# FEASIBILITY OF USING COLD FORMED STEEL FOR

# MEDIUM SPAN ROOF STRUCTURES IN SRI LANKA



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.

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#### **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. When other apperior steel that is cold formed sections their structural solution. When other apperior steel that is cold formed sections their structural 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 8.0 m to 10.0 m could gain a saving of  $23 \% \sim 25 \%$  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.



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# **NOTATION**

A	Area or Gross area of a cross-section
$A_e$	Effective net area of a section
$A_{\it eff}$	Effective area
$A_n$	Net area of a section
$A_{st}$	Area of an intermediate stiffener
$A_t$	Tensile stress area of a bolt
а	Effective throat size of a fillet weld
$a_1$	University of Moratuwa, Sri Lanka.  Net sectional engagements continue technique techn
$a_2$	Gross sectional area of unconnected elements
В	Overall width of an element
$B_f$	Half the overall flange width of an element
b	Flat width of an element
$b_{\it eff}$	Effective width of a compression element
$b_{\it er}$	Reduced effective width of a sub-element
$b_{eu}$	Effective width of an unstiffened compression element
$C_W$	Warping constant of a section

С	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	University of Moratuwa, Sri Lanka.  University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations
I	www.lib.mrt.ac.lk 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
$V$ $V_s$	design wind speed
	design wind speed University of Moratuwa, Sri Lanka.  The Electronic Theses & Dissertations
$V_s$	design wind speed University of Moratuwa, Sri Lanka.
$V_s$ h	design wind speed University of Moratuwa, Sri Lanka.  Flectronic Theses & Dissertations www.lib.mrt.ac.lk
$V_s$ $h$	design wind speed University of Moratuwa, Sri Lanka.  Flectronic Theses & Dissertations www.lib.mrt.ac.lk  width of building
$V_s$ $h$ $w$	design wind speed University of Moratuwa, Sri Lanka.  Flectronic Theses & Dissertations www.lib.mrt.ac.lk  width of building  dynamic pressure of wind (stagnation pressure)

#### **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
  - o Lightness
  - High strength and stiffness
  - o Fast and easy erection and installation
  - o More accurate detailing
  - o 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



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 University of Moratuwa, Sri Lanka. 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 hising cold formed 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 of Moratuwa, Sri Lanka.
- III. Analyzed the trass for permanent and wind load using finite element design software, SAP2000 lab full sadd 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  $T < -34^{\circ}C$  &  $T > 93^{\circ}C$  (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, Sri Lanka.
- Module of Elasticity leses & Dissertations
- Ductilityw.lib.mrt.ac.lk
- 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.

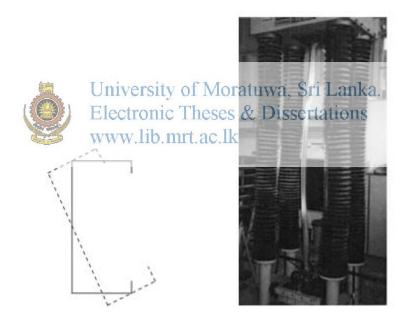


Figure 2-1 Flexural and Flexural-torsional Buckling (Young, 2005)

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  $\beta$  is 0.3 by his study.

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

Where;  $\Delta$  - magnitude of initial imperfection.

*t* - thickness of the plate

 $\beta$  - constant that can be adjusted to fit experimental results.

 $p_{\nu}$  - 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.



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.

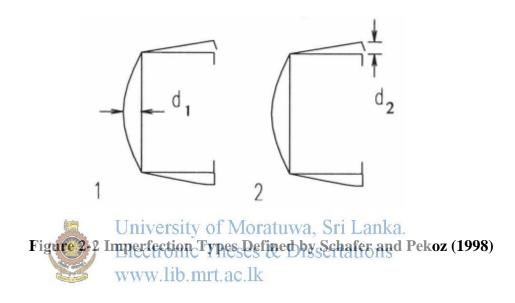
 $\Delta = 0.006w$  or  $\Delta = 6te^{-2t}$  for stiffened elements

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

 $\Delta$  - imperfection

w - width of the stiffened elements

*t* - plate thickness.



#### 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)

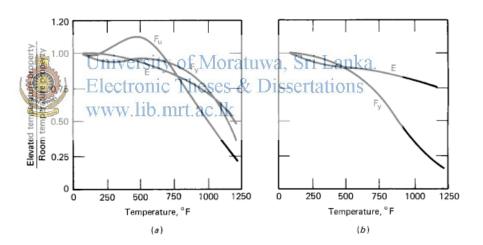


Figure 2-3 Effect of temperature on mechanical properties of low carbon steel

(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°C. 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	а	b	С	n
For yield stress,	22 ≤ T < 300	1.0	22	5.56x10 <sup>3</sup>	1
$\frac{f_{y,T}}{f_{y,normal}} = a - \frac{(T-b)^n}{c}$	300 ≤ T < 650	0.95	300	1.45x10 <sup>5</sup>	2
$f_{y,normal} = a - \frac{1}{c}$	650 ≤ T < 1000	0.105	650	5.00x10 <sup>3</sup>	1
For elastic modulus,	22 ≤ T < 450	1.0	22	1.25x10 <sup>3</sup>	1
$\frac{E_T}{T_{normal}} = a - \frac{(T - b)^n}{c}$	$450 \le T < 650$	-0.11	860	-2.20x105	2
ASONESO 7	trensity of Mora trensity of Mora v.lib.mrt.ac.lk	& Disse	rtation	S <sup>5</sup> .6x10 <sup>8</sup>	3
$\frac{f_{u,T}}{f_{u,normal}} = a - \frac{(T-b)^n}{c}$	450 ≤ T < 1000	0.043	1000	-1.12x10 <sup>11</sup>	4
For ultimate strain,					
$\frac{\mathcal{E}_{u,T}}{\mathcal{E}_{u,normal}} = a - \frac{(T-b)^n}{c}$	22 ≤ T < 1000	1.0	22	1.0x10 <sup>6</sup>	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 ileading to Irapid 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 smore than \$50min\$ when the load ratio was less than \$0.65. We 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)

University of Moratuwa, Sri Lanka.

Strength and Stiffness properties Theoretic 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.

