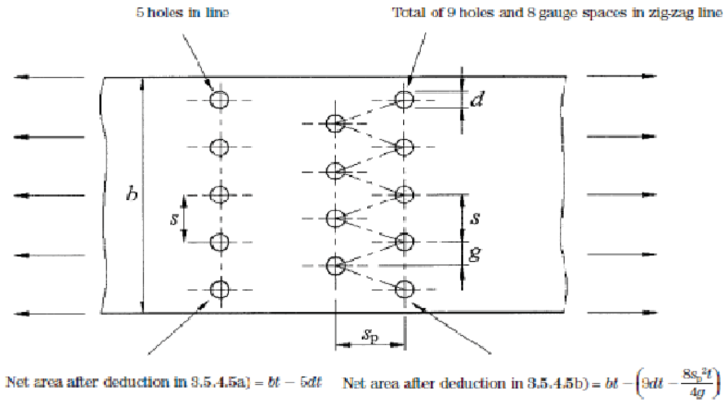


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes - 3

Holes Pattern - Zig-Zag

Hole Diameter - 8.00 mm

S_p - 20

g - 60

A_{net} - 232.8 mm²

Members in Tension

Cl 7.2.2 Tension Capcity

$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg

a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3

b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above

a_2 - as above

a_1 - 232.80 mm²

a_2 - 110.52 mm²

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24	

Connection type - a)

A_e - 328.22 mm^2

-

p_t - 91.90 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 10.957 kN

Applied Bending Moment about X-X Axis

M_x - 0.749 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.001 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.12 + 0.13 + 0.00 - 0.25

Local Capacity OK

	DESIGN CALCULATION	By	Date
	PART : Cold form -12.0m Span Truss-Bottom chord	BNC	2014-03-22

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	687	682	
Axial force	-	146.877	2.785	kN
M_{xx}	-	2.302	-1.763	kNm
M_{yy}	-	0.000	0.000	kNm
Unrestrained length of section (L)	-	1.010		m

Table. 09

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	$1.0L$
Effectively held in position at both ends and restrained in direction at one end	$0.85L$
Effectively held in position and partially restrained in direction at both ends	$0.85L$
Effectively held in position and restrained in direction at both ends	$0.7L$
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	$1.2L$
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	$1.5L$
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	$2.0L$

Effective length Factor (L_E) - 1.00 L

Material Properties

E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Double C section

D 200 x 2.0

[2/C200x75x20 -Back to Back Double Section]

Gross Section Property

A	-	13.46	cm^2
I_{XX}	-	797.3	cm^4
I_{YY}	-	106.4	cm^4
d	-	200	mm
t	-	2	mm
r_{xx}	-	7.70	cm
r_{yy}	-	2.81	cm
b_2	-	150	mm
b_3	-	20	mm

Effective Length (L_E) - 1.010 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Bottom chord	BNC	2014-03-22	

Members in Compression

Cl 6.2.2 Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 13.12

$\left(\frac{L_E}{r_{yy}}\right)$ - 35.92

Maximum Slenderness limit

1) For members resisting loads other than wind load - 180
2) For members resisting self weight and wind load - 250
3) For members acting as ties with reversal stressing - 350

load pattern with respect to Max.Slenderness limit - 2 — Max. slenderness OK

Cl 6.2.3

Singly Symetrical Section

Effective Cross-sectional area

a) Effective Breadth of Web Element

Annex B

f_c - 280 N/mm^2

K_1 - 4.88

For Lipped Channel : K_1 = $7 - \frac{1.8h}{0.15+h} - 1.43h^3$

$h = \frac{b_2}{b_1}$ - 0.76

b_1 - 198.0 mm

b_2 - 150.0 mm

t_1 - 1.96 mm

t_2 - 1.96 mm

$P_{cr} = 0.904EK_1\left(\frac{t}{b}\right)^2$ - 88.54 N/mm^2

$\frac{f_c}{P_{cr}}$ - 3.16 > 0.123

For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{eff}}{b}$ = 1

For $\frac{f_c}{P_{cr}}$ > 0.123 $\frac{b_{eff}}{b}$ = $\left[1+14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

DESIGN CALCULATION	By	Date
PART : Cold form -12.0m Span Truss-Bottom chord	BNC	2014-03-22

$\dfrac{b_{eff}}{b}$ - 0.442

b_{eff} - 87.52 mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\dfrac{t_1}{t_2} \right)^2$ - 2.80

$P_{cr} = 0.904 E K_2 \left(\dfrac{t}{b} \right)^2$ - 126.6 N/mm^2

$\dfrac{f_c}{P_{cr}}$ - 2.212 > 0.123

$\dfrac{b_{eff}}{b}$ - 0.528

b_{eff} - 79.16 mm

Q - Factore Representing Reduced Cross Section

Q - 0.73

A_{eff} - 982.58 mm^2

P_{cs} - 275.12 kN

P_E - 2111.1 kN

perry coefficient η - 0.03

ϕ - 1226.7

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}} \right)$$

P_C - 265.46 kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{ 1.13 - 0.0019 \frac{D_w}{t} \left(\frac{Y_s}{280} \right)^{1/2} \right\} p_y$$ 262.11 N/mm^2

DESIGN CALCULATION | By | Date

PART : Cold form -12.0m Span Truss-Bottom chord | BNC | 2014-03-22

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$$M_{cx} = p_0 \times Z_{xr}$$ - 19.554 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$$M_{cy} = p_0 \times Z_{y1r}$$ - 4.293 kNm

outstand in compression

$$M_c = p_0 \times Z_{y2r}$$ - 4.293 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4 **Buckling Resistance Under Axial Load for Single symmetrical section**

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 265.46 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 265.46 kN

Basic Requirment $F_c < (P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 2.79 kN

Buckling Resistance OK

Cl 6.4 **Combined bending and Compression**

Applied Bending Moment about X-X Axis

M_x - 1.763 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

Cl 6.4.2 **Local Capacity Check**

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$$

0.010 + 0.090 + 0.000 - 0.100 — Local Capacity OK

Cl 3.5.4 Net corss sectional area - A_{net}

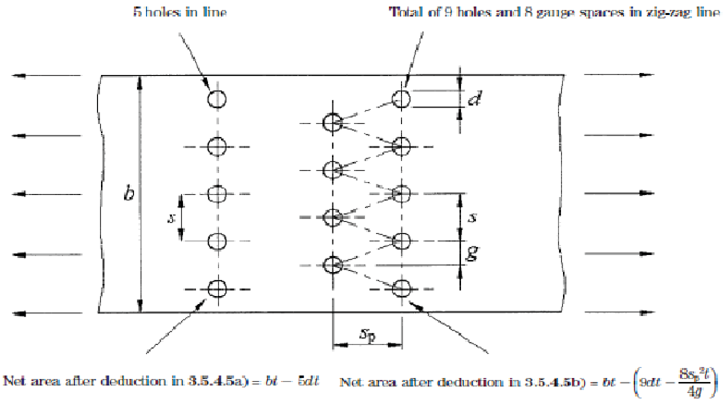


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.00 mm
s_p	-	30
g	-	60
A_{net}		734.0 mm²

Members in Tension

Cl 7.2.2 Tension Capcity

$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg
a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3 b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above
a_2 - as above

a_1 - 734.00 mm²
a_2 - 292.00 mm²

Connection type - b)

A_e - 1004.48 mm^2

-

p_t - 281.25 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 146.877 kN

Applied Bending Moment about X-X Axis

M_x - 2.302 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.52 + 0.12 + 0.00 - 0.64