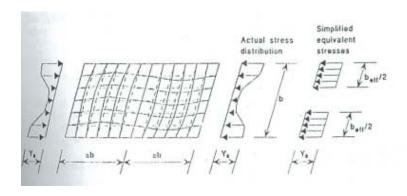


Figure 2-4 Illustration of effective width of a compression plate (SCI, 1993)

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

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

(2a)

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$$\frac{e^{ef}}{V_p}$$
 Dissertations  $\frac{1}{P_{cr}}$  (2b)

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Where  $f_c$  - compression stress of the element.

Per - 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.

$$P_{C} = \frac{P_{E}P_{CS}}{\phi + \sqrt{\phi^{2} - P_{E}P_{CS}}}$$
 (2.1a)

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

$$P_{CS} = A_{eff} P_{\gamma} \tag{2.1c}$$

 $A_{eff}$  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<sub>e</sub> is the effective length of the member about the critical axis University of Moratuwa, Sri Lanka.

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(2.1e)

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

$$P_c' = \frac{M_C P_C}{(M_C + P_C e_s)}$$
 (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

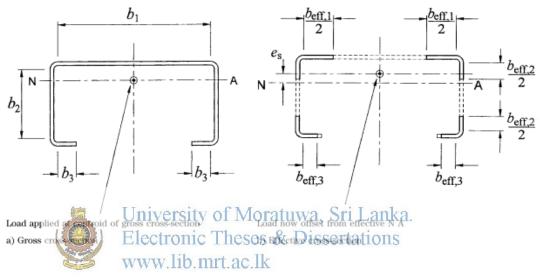


Figure 2-5 Compression of singly symmetrical section (BSI, [1998])

And,

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

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

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

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

$$P_{T} = \frac{1}{r_{0}^{2}} \left( GJ + 2 \frac{\pi^{2} E C_{w}}{L_{E}^{2}} \right)$$
 (2.2b)

$$\beta$$
 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.

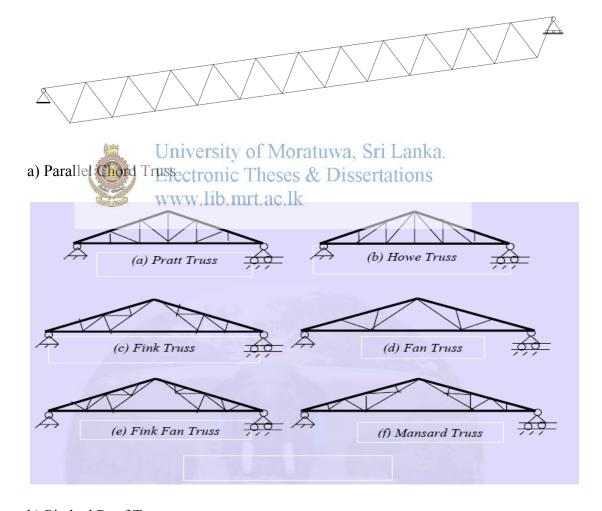
Ix is be second moment of area about the x-axis, www.lib.mrt.ac.lk

C<sub>w</sub> 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.



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 SAR 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	Linear	No	Linear Static	DEAD	1.0
(Service Dead-	Add		Linear Static	LIVE	1.0
Live Load)			Linear Static	SUPER DEAD	1.0
UDL	Linear	No	Linear Static	DEAD	1.4
Ultimate Dead-	Add		Linear Static	SUPER DEAD	1.4
Live load)			Linear Static	LIVE	1.6
ULDW(max)	Linear	No	Linear Static	DEAD	1.2
(Ultimate	Add		Linear Static	LIVE	1.2
Dead-Live-			Linear Static	SUPER DEAD	1.2
Wind [Inward] load)			Linear Static	Wind(max)	1.2
UDLW(min)	Linear	No	Linear Static	DEAD	1.2
(Ultimate	Add		Linear Static	LIVE	1.2
Dead-Live-			Linear Static	SUPER DEAD	1.2
Wind [outward] [oad)		A STATE OF THE STA	oralearstationi es & Dissert	\ /	1.2
ENV	Envelope;			DEAD	1.0
	77 77 77		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(+)
- University of Moratuwa, Sri Lanka.

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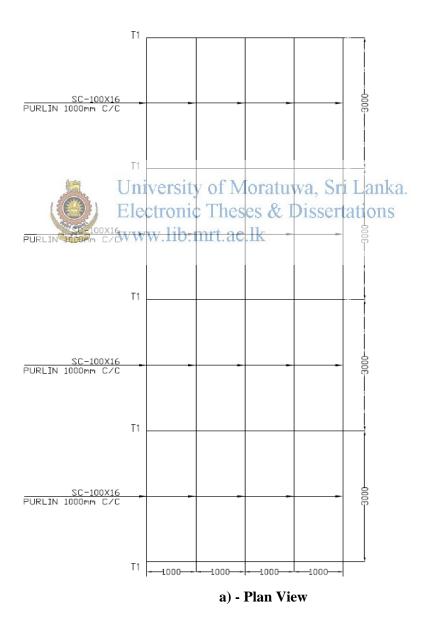
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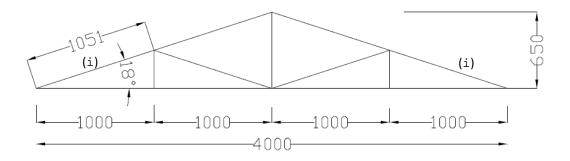
The general geometry of 4.0m span structure is shown in *Figure 4-1* below. Exactly same geometry was implemented on both CFS & HRS roof structures. The bay spacing for roof trusses were 3.0m according to column grid of the building and four bays were included in 3D analysis.

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.