Local Capacity OK

DESIGN CALCULATION		By	Date
PART :	Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	634	632	
Axial force	-	53.956	-47.702	kN
M_{xx}	-	1.578	0.064	kNm
M_{yy}	-	0.000	-0.005	kNm
Unrestrained length of section (L)	-	1.032		m

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	1.0*L*
Effectively held in position at both ends and restrained in direction at one end	0.85*L*
Effectively held in position and partially restrained in direction at both ends	0.85*L*
Effectively held in position and restrained in direction at both ends	0.7*L*
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	1.2*L*
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	1.5*L*
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	2.0*L*

Table. 09

Effective length Factor (L_E) 1.00 L

Material Properties

E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Single C section
S 200 x 2.0
[C200x75x20]

Gross Section Property

A	-	6.73	cm^2
I_{XX}	-	398.7	cm^4
I_{YY}	-	34.03	cm^4
d	-	200	mm
t	-	2	mm
r_{xx}	-	7.70	cm
r_{yy}	-	2.25	cm
b_2	-	75	mm
b_3	-	20	mm

Effective Length (L_E) - 1.032 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22	

Members in Compression

Cl 6.2.2

Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 13.41

$\left(\frac{L_E}{r_{yy}}\right)$ - 45.89

Maximum Slenderness limit

1) For members resisting loads other than wind load	-	180
2) For members resisting self weight and wind load	-	250
3) For members acting as ties with reversal stressing	-	350

load pattern with respect to Max.Slenderness limit - 2 — Max. slenderness OK

Cl 6.2.3

Singly Symetrical Section

Effective Cross-sectional area

a) Effective Breadth of Web Element

f_c - 280 N/mm^2

Annex B

K

For Lipped Channel : K_1 = $7-\frac{1.8h}{0.15+h}-1.43h^3$ 5.63

$h=\frac{b_2}{b_1}$ - 0.38

b_1 - 198.0 mm

b_2 - 75.0 mm

t_1 - 1.96 mm

t_2 - 1.96 mm

$P_{cr}=0.904EK_1\left(\frac{t}{b}\right)^2$ - 102.29 N/mm^2

$\frac{f_c}{P_{cr}}$ - 2.74 > 0.123

For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{eff}}{b}$ = 1

For $\frac{f_c}{P_{cr}}$ > 0.123 $\frac{b_{eff}}{b}$ = $\left[1+14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

DESIGN CALCULATION	By	Date
PART : Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22

$\frac{b_{eff}}{b}$ - 0.475

b_{eff} - 93.97 mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ - 0.81

$P_{cr} = 0.904\, EK_2 \left(\frac{t}{b}\right)^2$ - 506.3 N/mm^2

$\frac{f_c}{P_{cr}}$ - 0.553 > 0.123

$\frac{b_{eff}}{b}$ - 0.944

b_{eff} - 70.77 mm

Q - Factore Representing Reduced Cross Section

Q - 0.727

A_{eff} - 489.27 mm^2

P_{cs} - 137.00 kN

P_E - 646.5 kN

perry coefficient η - 0.05

ϕ - 408.5

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}}\right)$$

P_C - 128.68 kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{1.13 - 0.0019\frac{D_w}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\} p_y$$ 262.11 N/mm^2

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22	

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$$M_{cx} = p_0 \times Z_{xr}$$ - 9.777 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$$M_{cy} = p_0 \times Z_{y1r}$$ - 2.506 kNm

outstand in compression

$$M_c = p_0 \times Z_{y2r}$$ - 1.856 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4

Buckling Resistance Under Axial Load for Single symmetrical section

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 98.64 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 91.18 kN

Basic Requirment F_c < $(P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 47.70 kN

Buckling Resistance OK

Cl 6.4

Combined bending and Compression

Applied Bending Moment about X-X Axis

M_x - 0.064 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.005 kNm

Cl 6.4.2

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22	

0.348 + 0.007 + 0.003 - 0.358

Local Capacity OK

Cl 3.5.4

Net corss sectional area - A_{net}

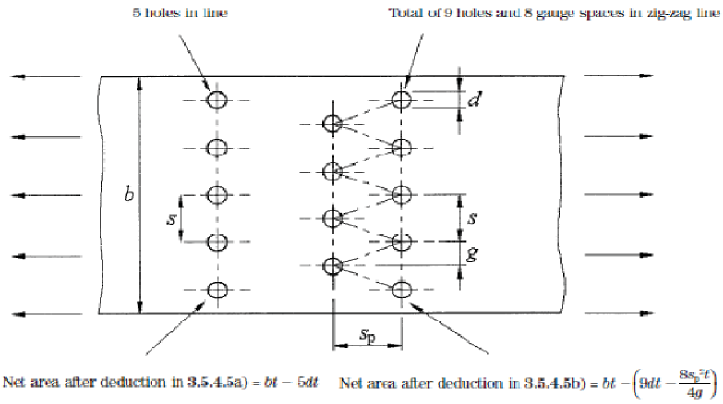


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes - 3

Holes Pattern - Zig-Zag

Hole Diameter - 8.00 mm

s_p - 30

g - 60

A_{net} - 367.0 mm^2

Members in Tension

Cl 7.2.2

Tension Capcity

$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg

a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3

b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above

a_2 - as above

a_1 - 367.00 mm^2

a_2 - 142.00 mm^2

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22	

Connection type - a)

A_e - 492.78 mm^2

-

p_t - 137.98 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 53.956 kN

Applied Bending Moment about X-X Axis

M_x - 1.578 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.39 + 0.16 + 0.00 - 0.55

Local Capacity OK

DESIGN CALCULATION	By	Date
PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24

DETAILED DESIGN

		Tension	Compression	
Frame Text	-	702	696	
Axial force	-	-1.147	-145.559	kN
M_{xx}	-	-0.084	0.127	kNm
M_{yy}	-	0.000	0.000	kNm
Unrestrained length of section (L)	-	1.010		m

Table 9 — Effective lengths, L_E for compression members

Conditions of restraint at ends (in plane under consideration)	Effective length
Effectively held in position at both ends but not restrained in direction	$1.0L$
Effectively held in position at both ends and restrained in direction at one end	$0.85L$
Effectively held in position and partially restrained in direction at both ends	$0.85L$
Effectively held in position and restrained in direction at both ends	$0.7L$
Effectively held in position and restrained in direction at one end with the other end effectively restrained in direction but not held in position	$1.2L$
Effectively held in position and restrained in direction at one end with the other end partially restrained in direction but not held in	$1.5L$
Effectively held in position and restrained in direction at one end position but not held in position or restrained in direction at the other end	$2.0L$

Table 09

Effective length Factor (L_E) 1.00 L

Material Properties

E - 205.0 kN/mm^2

P_y - 280.0 N/mm^2

TYPE OF SECTION - Double C section

D 200 x 2.0

[2/C200x75x20 -Back to Back Double Section]

Gross Section Property

A	-	13.46	cm^2
I_{XX}	-	797.3	cm^4
I_{YY}	-	106.4	cm^4
d	-	200	mm
t	-	2	mm
r_{xx}	-	7.70	cm
r_{yy}	-	2.81	cm
b_2	-	150	mm
b_3	-	20	mm

Effective Length (L_E) - 1.010 m

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24	

Members in Compression

Cl 6.2.2

Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 13.12

$\left(\frac{L_E}{r_{yy}}\right)$ - 35.92

Maximum Slenderness limit

1) For members resisting loads other than wind load	-	180
2) For members resisting self weight and wind load	-	250
3) For members acting as ties with reversal stressing	-	350

load pattern with respect to Max.Slenderness limit - 2 — Max. slenderness OK

Cl 6.2.3

Singly Symetrical Section

Effective Cross-sectional area

a) Effective Breadth of Web Element

Annex B

f_c - 280 N/mm^2

K_1 - 4.88

For Lipped Channel : K_1 = $7 - \frac{1.8h}{0.15+h} - 1.43h^3$

$h = \frac{b_2}{b_1}$ - 0.76

b_1 - 198.0 mm

b_2 - 150.0 mm

t_1 - 1.96 mm

t_2 - 1.96 mm

$P_{cr} = 0.904EK_1\left(\frac{t}{b}\right)^2$ - 88.54 N/mm^2

$\frac{f_c}{P_{cr}}$ - 3.16 > 0.123

For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{eff}}{b}$ = 1

For $\frac{f_c}{P_{cr}}$ > 0.123 $\frac{b_{eff}}{b}$ = $\left[1+14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)-0.35\right\}^4\right]^{-0.2}$

$\frac{b_{eff}}{b}$ - 0.442

b_{eff} - 87.52 mm

b) Effective Breadth of Flange Element

$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ - 2.80

$P_{cr} = 0.904\, EK_2 \left(\frac{t}{b}\right)^2$ - 126.6 N/mm^2

$\frac{f_c}{P_{cr}}$ - 2.212 > 0.123

$\frac{b_{eff}}{b}$ - 0.528

b_{eff} - 79.16 mm

Q - Factore Representing Reduced Cross Section

Q - 0.73

A_{eff} - 982.58 mm^2

P_{cs} - 275.12 kN

P_E - 2111.1 kN

perry coefficient η - 0.03

ϕ - 1226.7

Buckling Resistance Under Axial Load for section symmetrical about Both axes or Closed section

$$P_c = \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}}\right)$$

P_C - 265.46 kN

Determination Of Moment Capacity

a) Limiting Stress for Stiffened Web in Bending

$$p_0 = \left\{1.13 - 0.0019 \frac{D_w}{t}\left(\frac{Y_s}{280}\right)^{1/2}\right\} p_y$$ 262.11 N/mm^2

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24	

b) Moment Capacity - M_c

i) Moment Capacity about X-X axise

$$M_{cx} = p_0 \times Z_{xr}$$ - 19.554 kNm

ii) Moment Capacity about Y-Y axise

outstand in tension

$$M_{cy} = p_0 \times Z_{y1r}$$ - 4.293 kNm

outstand in compression

$$M_c = p_0 \times Z_{y2r}$$ - 4.293 kNm

Where

Z_{xr} - Reduced section modulus for major axise bending

Z_{y1r} - Reduced section modulus for minor axise bending (outstand in tension)

Z_{y2r} - Reduced section modulus for minor axise bending (outstand in compression)

Cl 6.2.4

Buckling Resistance Under Axial Load for Single symmetrical section

i) P'_c (about Y-Y) Outstand in tension

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 265.46 kN

ii) P'_c (about Y-Y) Outstand in compression

$$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$$ - 265.46 kN

Basic Requirment $F_c < (P'_c)_{min}$

Applied Axial Load-Compression - F_C

F_C - 145.56 kN

Buckling Resistance OK

Cl 6.4

Combined bending and Compression

Applied Bending Moment about X-X Axis

M_x - 0.127 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

Cl 6.4.2

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24	

0.529 + 0.006 + 0.000 - 0.536

Local Capacity
OK

-

Cl 3.5.4

Net corss sectional area A_{net}

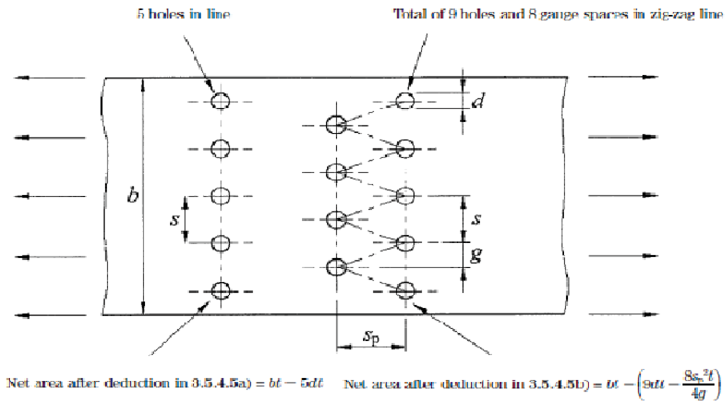


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.00 mm
s_p	-	20
g	-	60
A_{net}	-	717.3 mm^2

Members in Tension

Cl 7.2.2

Tension Capcity
$$P_t = A_e p_y$$

a) For single angle ties connected through one leg only
For plain channel section connected only through web
For "T" sections connected only through the flange

$$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$$

a_1 - Net sectional area of connected leg
a_2 - Gross sectional area of unconnected leg or legs

Cl 7.2.3

b) If two component are parellel back to back

$$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$$

a_1 - as above
a_2 - as above

a_1 - 717.33 mm^2
a_2 - 292.00 mm^2

	DESIGN CALCULATION	By	Date	
	PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24	

Connection type - b)

A_e - 987.35 mm^2

-

p_t - 276.46 kN

Cl 7.3 **Members in Combined Bending & Tension**

Applied Tensile Load F_t - 1.147 kN

Applied Bending Moment about X-X Axis

M_x - 0.084 kNm

Applied Bending Moment about Y-Y Axis

M_y - 0.000 kNm

$$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

0.00 + 0.00 + 0.00 - 0.01

Local Capacity OK

C.2 DESIGN OF HOT ROLLED SECTION

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -10.0m Span Truss-Bottom chord	BNC	2014-03-24

Cl 4.7

Design of Members

Sign convention	- Compression		-ve
	- Tension		+ve
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

Analysis Sumary

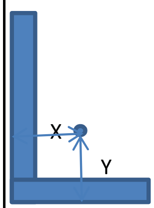
	Tension		Compression	
Member Assignment	2L3X3X1/4		2L3X3X1/4	
	Double Angle		Double Angle	
Frame Text	49		50	
Axial Forces	139.62	kN	113.40	kN
M_{xx}	0.241	kNm	-0.106	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	1.667	m	1.667	m

Section Propeties

Cross Section Area	A	-	1852	mm^2
Second Moment of Inertia	I_{xx}	-	1024000	mm^4
	I_{yy}	-	1883000	mm^4
Plastic section Modulus	S_{xx}	-	33000	mm^3
	S_{yy}	-	40000	mm^3
Section Modulus	Z_{xx}	-	19000	mm^3
	Z_{yy}	-	25000	mm^3
Radius of gyration	R_{xx}	-	23.51	mm
	R_{yy}	-	31.89	mm

For single Angle / Double Angle only

	t_w	-	6.35	mm
	t_f	-	6.35	mm
	t_3	-	76.20	mm
	t_2	-	152.40	mm
	Y	-	11.5	mm
	X	-	0.00	mm
Product of Moment of Inertia	I_{xy}	-	0	mm^4
	$I_{max(u-u)}$	-	0	mm^4
	$I_{min(v-v)}$	-	0	mm^4
	R_{uu}	-	0.00	mm
	R_{vv}	-	0.00	mm