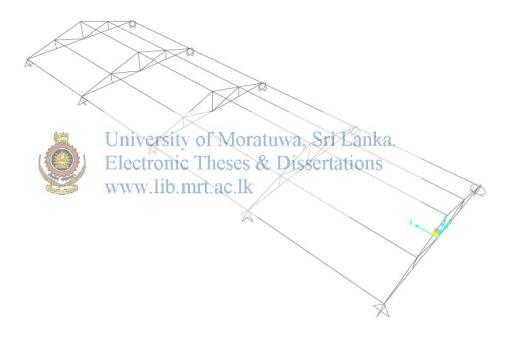
Table 4-1: Optimum Design section for 4.0m Span Roof Truss

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			





## 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)				
	Eleme <b>nt</b>	Cold formed (C) versity of Moratuv	Hot Rolled (H) wa, Sri Lanka.	<b>Ratio</b> C/H		
ssn	0000	v.lib.mrt.ac.ll <sub>42.5</sub>	15.1	0.83		
Outer most truss	17	12.4	14.9	0.83		
ter m	18	12.4	14.9	0.83		
no	19	12.5	15.1	0.83		
nss	23	21.1	24.8	0.85		
1 <sup>st</sup> interior truss	24	21	24.5	0.86		
inter	25	21	24.5	0.86		
13	26	21.1	24.8	0.85		
interior truss	62	21.1	24.8	0.85		
inté	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

	Eleme	Element Forces - Frames -Top chord (Combo - ENV)					
		Minimum Force	(Compression)				
	Element Label	Cold formed (C)	Hot Rolled (H)	Ratio			
	Univers	lty of Moratuwa	Sri Lanka.	C/H			
SSN	2	ic Theses & 1D5s		0.84			
ost tr	12ww.lil	.mrt.ac.lk <sub>-10.8</sub>	-11.3	0.96			
Outer most truss	13	-10.8	-11.3	0.96			
no	14	-13.5	-16.1	0.84			
SSI	15	-22.7	-26.4	0.86			
ior tru	20	-18	-18.3	0.98			
1 <sup>st</sup> interior truss	21	-18	-18.3	0.98			
1st	22	-22.7	-26.4	0.86			
interior truss	58	-22.7	-26.4	0.86			
inte trı	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.

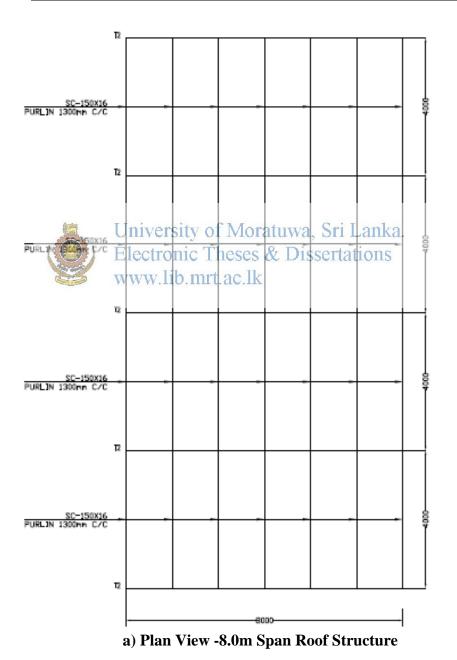
Design section and analysis sections were verified that they converge on each other after few iteration between design & analysis. The following Table 4-4& Table 4-5, shows the selected optimum design sections for both CFS and HRS roof structure.

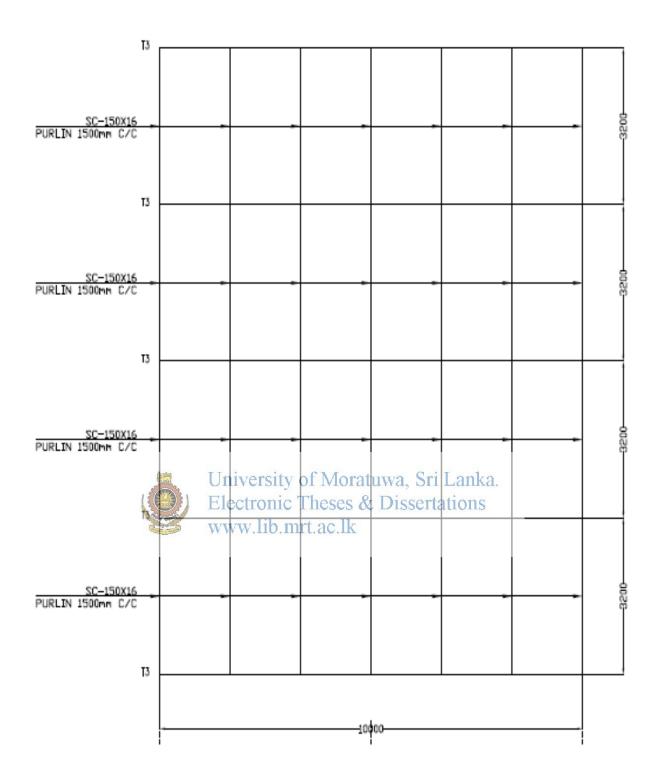
Table 4-4: Optimum Design section for 8.0m Span Roof Truss

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			

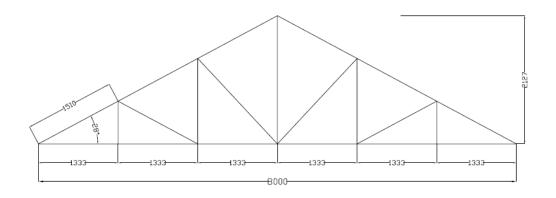
Table 4-5: Optimum Design section for 10.0m Span Roof Truss

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)			



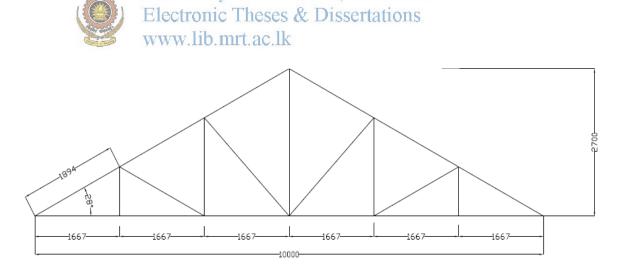


b) Plan View -10.0m Span Roof Structure



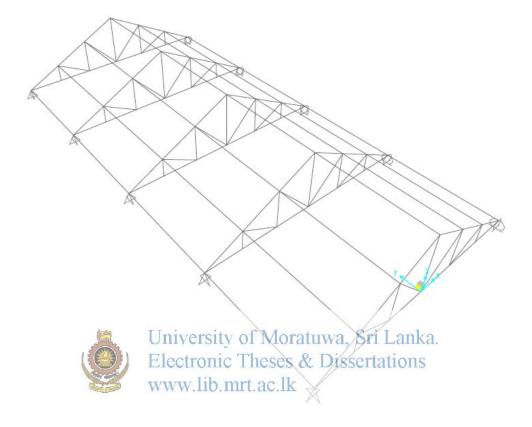
# c) Sectional View - 8.0m Span roof Structure