Cl 3.5.1

Member Loacal Buckling Check

	$\frac{b}{t}$	-	11.75
	$\frac{d}{t}$	-	12
	ε	-	1

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -10.0m Span Truss-Bottom chord	BNC	2014-03-24	

Table 11 — 15. ε — 15

Section is Non Slender

Design of Compression Members

Cl.4.7.10.2		λ	-	79.62	
	Design Strength of Steel	p_y	-	275	N/mm^2
		λ_0	-	17.15	
		η	-	0.34	
		P_E	-	319.196	N/mm^2
		ϕ	-	351.93	
		p_c	-	162.00	N/mm^2

Compression Resistance

Cl.4.7.4 — P_c - $A_g p_c$

- 270.02 kN > 113.398 kN OK

Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

61.2 + 5.55 + 3.22E-13 - 66.78 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area - A_{net}

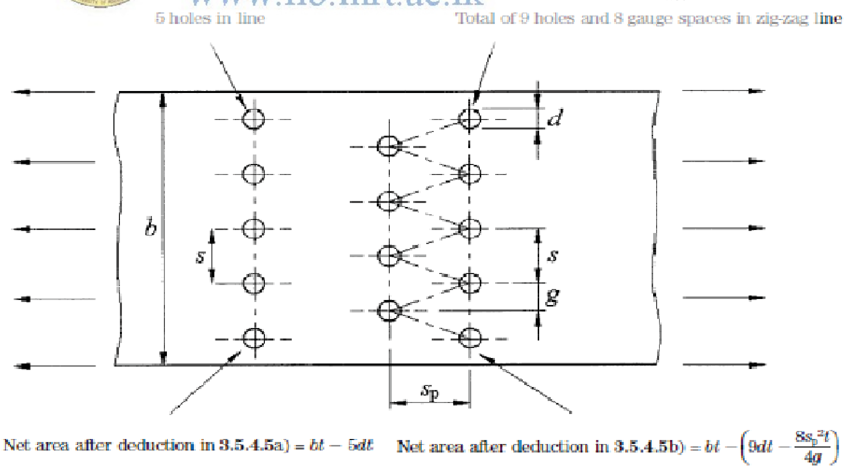


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.00 mm
s_p	-	20
g	-	60
A_{net}	-	352.64 mm^2

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -10.0m Span Truss-Bottom chord	BNC	2014-03-24	

a_1 - Net sectional area of connected leg

a_2 - A_g - a_1

	a_1	-	352.64 mm^2	
	A_e	-	1280.05 mm^2	
	a_2	-	1499.36	
Cl 4.6.3	Connnection Type	-	Bolt connection	
	Single Angle			
	- For bolt connection P_t	-	$p_y.(A_e\text{-}0.5a_2)$	
	- For weld connection P_t	-	$p_y.(A_g\text{-}0.3a_2)$	
	Double Angle			
	- For bolt connection P_t	-	$p_y.(A_e\text{-}0.25a_2)$	
	- For weld connection P_t	-	$p_y.(A_g\text{-}0.15a_2)$	
	P_t	-	248.93 kN	
			> 139.615 kN	OK

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -10.0m Span Truss-Bracing Member	BNC	2014-03-24

Cl 4.7

Design of Members

Sign convention	- Compression		-ve
	- Tension		+ve
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

Analysis Sumary

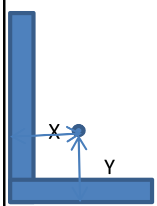
	Tension		Compression	
Member Assignment	2L2X2X1/4		2L2X2X1/4	
	Double Angle		Double Angle	
Frame Text	45		60	
Axial Forces	63.17	kN	-40.66	kN
M_{xx}	0.000	kNm	-0.146	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	2.700	m	2.453	m

Section Propeties

Cross Section Area	A	-	1219 mm^2
Second Moment of Inertia	I_{xx}	-	287600 mm^4
	I_{yy}	-	564700 mm^4
Plastic section Modulus	S_{xx}	-	14000 mm^3
	S_{yy}	-	18000 mm^3
Section Modulus	Z_{xx}	-	8008 mm^3
	Z_{yy}	-	11000 mm^3
Radius of gyration	R_{xx}	-	15.36 mm
	R_{yy}	-	21.52 mm

For single Angle / Double Angle only

	t_w	-	6.35 mm
	t_f	-	6.35 mm
	t_3	-	50.80 mm
	t_2	-	101.60 mm
	Y	-	8.3 mm
	X	-	0.00 mm
Product of Moment of Inertia	I_{xy}	-	0 mm^4
	$I_{max(u-u)}$	-	0 mm^4
	$I_{min(v-v)}$	-	0 mm^4
	R_{uu}	-	0.00 mm
	R_{vv}	-	0.00 mm



Cl 3.5.1

Member Loacal Buckling Check

$\frac{b}{t}$	-	7.75
$\frac{d}{t}$	-	8
ε	-	1

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -10.0m Span Truss-Bracing Member	BNC	2014-03-24

Table 11		15. ε		15	
				Section is Non Slender	
	Design of Compression Members				
Cl.4.7.10.2		λ	-	161.19	
	Design Strength of Steel	p_y	-	275 N/mm^2	
		λ_0	-	17.15	
		η	-	0.79	
		P_E	-	77.875 N/mm^2	
		ϕ	-	207.28	
		p_c	-	60.48 N/mm^2	
	Compression Resistance				
Cl.4.7.4		P_c	-	$A_g p_c$	
			-	66.35 kN > 40.656 kN	OK

Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

33.4 + 18.3 + 2.70E-14 - 51.63 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area - A_{net}

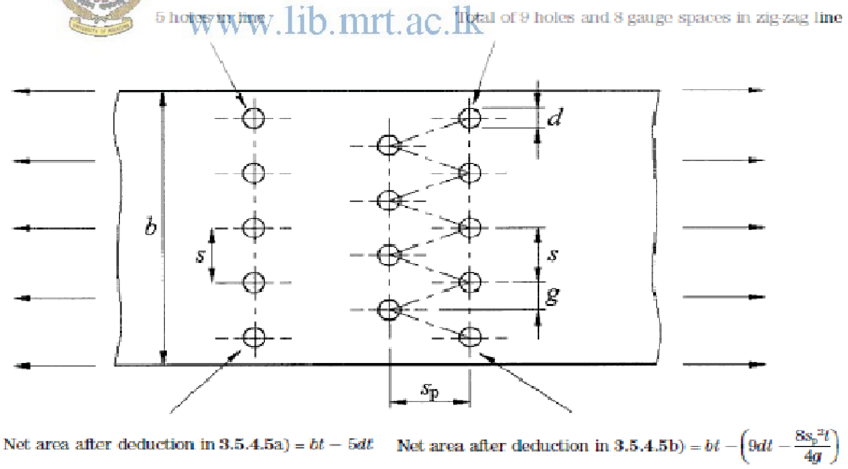


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.00 mm
s_p	-	20
g	-	60
A_{net}	-	191.35 mm^2

a_1 - Net sectional area of connected leg

a_2 - A_g - a_1

a_1	-	191.35	mm^2
A_e	-	796.18	mm^2
a_2	-	1027.65	

Cl 4.6.3

Connnection Type - Bolt connection

Single Angle

- For bolt connection P_t - $p_{y.}(A_e\text{-}0.5a_2)$
- For weld connection P_t - $p_{y.}(A_g\text{-}0.3a_2)$

Double Angle

- For bolt connection P_t - $p_{y.}(A_e\text{-}0.25a_2)$
- For weld connection P_t - $p_{y.}(A_g\text{-}0.15a_2)$