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d) Sectional View - 10.0m Span roof Structure

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#### 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)								
		8.0	8.0m (Tension)			<b>10.0m</b> (Tension)		
	Element Label	Cold formed	Hot Rolled	Ratio	Cold formed	Hot Rolled	Ratio	
	I Ia	kN	kN	C/H	kN	kN	C/H	
	1	28	32.4	0.86	29.3	34.9	0.84	
russ	2	28.2	32.3	0.87	29.4	34.8	0.84	
Outer most truss	3	22.1	24.6	0.90	22.7	26.4	0.86	
ter m	4	22.1	24.6	0.90	22.7	26.4	0.86	
Out	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	
	7	44.8	49.7	0.90	44.8	48.6	0.92	
russ	8	Univers Univers	ity 95.3V	loratuwa	a, <b>S</b> ri <sub>4</sub> I <sub>9</sub> a	nka <sub>48.5</sub>	0.93	
1st interior truss	9	Electron ww36.1ii	110 Thes	es & D1	35.4	ons 37.4	0.95	
inte	10	36.1	38.2	0.95	35.4	37.4	0.95	
1st	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	
truss	13	41	46.1	0.89	42.9	47.9	0.90	
Interior truss	14	41.4	45.9	0.90	43	47.9	0.90	
Inte	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

Memb	Member Forces - Frames -Top chord-(COMBO-ENV)								
		8.0n	n (compression	on)	10.0n	ı (compress	ion)		
	Element Label	Cold formed	Hot Rolled	Ratio	Cold formed	Hot Rolled	Ratio		
	EL	kN	kN	C/H	kN	kN	C/H		
	1	-33	-37.8	0.87	-34.6	-40.8	0.85		
russ	2	-28.7	-31.4	0.91	-29.5	-33.8	0.87		
Outer most truss	3	-23.2	-25.1	0.92	-23.8	-26.9	0.88		
ter m	4	-23.2	-25	0.93	-23.7	-26.8	0.88		
Out	5	Unive	rsity of M	oratuwa,	Sri Lank	a33.6	0.88		
	6	Electr -33 WWW	onic These	es & Diss 1k 0.88	-34.6	-40.6	0.85		
	7	-52.1	-57.5	0.91	-52.2	-56.4	0.93		
russ	8	-44.7	-47.2	0.95	-44.1	-46.3	0.95		
1st interior truss	9	-35.4	-37	0.96	-34.8	-36.3	0.96		
inte	10	-35.4	-36.8	0.96	-34.7	-36.2	0.96		
1st	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		
russ	13	-47.7	-53.3	0.89	-50.1	-55.7	0.90		
Interior truss	14	-41.1	-44	0.93	-42.4	-45.7	0.93		
Inte	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 ideflections, wibration, 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 114.0m, 8.0m & 10.0m Span Roof Structures Rurlin continuous over trusses

	www.lib.mrt.ac.lk							
TABLE: Vertical Deflection at the Mid Span of truss								
		4.0m Span Roof		8.0m Span Roof		10.0m Span Roof		
		Tru	uss	Truss		Tru	uss	
Truss	Output	CFS	HRS	CFS	HRS	CFS	HRS	
Location	Case	(Z-Axis)	(Z-Axis)	(Z-Axis)	(Z-Axis)	(Z-Axis)	(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	

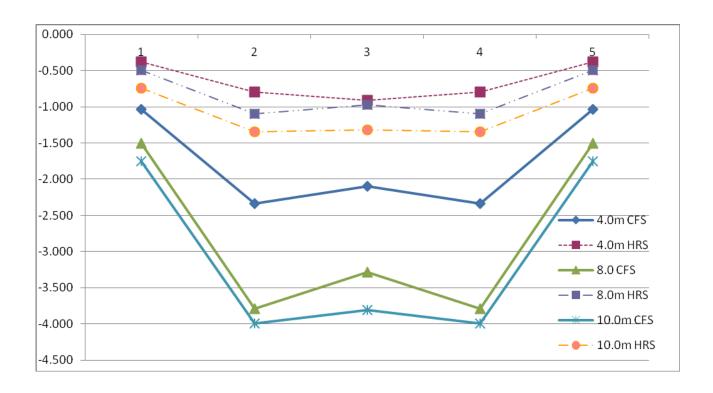


Figure 4-3: Variation of Deflection at Mid Span of Trusses

Table 4-10. Figure 44 below shows the vertical deflection of each truss for both steel families under the condition of purlins are discontinuous at each truss locations. www.lib.mrt.ac.lk

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

TABLE: Vert	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	

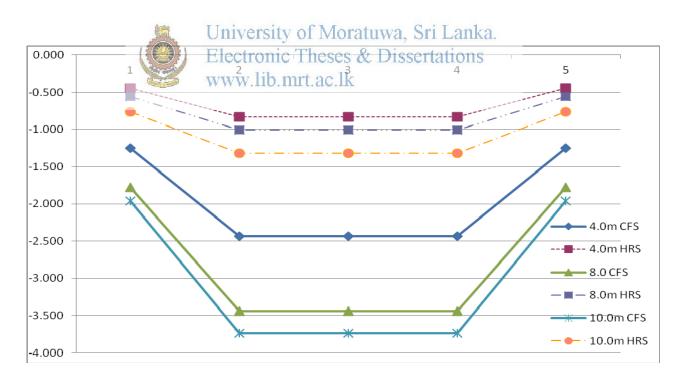


Figure 4-4: Variation of Deflection at Mid Span of Trusses

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).

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

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 oven in Fable 418, Turber comparison will be scarried out for 12.0m span roof systems, inversed by the ability requirement.

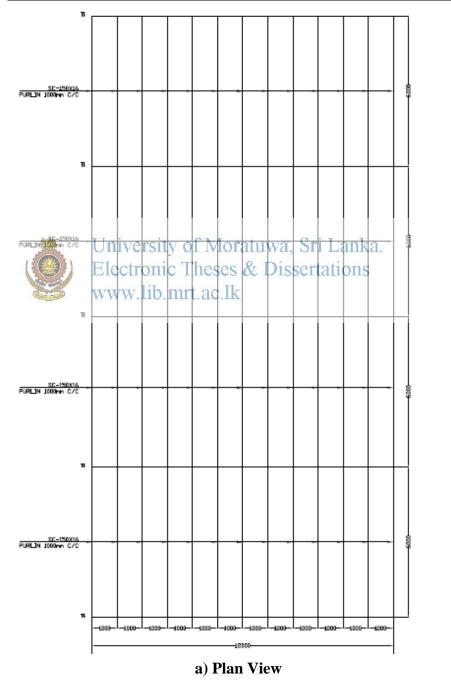
#### 4.1.4 12.0m SPAN ROOF STRUCTURE

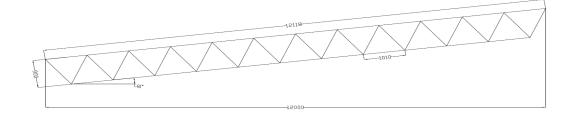
The general sectional Geometry of 12.0m span truss, for both CFS & HRS is shown in Figure 4-5 below. The bay spacing of the trusses were govern by the column grid of the building and was set as 6.0m.

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			
Top Member	2//3x/3x611111	(Back to Back-Double section)			
Bottom	2/75x75x8mm	C 200x75x20mm			
Member	2//5x/5x811111	(Back to Back-Double section)			
Bracin	75v50v6mm	C 200v7Ev20mm			
Member	75x50x6mm	C 200x75x20mm			





## 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 Fo	rce (Tension)				
	ent	Cold formed	Hot Rolled	Ratio			
	Element Label						
		KN	KN	C/H			
	25	23.3	25.2	0.92			
	26	42.3	45.8	0.92			
	27	57.2	61.8	0.93			
russ	28	67.9	73.3	0.93			
Outer most truss	29	74.3	80.2	0.93			
шо	30	76.5	82.5	0.93			
	31	74.3	80.2	0.93			
ŏ	32	67.9	73.3	0.93			
	33	57.3	61.9	0.93			
	Uni <sup>34</sup> ersity	of Moratifwa	Sri Lank <sup>45.8</sup>	0.93			
	Electronic	Theses & 23,4s	sertations <sup>25.3</sup>	0.92			
disperie	635 lih m	rt ac 1k 53.7	57.1	0.94			
	636	97.8	103.9	0.94			
	637	132.2	140.3	0.94			
ssn	638	156.9	166.3	0.94			
or tri	639	171.6	181.9	0.94			
1st interior truss	640	176.6	187.1	0.94			
t int	641	171.7	181.9	0.94			
15	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			
	682	44.6	47.5	0.94			
SSr	683	81.2	86.4	0.94			
r tru	684	109.9	116.8	0.94			
nterior truss	685	130.4	138.4	0.94			
Into	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.

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The Table Table variation of member forces at top chord of two families.

The force distribution pattern in the 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)				
	ent Jel	Caldfamaad	Hat Ballad	Dati-	
	Element Label	Cold formed	Hot Rolled	Ratio	
		KN	KN	C/H	
	1	-9.4	-9.6	0.98	
	2	-30.2	-32.8	0.92	
	3	-50.6	-54.7	0.93	
SSr	4	-63	-68	0.93	
Outer most truss	5	-71.1	-76.8	0.93	
mos	6	-74.9	-80.9	0.93	
ter	7	-74.5	-80.5	0.93	
nO	8	-69.8	-75.4	0.93	
	9	-60.9	-65.8	0.93	
	10	-47.7	-51.6	0.92	
As the second	11	-30.2	-32.8 Sri Lank <b>.9</b> .6	0.92	
1.5	University	of Moratus 4a		0.98	
	Eletaronic		sertation 21.7	0.99	
	ww4.lib.m		-74.7	0.94	
	15	-119.2	-126.4	0.94	
SSr	16	-147.5	-156.4	0.94	
interior truss	17	-166 174.6	-175.9	0.94	
erio	18	-174.6	-185	0.94	
	19	-173.4	-183.7	0.94	
1st	20 21	-162.4	-172.1	0.94 0.94	
		-141.5	-150		
	22	-110.7	-117.6	0.94	
	23 24	-69.9	-74.7 21.7	0.94	
	25	-21.5 -35	-21.7 -35.7	0.99 0.98	
	26	-80.8	-35.7	0.98	
ssn	27			0.94	
ır trı	28	-99.5 -123.1	-105.7 -130.6	0.94	
Interior truss	29	-123.1	-130.8	0.94	
<u>= </u>	30	-145.6	-140.8	0.94	
	30	-145.0	-134.4	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.

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

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	
interior truss	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	



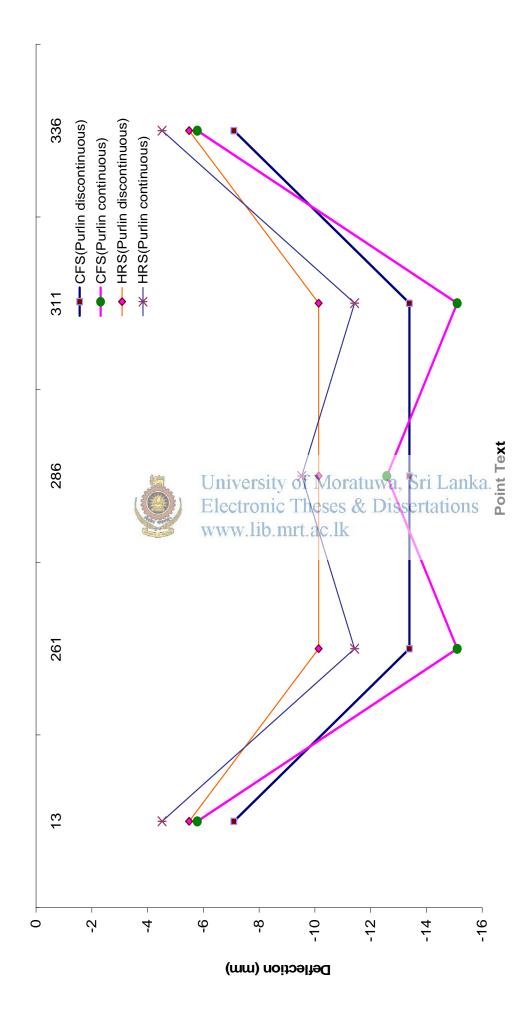


Figure 4-6: Variation of Deflection at Bottom Mid Span of Truss

#### 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	Hot Rolled	Cold form	%		
Structure	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		