P_t - 148.30 kN

> 63.173 kN OK

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -10.0m Span Truss-Top chord	BNC	2014-03-24

Cl 4.7

Design of Compression Members

Sign convention	- Compression		-ve
	- Tension		+ve
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

Analysis Sumary

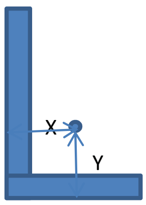
	Tension		Compression	
Member Assignment	2L3X3X1/4		2L3X3X1/4	
	Double Angle		Double Angle	
Frame Text	56		54	
Axial Forces	-95.28	kN	-158.55	kN
M_{xx}	-0.286	kNm	0.151	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	1.894	m	1.894	m

Section Propeties

Cross Section Area	A	-	1852 mm^2
Second Moment of Inertia	I_{xx}	-	1024000 mm^4
	I_{yy}	-	1883000 mm^4
Plastic section Modulus	S_{xx}	-	33000 mm^3
	S_{yy}	-	40000 mm^3
Section Modulus	Z_{xx}	-	19000 mm^3
	Z_{yy}	-	25000 mm^3
Radius of gyration	R_{xx}	-	23.51 mm
	R_{yy}	-	31.89 mm

For single Angle / Double Angle only

	t_w	-	6.35 mm
	t_f	-	6.35 mm
	t_3	-	76.20 mm
	t_2	-	152.40 mm
	Y	-	11.5 mm
	X	-	0.00 mm
Product of Moment of Inertia	I_{xy}	-	0 mm^4
	$I_{max(u-u)}$	-	0 mm^4
	$I_{min(v-v)}$	-	0 mm^4
	R_{uu}	-	0.00 mm
	R_{vv}	-	0.00 mm



Cl 3.5.1

Member Loacal Buckling Check

	$\frac{b}{t}$	-	11.75
	$\frac{d}{t}$	-	12
	ε	-	1

DESIGN CALCULATION	By	Date
PART : Hot Rolled Section -10.0m Span Truss-Top chord	BNC	2014-03-24

Table 11 — 15. ε — 15

Section is Non Slender

Design of Compression Members

Cl.4.7.10.2		λ	-	86.39
	Design Strength of Steel	p_y	-	275 N/mm^2
		λ_0	-	17.15
		η	-	0.38
		P_E	-	271.116 N/mm^2
		ϕ	-	324.68
		p_c	-	149.01 N/mm^2

Compression Resistance

Cl.4.7.4 — P_c - $A_g p_c$

- 248.38 kN > 158.554 kN OK

Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

85.6 + 7.94 + 2.30E-12 - 93.55 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area - A_{net}

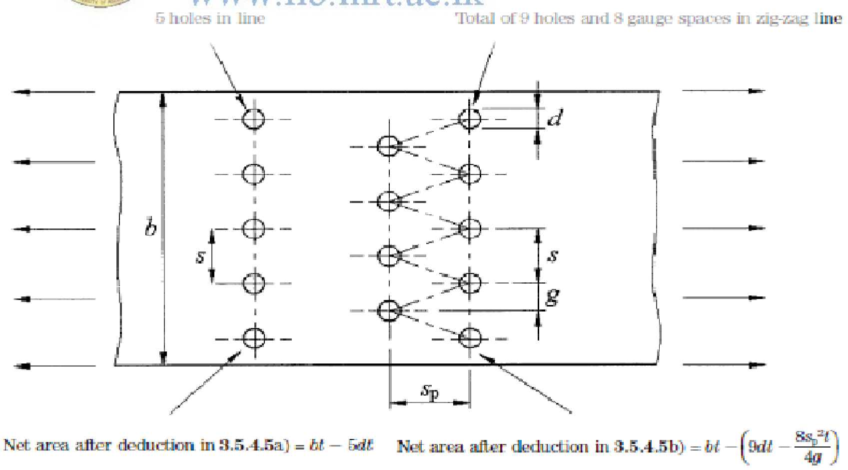


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes		-	3
Holes Pattern		-	Zig-Zag
Hole Diameter		-	8.00 mm
	s_p	-	20
	g	-	60
	A_{net}	-	352.64 mm^2

a_1 - Net sectional area of connected leg
a_2 - A_g - a_1

a_1	-	352.64	mm^2
A_e	-	1280.05	mm^2
a_2	-	1499.36	

Cl 4.6.3

Connnection Type - Bolt connection

Single Angle

- For bolt connection P_t - $p_y.(A_e-0.5a_2)$
- For weld connection P_t - $p_y.(A_g-0.3a_2)$

Double Angle

- For bolt connection P_t - $p_y.(A_e-0.25a_2)$
- For weld connection P_t - $p_y.(A_g-0.15a_2)$

P_t - 248.93 kN
$>$ 95.284 kN OK

DESIGN CALCULATION		By	Date
PART :	Hot Rolled Section -12.0m Span Truss-Bottom chord	BNC	2014-03-24

Cl 4.7

Design of Compression Members

Sign convention	- Compression		-ve
	- Tension		+ve
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

Analysis Sumary

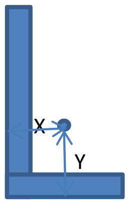
	Tension(Max.Forces)		Compression (Min Forces)	
Member Assignment	2L3X3X5/16 Double Angle		2L3X3X5/16 Double Angle	
Frame Text	687		682	
Axial Forces	155.82	kN	3.75	kN
M_{xx}	0.195	kNm	-0.303	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	1.010	m	1.010	m

Section Propeties

Cross Section Area	A	-	2290 mm^2
Second Moment of Inertia	I_{xx}	-	1257000 mm^4
	I_{yy}	-	2346000 mm^4
Plastic section Modulus	S_{xx}	-	42000 mm^3
	S_{yy}	-	50000 mm^3
Section Modulus	Z_{xx}	-	23000 mm^3
	Z_{yy}	-	31000 mm^3
Radius of gyration	R_{xx}	-	23.43 mm
	R_{yy}	-	32.01 mm

For single Angle / Double Angle only

	t_w	-	7.94 mm
	t_f	-	7.94 mm
	t_3	-	76.20 mm
	t_2	-	152.40 mm
	Y	-	12.1 mm
	X	-	0.00 mm
Product of Moment of Inertia	I_{xy}	-	0 mm^4
	$I_{max(u-u)}$	-	0 mm^4
	$I_{min(v-v)}$	-	0 mm^4
	R_{uu}	-	0.00 mm
	R_{vv}	-	0.00 mm



Cl 3.5.1

Member Loacal Buckling Check

$\frac{b}{t}$	-	9.3494
$\frac{d}{t}$	-	9.5994
ε	-	1

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -12.0m Span Truss-Bottom chord	BNC	2014-03-24

Table 11 15. ε 15

Section is Non Slender

Design of Compression Members

Cl.4.7.10.2		λ	-	60.17
	Design Strength of Steel	p_y	-	275 N/mm^2
		λ_0	-	17.15
		η	-	0.24
		P_E	-	558.82 N/mm^2
		ϕ	-	483.02
		p_c	-	200.83 N/mm^2

Compression Resistance

Cl.4.7.4 P_c - $A_g p_c$

\- 413.91 kN > 3.752 kN OK

Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

1.64 + 13.2 + 1.68E-13 - 14.81 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area A_{net}