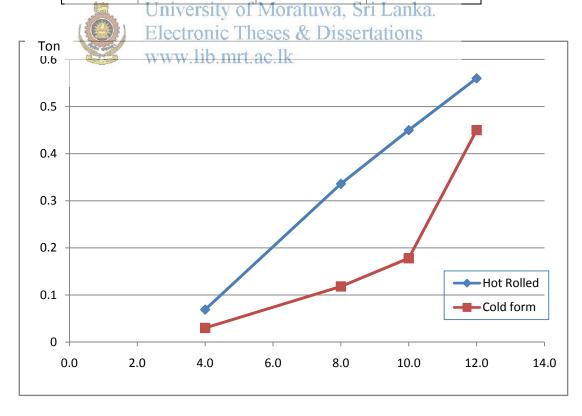


Figure 5-1: Steel Quantity Comparison for Individual Truss

#### 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

TOTAL COST (Rs) **Truss Span Cold Form Hot Rolled Net Saving** 482,750.00 559,250.00 76,500.00 4m 393,750.00 8m1,180,500.00 1,574,250.00 1,294,750.00 1,700,150.00 405,400.00 10m

**Table 5-2: Total Cost of Structure** 

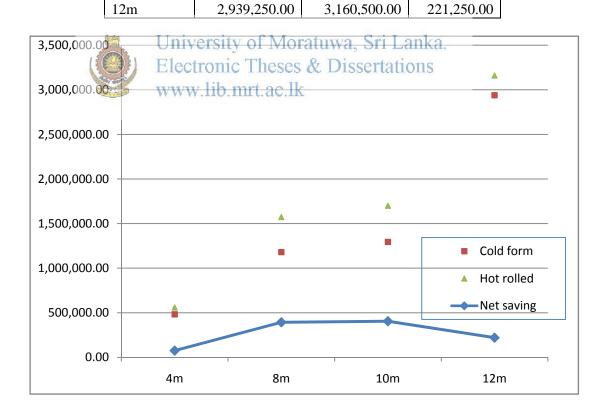


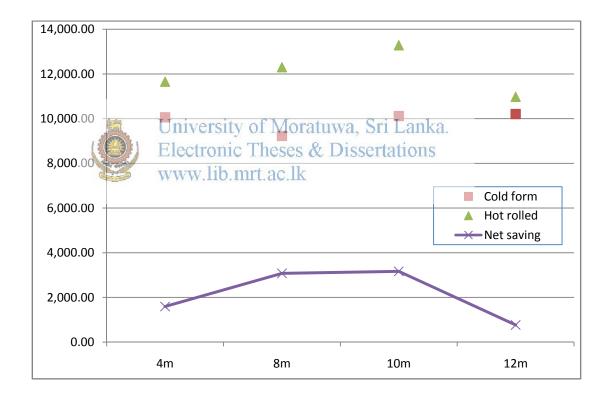
Figure 5-2: Total Cost of Roof Structure - Comparison

## 5.3. UNIT COST OF ROOF STRUCTURE

Extensive study of cost evaluation for 1m<sup>2</sup> 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 <sup>2</sup>	Cold Form	<b>Hot Rolled</b>	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 to study of the capacity of the construction of

According to the current market price of material in Sri Lanka, RS 3000/- to RS 3200/- per unit area of (1.0m<sup>2</sup>) 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.



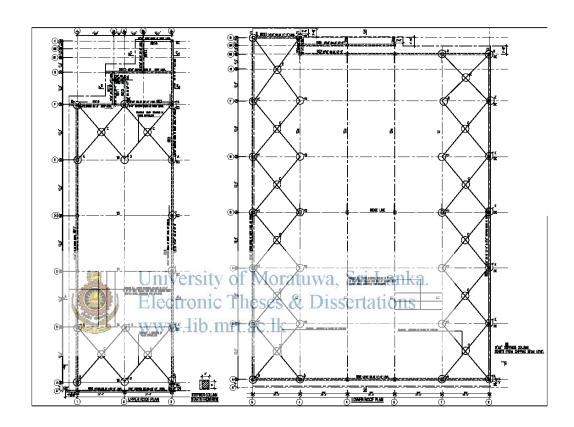
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## ANNEX: A - GENERAL LAYOUT & FEATURES OF SELECTED CASES

## 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<sup>O</sup> Degree

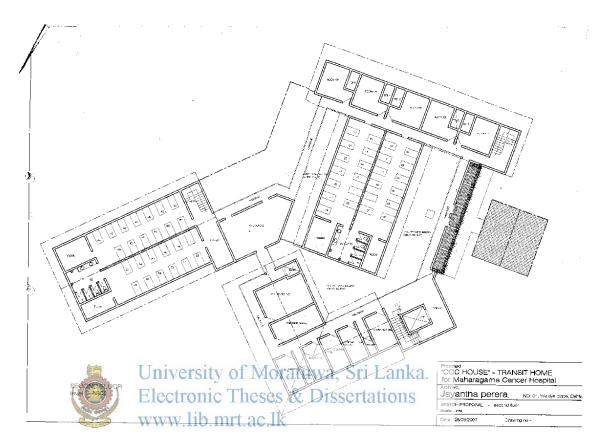
• MINIMUM EAVE HEIGHT - 3.6m

• 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<sup>O</sup>,

MINIMUM EAVE HEIGHT

- From top most storey – 3.1m

• BAY SPACING

- 3.2m

• SUPPORTING ELEMENT TYPE - RC Columns & Beams, Steel columns

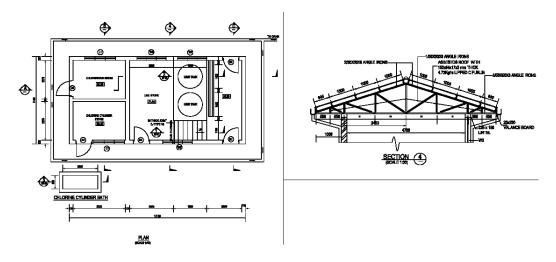
• WIND EXPOSURE CATEGORY - Post disaster structure

(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 niversity of Moratusa, Sri Lanka.