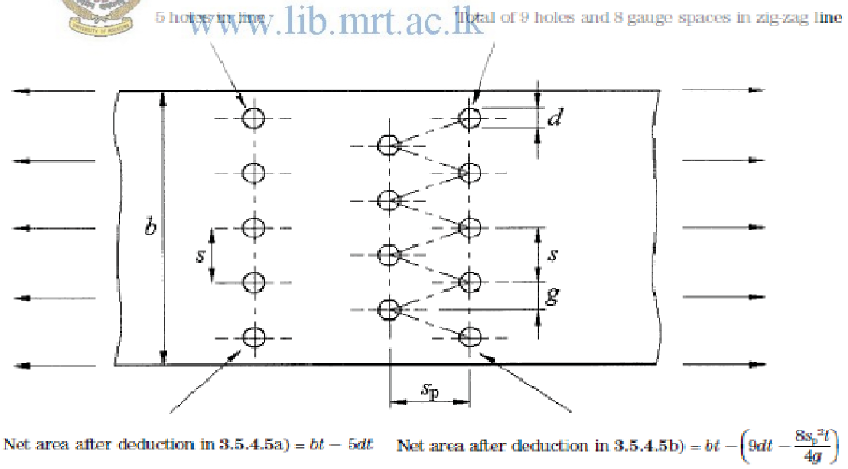


Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.0 mm
s_p	-	20
g	-	60
A_{net}	-	440.82 mm^2

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -12.0m Span Truss-Bottom chord	BNC	2014-03-24	

a_1 - Net sectional area of connected leg

a_2 - A_g - a_1

	a_1	-	440.82 mm^2	
	A_e	-	1587.56 mm^2	
	a_2	-	1849.18	
Cl 4.6.3	Connnection Type	-	Bolt connection	
	Single Angle			
	- For bolt connection P_t	-	$p_{y.}(A_e\text{-}0.5a_2)$	
	- For weld connection P_t	-	$p_{y.}(A_g\text{-}0.3a_2)$	
	Double Angle			
	- For bolt connection P_t	-	$p_{y.}(A_e\text{-}0.25a_2)$	
	- For weld connection P_t	-	$p_{y.}(A_g\text{-}0.15a_2)$	
	P_t	-	309.45 kN	
			> 155.817 kN	OK

	DESIGN CALCULATION	By	Date
	PART : Hot Rolled Section -12.0m Span Truss-Bracing Member	BNC	2014-03-24

Cl 4.7

Design of Compression Members

Sign convention	- Compression	-ve	
	- Tension	+ve	
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

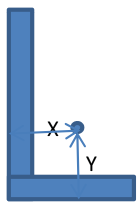
Analysis Sumary

	Tension		Compression	
Member Assignment	L3X2X1/4 Angle		L3X2X1/4 Angle	
Frame Text	681		679	
Axial Forces	48.59	kN	-47.60	kN
M_{xx}	0.217	kNm	0.011	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	1.032	m	1.032	m

Section Propeties

Cross Section Area	A	-	768	mm^2
Second Moment of Inertia	I_{xx}	-	453700	mm^4
	I_{yy}	-	163200	mm^4
Plastic section Modulus	S_{xx}	-	16000	mm^3
	S_{yy}	-	7669	mm^3
Section Modulus	Z_{xx}	-	8882	mm^3
	Z_{yy}	-	4261	mm^3
Radius of gyration	R_{xx}	-	24.31	mm
	R_{yy}	-	14.58	mm

For single Angle / Double Angle only



	t_w	-	6.35	mm
	t_f	-	6.35	mm
	t_3	-	76.20	mm
	t_2	-	50.80	mm
	Y	-	25.2	mm
	X	-	12.50	mm
Product of Moment of Inertia	I_{xy}	-	235762	mm^4
	$I_{max(u\text{-}u)}$	-	585364	mm^4
	$I_{min(v\text{-}v)}$	-	31536.4	mm^4
	R_{uu}	-	27.61	mm
	R_{vv}	-	6.41	mm

Cl 3.5.1

Member Loacal Buckling Check

$\frac{b}{t}$	-	8
$\frac{d}{t}$	-	12
ε	-	1

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -12.0m Span Truss-Bracing Member	BNC	2014-03-24	

Table 11 — 15. ε — 15

Section is Non Slender

Design of Compression Members

Cl.4.7.10.2		λ	-	136.89	
	Design Strength of Steel	p_y	-	275	N/mm^2
		λ_0	-	17.15	
		η	-	0.66	
		P_E	-	107.98	N/mm^2
		ϕ	-	227.04	
		p_c	-	79.21	N/mm^2

Compression Resistance

Cl.4.7.4	P_c	-	A_gp_c		
		-	54.75 kN	> 47.599 kN	OK

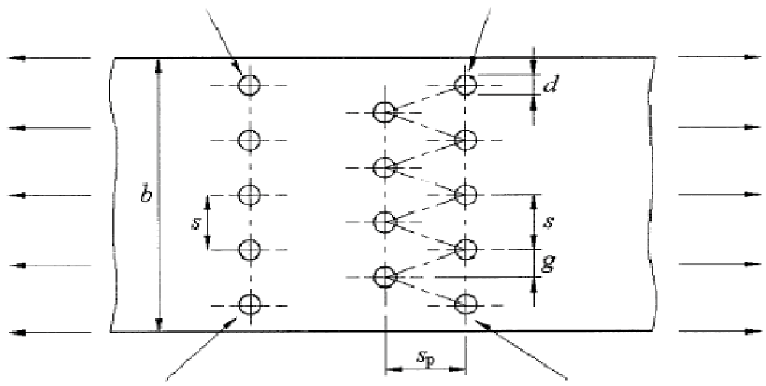
Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

62 + 1.26 + 1.37E-15 - 63.24 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area — A_{net}



Net area after deduction in 3.5.4.5a) = $bt - 5dt$ Net area after deduction in 3.5.4.5b) = $bt - \left(9dt - \frac{8s_p^2 t}{4g}\right)$

Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3	
Holes Pattern	-	Zig-Zag	
Hole Diameter	-	8.0 mm	
s_p	-	20	
g	-	60	
A_{net}	-	352.64	mm^2

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -12.0m Span Truss-Bracing Member	BNC	2014-03-24	

a_1 - Net sectional area of connected leg

a_2 - A_g - a_1

	a_1	-	352.64 mm^2
	A_e	-	634.89 mm^2
	a_2	-	415.36

Cl 4.6.3

Connnection Type - Bolt connection

Single Angle

- For bolt connection P_t - $p_{y.}(A_e\text{-}0.5a_2)$
- For weld connection P_t - $p_{y.}(A_g\text{-}0.3a_2)$

Double Angle

- For bolt connection P_t - $p_{y.}(A_e\text{-}0.25a_2)$
- For weld connection P_t - $p_{y.}(A_g\text{-}0.15a_2)$

P_t - 117.48 kN

> 48.59 kN OK

DESIGN CALCULATION		By	Date
PART :	Hot Rolled Section -12.0m Span Truss-Top chord	BNC	2014-03-24

Cl 4.7

Design of Compression Members

Sign convention	- Compression		-ve
	- Tension		+ve
Steel Grade	-	275	N/mm^2
E	-	205000	N/mm^2

Analysis Sumary

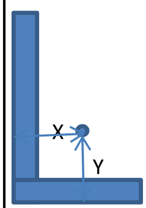
	Tension		Compression	
Member Assignment	2L3X3X5/16 Double Angle		2L3X3X5/16 Double Angle	
Frame Text	702		696	
Axial Forces	-1.47	kN	-154.40	kN
M_{xx}	-0.020	kNm	0.001	kNm
M_{yy}	0.000	kNm	0.000	kNm
Unrestrain Length	1.010	m	1.010	m