  Electronic Theses & Dissertations
- MINIMUM EAVE HEIGHT 2011 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)

 $Vs = S_1 x S_2 x S_3 x V$ 

S<sub>1</sub> - 1.00 Level terrain

S<sub>2</sub> - 0.74 Open country with scatter wind break

S<sub>3</sub> - 1.00 H-5.0m

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

 $V_S = 36.26$   $.q = k V_s^2$ 

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

University of Moratuwa, Sri Lanka. Electronic Theses & Dissertations www.flo.mrt.ac.lk

Table 13

h/w - 3.6 / 5.15 = 0.699

Roof angle - 18<sup>0</sup>

Ф=0.2

## Overall Coefficient

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

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

$$P = C_P \cdot q$$

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

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

For 3.0m Bay spacing 
$$-0.48354 \times 3 = +1.45 \text{ kN/m}$$

$$-0.78978 \times 3 = -2.37 \text{ kN/m}$$

### WIND LOAD CALCULATION TO CP 3: Chapter V

# CCC House @ Maharagama (Wind zone – 03 Post Disaster Structre)



S<sub>2</sub> - 0.88 Open country with scatter wind break

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

$$S_3 - 1.00$$

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

 $V_S = 33.44$ 

$$.q = k V_s^2$$
  $k - 0.613$ 

Table 13

$$h/w - 9 / 11.5 = 0.78$$

 $= 685.5 \text{ N/m}^2$ 

Roof angle - 28<sup>0</sup>

 $\Phi = 0.2$ 

#### Overall Coefficient

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

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

 $P = C_P \cdot q$ 

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

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

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

 $-0.72663 \times 4 = -2.91 \text{ kN/m}$ 

For 3.2m Bay spacing niversity of M.61695 v. 3; 25rt 1.974kN/m

Electronic These 72663 Ris sert 213251 kN/m

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# WIND LOAD CALCULATION TO CP 3: Chapter V

# Cargills BigCity @ Maharagama

 $Vs = S_1xS_2xS_3 \times V$ 

S<sub>1</sub> - 1.00 Level terrain

S<sub>2</sub> - 0.74 Open country with scatter wind break

S<sub>3</sub> - 1.00 H-5.0m

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

$$V_S = 24.42$$

$$q = k V_s^2$$

$$k - 0.613$$

$$= 365.6 \text{ N/m}^2$$

Table 13

$$h/w - 3.3 / 42 = 0.0.078$$

 $\Phi = 0.2$ 

**Overall Coefficient** 

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



Minimum -verpressure coefficient: Cpwa-0.82i Lanka. Electronic Theses & Dissertations www.lib.mrt.ac.lk (Maximum +ve pressure) down ward

 $P = 146.24 \text{ N/m}^2$ 

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

For 6.0m truss spacing

-0.14624x 6.0 = +0.8774 kN/m

 $-0.29979 \times 6.0 = -1.799 \text{ kN/m}$ 

#### **B.2** GRAVITY LOAD

#### GENARAL LOADINGS FOR ROOF TRUSS ANALYSIS

Live load - 0.25kN/m<sup>2</sup>

(Maximum possible impose load to calculated by analysis)

Purlin loads - NA kN/m<sup>2</sup> (Purlin have been modeled)

Roofing sheet -  $0.5 \text{ kN/m}^2$ 

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

#### LOAD EVALUATION ON ROOF STRUCTURE

### Pump House at Ampara - 4m spacing roof structure

Purlin Spacing - 1.0m

Therefore SDL UDL on perlin  $-(0.5 + 0.1) \times 1.0$ 

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UDL on perlin
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CCC House @ Maharagama - 8.0m spacing roof structure

Purlin Spacing - 1.30m

Therefore SDL UDL on perlin  $-(0.5 + 0.1) \times 1.30$ 

- 0.780 kN/m

Live load UDL on perlin - 0.325 kN/m

## CCC House @ Maharagama - 10.0m spacing roof structure

Purlin Spacing - 1.60m

Therefore SDL UDL on perlin  $-(0.5 + 0.1) \times 1.60$ 

- 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 -  $(0.5 + 0.1) \times 1.0$ 