Section Propeties

Cross Section Area	A	-	2290	mm^2
Second Moment of Inertia	I_{xx}	-	1257000	mm^4
	I_{yy}	-	2346000	mm^4
Plastic section Modulus	S_{xx}	-	42000	mm^3
	S_{yy}	-	50000	mm^3
Section Modulus	Z_{xx}	-	23000	mm^3
	Z_{yy}	-	31000	mm^3
Radius of gyration	R_{xx}	-	23.43	mm
	R_{yy}	-	32.01	mm

For single Angle / Double Angle only

	t_w	-	7.94	mm
	t_f	-	7.94	mm
	t_3	-	76.20	mm
	t_2	-	152.40	mm
	Y	-	12.1	mm
	X	-	0.00	mm
Product of Moment of Inertia	I_{xy}	-	0	mm^4
	$I_{max(u-u)}$	-	0	mm^4
	$I_{min(v-v)}$	-	0	mm^4
	R_{uu}	-	0.00	mm
	R_{vv}	-	0.00	mm



Cl 3.5.1

Member Loacal Buckling Check

$\frac{b}{t}$	-	9.3494
$\frac{d}{t}$	-	9.5994
ε	-	1

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -12.0m Span Truss-Top chord	BNC	2014-03-24	

Table 11 15. ε 15

Section is Non Slender

Design of Compression Members

Cl.4.7.10.2		λ	-	60.17		
	Design Strength of Steel	p_y	-	275	N/mm^2	
		λ_0	-	17.15		
		η	-	0.24		
		P_E	-	558.82	N/mm^2	
		ϕ	-	483.02		
		p_c	-	200.83	N/mm^2	

Compression Resistance

Cl.4.7.4 P_c - $A_g p_c$

\- 413.91 kN > 154.401 kN OK

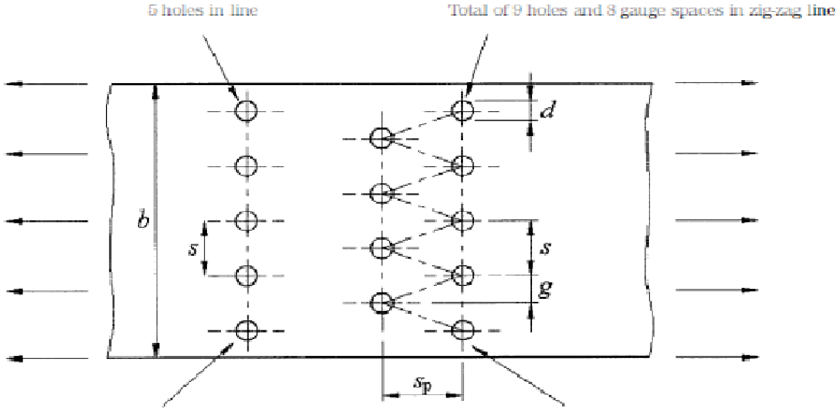
Annex: I.3

$$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq p_y$$

67.4 + 0.06 + 1.54E-12 - 67.48 < 275 kN/mm2 OK

Cl 3.4.1 **Design of Tension Members**

Net corss sectional area - A_{net}



Net area after deduction in 3.5.4.5a) = $bt - 5dt$ Net area after deduction in 3.5.4.5b) = $bt - \left(9dt - \frac{8s_p^2 t}{4g}\right)$

Figure 1 — Nomenclature for staggered holes with example

Number of Bolt Holes	-	3
Holes Pattern	-	Zig-Zag
Hole Diameter	-	8.0 mm
s_p	-	20
g	-	60
A_{net}	-	440.82 mm^2

	DESIGN CALCULATION	By	Date	
	PART : Hot Rolled Section -12.0m Span Truss-Top chord	BNC	2014-03-24	

a_1 - Net sectional area of connected leg

a_2 - A_g - a_1

	a_1	-	440.82 mm^2	
	A_e	-	1587.56 mm^2	
	a_2	-	1849.18	
Cl 4.6.3	Connnection Type	-	Bolt connection	
	Single Angle			
	- For bolt connection P_t	-	$p_{y.}(A_e\text{-}0.5a_2)$	
	- For weld connection P_t	-	$p_{y.}(A_g\text{-}0.3a_2)$	
	Double Angle			
	- For bolt connection P_t	-	$p_{y.}(A_e\text{-}0.25a_2)$	
	- For weld connection P_t	-	$p_{y.}(A_g\text{-}0.15a_2)$	
	P_t	-	309.45 kN	
			> 1.466 kN	OK

ANNEX : D - COST EVALUATION

ROOF COVERING - 12m Span Roof Structure

No.	Description	Qty.	Unit	Rate LKR	Amount LKR
1.0	**Zinc Alum Roofing**				
1.1	**Note:**				
1.1.1	Colour Bonded High Tensile Steel Roofing sheets, Type Glament G5 A40 Sheet with a total coated of 0.47mm.			(Note)	
1.1.2	Rate shall include for supplying and install complete on site with all accessories, fixings, ridging, valley gutters and flashings for water tightness all complete as per drawings in total conformity with reputed manufacturer's recommendation such as BHP Steel (Blue Scope) (Australia) suited to a building exposed to marine climate and a wind speed of up to 85 miles per hour.			(Note)	
1.1.3	The following requirements are to be noted and the rates shall cover same.			(Note)	
1.1.4	Fine Stable and uniformly extruded single layer closed cell polyethylene foam with a thermal conductivity not more than 0.038 w/k, both sides laminated with pure aluminium foil. Use heat guard roofing foil 8mm thick or approved equivalent.			(Note)	
1.1.5	Sheets to be of non colour			(Note)	
1.1.6	Steel roof framework & lipped channels provided separately.			(Note)	

ROOF COVERING - 12m Span Roof Structure

No.	Description	Qty.	Unit	Rate LKR	Amount LKR
1.2	**<u>Roof Covering</u>**				
1.2.1	Supply and lay colour bonded high tensile glamet G5 A40 roofing sheets(.35mm tk.) with and including roof insulation and all other items completely as specified.	291.00	m^2	3,500.00	1,018,500.00
	Bill No. 1 - Roof Covering & Insulation **Total Carried to Summary**				1,018,500.00

STRUCTURAL STEEL WORK - 12m Span Roof Structure (CFS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
2.0	**Structural Steel Work**				
2.1	**Notes**				
2.1.1	The Bidder shall refer the following prior to pricing the items in this trade a. All relevant drawings b. Specifications c. Locations of steel work d. Access to locations of installation e. Method of transportation to locations f. Method of handing & installation.			(Note)	
2.1.2	Rates shall include for all required plant and machinery required for hoisting and erecting.			(Note)	
2.1.3	Rates shall include for all required labour for fabrication, transportation, erection and installation.			(Note)	
2.1.4	Rates for structural steel work shall include for connected steel fixtures such as plates, bolts, nuts, cleats, haunches, etc (please refer method of payment)			(Note)	
2.1.5	Where members of steel structures are fastened together by means of reverts, bolts or by welding and all such connection shall be finished neatly.			(Note)	
	Welding				
2.1.6	Welding will be of good clean metal deposited by procedure which will ensure uniformity and continuity of work. The surface of the weld will have an even contour and regular finish and will indicate proper fusion with parent metal. All slag shall be removed making each run by light hammering followed by wire brushing.			(Note)	
2.1.7	Weld metal shall not be allowed to spatter on surfaces that are visible in the final works.			(Note)	
2.1.8	Butt welds which will be visible in the completed works should be dressed off smooth and flushed with adjacent surface.			(Note)	

STRUCTURAL STEEL WORK - 12m Span Roof Structure (CFS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
	Fabrication				
2.1.9	All materials shall be straight and if necessary before being worked should be straightened or flattened by pressure unless required to be curvilinear form and should be free from twist.			(Note)	
	Erection				
2.1.10	The suitability and capacity of all plant and equipment used for erection should be to the satisfaction of the Consultant.			(Note)	
2.1.11	All structural steel should be stored and handled at the site in such a way that the members are not subjected to excessive stresses and damage.			(Note)	
2.1.12	The positioning and level of the steel work, the plumb of stanchions and the placing of every part of structure with accuracy should be in accordance with the approved drawings/shop drawings to the satisfaction of the Consultant.			(Note)	
2.1.13	During erection the steel work should be securely bolted or otherwise fastened, and when necessary temporarily braced to provide for all load to be carried by the structure during erection including those due to erection equipment and its position.			(Note)	
2.1.14	No reverting, permanent bolting or welding should be done until proper alignment has been obtained.			(Note)	
2.1.15	The Final Installation to be painted with 2 coats zinc rich primer and 2 coats of enamel paint of an approved quality and to a colour to be specified by the Consultant.			(Note)	
2.1.16	Where requested the Contractor shall provide all necessary shop drawings to the approval of the Engineer.			(Note)	

STRUCTURAL STEEL WORK - 12m Span Roof Structure (CFS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
	Method of Payment				
2.1.17	Where the Unit Rate is given in the Bill of Quantities is in t (imperial ton) only the weight of the Main Steel sections will be considered for payment, unless otherwise provided separately in the BOQ. As such the rate shall include for all other fixtures as defined in the previous paragraphs and for welding, transporting, assembling, erecting, painting, etc.			(Note)	
2.1.18	The quoted price shall include for all the above referred items/specifications and no additional payments will be entertained.			(Note)	
2.2	**Structural Steel Work**				
2.2.1	Steel Top Member,C200mm x 75mm x 20mm (6.18kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	0.750	t	375,000.00	281,250.00
2.2.2	Steel Bottom Member,C200mm x 75mm x 20mm (6.18kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	0.690	t	375,000.00	258,750.00
2.2.3	Steel Bracing,C200mm x 75mm x 20mm (6.18kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	0.770	t	375,000.00	288,750.00
2.2.4	Galvanized "C" perlins,type C100-16 fixed in position as specified.	312.00	m	3,500.00	1,092,000.00
	Bill No. 2 - Structural Steel Work **Total Carried to Summary**				1,920,750.00

GRAND SUMMARY - 12m Span Roof Structure (CFS)

Item No.	Description	Amount LKR
1.0	ROOF COVERING & INSULATION	1,018,500.00
2.0	STRUCTURAL STEEL WORK	1,920,750.00
3.0	**TOTAL ESTIMATED COST**	2,939,250.00
4.0	(VAT not included)	

Date of Estimation - April 2013

STRUCTURAL STEEL WORK - 12m Span Roof Structure (HRS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
2.0	**Structural Steel Work**				
2.1	**Notes**				
2.1.1	The Bidder shall refer the following prior to pricing the items in this trade a. All relevant drawings b. Specifications c. Locations of steel work d. Access to locations of installation e. Method of transportation to locations f. Method of handing & installation.			(Note)	
2.1.2	Rates shall include for all required plant and machinery required for hoisting and erecting.			(Note)	
2.1.3	Rates shall include for all required labour for fabrication, transportation, erection and installation.			(Note)	
2.1.4	Rates for structural steel work shall include for connected steel fixtures such as plates, bolts, nuts, cleats, haunches, etc (please refer method of payment)			(Note)	
2.1.5	Where members of steel structures are fastened together by means of reverts, bolts or by welding and all such connection shall be finished neatly.			(Note)	
	Welding				
2.1.6	Welding will be of good clean metal deposited by procedure which will ensure uniformity and continuity of work. The surface of the weld will have an even contour and regular finish and will indicate proper fusion with parent metal. All slag shall be removed making each run by light hammering followed by wire brushing.			(Note)	
2.1.7	Weld metal shall not be allowed to spatter on surfaces that are visible in the final works.			(Note)	
2.1.8	Butt welds which will be visible in the completed works should be dressed off smooth and flushed with adjacent surface.			(Note)	

STRUCTURAL STEEL WORK - 12m Span Roof Structure (HRS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
	Fabrication				
2.1.9	All materials shall be straight and if necessary before being worked should be straightened or flattened by pressure unless required to be curvilinear form and should be free from twist.			(Note)	
	Erection				
2.1.10	The suitability and capacity of all plant and equipment used for erection should be to the satisfaction of the Consultant.			(Note)	
2.1.11	All structural steel should be stored and handled at the site in such a way that the members are not subjected to excessive stresses and damage.			(Note)	
2.1.12	The positioning and level of the steel work, the plumb of stanchions and the placing of every part of structure with accuracy should be in accordance with the approved drawings/shop drawings to the satisfaction of the Consultant.			(Note)	
2.1.13	During erection the steel work should be securely bolted or otherwise fastened, and when necessary temporarily braced to provide for all load to be carried by the structure during erection including those due to erection equipment and its position.			(Note)	
2.1.14	No reverting, permanent bolting or welding should be done until proper alignment has been obtained.			(Note)	
2.1.15	The Final Installation to be painted with 2 coats zinc rich primer and 2 coats of enamel paint of an approved quality and to a colour to be specified by the Consultant.			(Note)	
2.1.16	Where requested the Contractor shall provide all necessary shop drawings to the approval of the Engineer.			(Note)	

STRUCTURAL STEEL WORK - 12m Span Roof Structure (HRS)

Item No.	Description	Qty.	Unit	Rate LKR	Amount LKR
	Method of Payment				
2.1.17	Where the Unit Rate is given in the Bill of Quantities is in t (imperial ton) only the weight of the Main Steel sections will be considered for payment, unless otherwise provided separately in the BOQ. As such the rate shall include for all other fixtures as defined in the previous paragraphs and for welding, transporting, assembling, erecting, painting, etc.			(Note)	
2.1.18	The quoted price shall include for all the above referred items/specifications and no additional payments will be entertained.			(Note)	
2.2	**Structural Steel Work**				
2.2.1	Steel Top Member,75mm x75mm x 8mm angles (9.03kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	1.100	t	375,000.00	412,500.00
2.2.2	Steel Bottom Member,75mm x 75mm x 8mm angles (9.03kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	1.000	t	375,000.00	375,000.00
2.2.3	Steel Bracing,75mm x 50mm x 6mm angles (5.65kg/m) fixed in position , as specified including all items as defined in the notes to the trade bill.	0.700	t	375,000.00	262,500.00
2.2.4	Galvanized "C" perlins,type C150-16 fixed in position as specified.	312.00	m	3,500.00	1,092,000.00
	Bill No. 2 - Structural Steel Work **Total Carried to Summary**				2,142,000.00

GRAND SUMMARY

Item No.	Description	Amount LKR
1.0	ROOF COVERING & INSULATION	1,018,500.00
2.0	STRUCTURAL STEEL WORK	2,142,000.00
3.0	**TOTAL ESTIMATED COST**	3,160,500.00
4.0	(VAT not included)	

Date of Estimation - April 2013