- 0.6 kN/m

Live load UDL on perlin - 0.25 kN/m



68

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

70

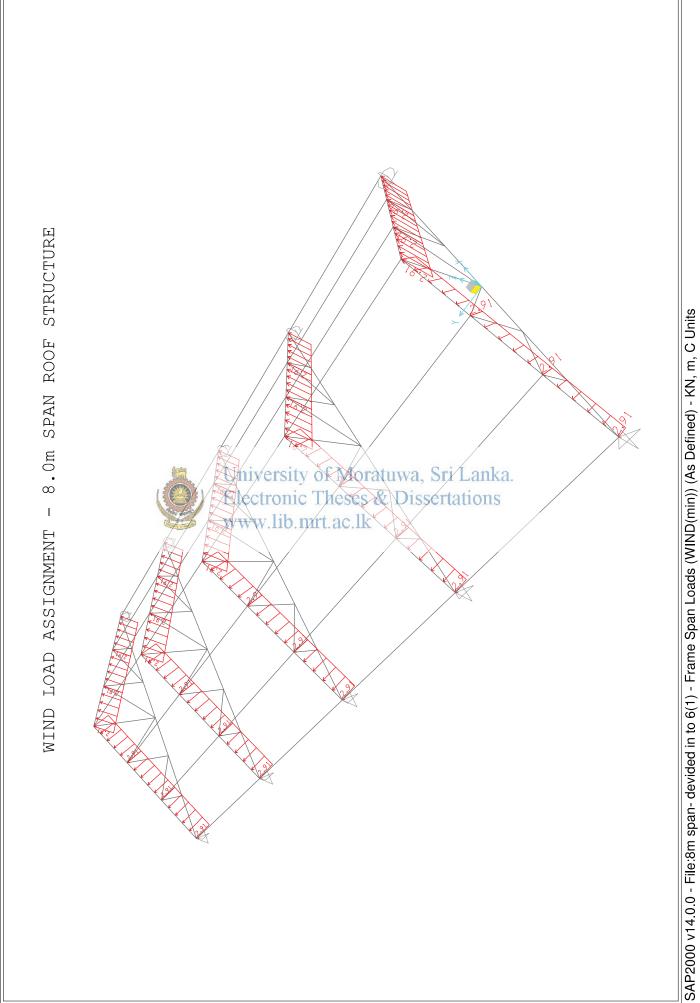
71

SAP2000

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

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

74



77

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

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

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

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

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

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	Ву	Date
PART : Cold form -10.0m Span Truss-Bot. ch		2014-03-24
DETAILED DESIGN	Tension Compre	ession
Frame Text	- 31	13
Axial force	- 44.868	-8.442 kN
M <sub>xx</sub>	- 0.089	-0.114 kNm
M <sub>yy</sub>	- 0.098	0.000 kNm
Unrestrained length of section (L)	- 1.667	m
Table 9 — Effective lengths, I		
Conditions of restraint at ends (in plane u	ınder consideration)	Effective length
Effectively held in position at both ends but not restrained	ed in direction	1.0L
Effectively held in position at both ends and restrained in	n direction at one end	0.85L
Effectively held in position and partially restrained in dir	ection 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 effectively restrained in direction but not held in position		1.2L
Effectively held in position and restrained in direction at partially restrained in direction but not held in	one end with the other end	1.5L
Effectively held in position and restrained in direction at position or restrained in direction at the other end	one end position but not hel	d in 2.0L
Effective length Factor (L <sub>E</sub> ) Electronic T WWW.Lib.mr	t.ac.lb <sub>5.0 kN/mm²</sub>	ertations
TYPE OF SECTION	- Single C section \$ 150 x 1.8 [C150x65x25(1.8)]	
Gross Section Property		
A _ 4.84 cm <sup>2</sup>		
I <sub>XX</sub> _ 165.4 cm <sup>4</sup>		
I <sub>YY</sub> - 18.87 cm <sup>4</sup>		
d _ 150 mm		
t _ 1.8 mm		
r <sub>xx</sub> - 5.85 cm		
r <sub>yy</sub> . 1.97 cm		
b <sub>2</sub> . 65 mm		
b <sub>3</sub> _ 20 mm		
Effective Length (L <sub>E</sub> )	<sub>-</sub> 1.667 m	

	DESIGN CALCULATION PART: Cold form -10.0m Span Truss-Bot. chord	By BNC	Date 2014-03-24	
	Members in Compression	5.10	2021 00 21	
Cl 6.2.2	Slenderness ratio $\left( \frac{L_E}{r_{xx}} \right)$ -	28.51		
	$\left(\begin{array}{c}L_E \\ \overline{r}_{yy}\end{array}\right)$	84.41		
	Maximum Slenderness limit			
	<ol> <li>For members resisting loads other than wind load</li> <li>For members resisting self weight and wind load</li> </ol>	-	180 250	
	3) For members resisting self-weight and wind load 3) For members acting as ties with reversal stressing	<del>-</del> -	350	
	load pattern with respect to Max.Slenderr	ness limit -	2	Max. slenderness OK
CI 6.2.3	Singly Symetrical Section			
	Effective Cross-sectional area  a) Effective Breadth of Web Element			
Annex B	For Lipped Channel: $K_1$ University of N  Electronic These www.libs/mrt.ac $7 - \frac{1}{0.15 + h} - 1.45$	ses & Dissertat		
	$h = \frac{b_2}{b_1}$ $b_1 - 148.2 \text{ mm}$ $b_2 - 65.0 \text{ mm}$ $t_1 - 1.76 \text{ mm}$ $t_2 - 1.76 \text{ mm}$	- 0.44		
	$P_{cr} = 0.904EK_1 \left(\frac{t}{b}\right)^2$	- 144.75	N/mm <sup>2</sup>	
	$rac{f_c}{P_{cr}}$	- 1.93	> 0.123	
	For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{\it eff}}{b}$ = 1			
	For $\frac{f_c}{P_{cr}} > 0.123$ $\frac{b_{eff}}{b} = \left[1 + 14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)\right\}\right]$	$\left[-0.35\right]^{4}$		

PART: Cold form -10.0m Span Truss-Bot. chord		Ву	Date
		BNC	2014-03-24
$b_{\scriptscriptstyle{\mathit{nff}}}$	_	0.565	
$rac{b_{_{e\!f\!f}}}{b}_{_{e\!f\!f}}$		0.000	
$b^{\prime\prime}_{\it eff}$	-	83.67	mm
b) Effective Breadth of Flange Element			
$K_2 = K_1 h^2 \left( \frac{t_1}{t_1} \right)$	-	1.07	
$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ $P_{cr} = 0.904 EK_2 \left(\frac{t}{b}\right)^2$			
$P_{cr} = 0.904  EK_2 \left( \frac{b}{b} \right)$	-	543.5	N/mm <sup>2</sup>
$\underline{f_c}$	_	0.515	> 0.123
$rac{f_c}{P_{cr}}$			
$rac{b_{\it eff}}{b}$	-	0.955	
$\stackrel{\scriptstyle o}{b}_{\scriptstyle \it eff}$	_	62.10	mm
eff			
ACCOUNT OF THE PROPERTY OF THE		Dissertat	ions
$\begin{array}{cccccccccccccccccccccccccccccccccccc$			ions
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	DESIGN CALCULATION	Ву	Date	
	PART : Cold form -10.0m Span Truss-Bot. chord	BNC	2014-03-24	
	b) Moment Capacity	- <b>M</b> <sub>0</sub>	Ç.	
	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			
	$M_{cy} = p_0 \times Z_{y1r}$ outstand in compression	-	1.770 kNm	
	$M_c = p_0 \times Z_{y2r}$	-	1.309 kNm	
	Where			
		on modulus for n	najor	
	Z <sub>y1r</sub> - Reduced section	on modulus for n (outstand in tens		
	_	on modulus for n		
	·	outstand in com	pression)	
CI 6.2.4	University of M  Buckling Resistance Under Axial-Load for Single symmetric Life Circuit Confession (1) P'c fabout Y-Y) Outstand the find it. ac.	ical section Disso	Sri Lanka. ertations	
	$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$	-	70.12 kN	
	ii) P' <sub>c</sub> (about Y-Y) Outstand in compression	ı		
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c} + P_{c}e_{s}}$	-	66.67 kN	
	Basic Requirment F <sub>c</sub> < (P' <sub>c</sub> ) <sub>min</sub>			
	Applied Axial Load-Compression	-	F <sub>C</sub>	
CI C 4	F <sub>C</sub>	-	8.44 kN	Buckling Resistance
Cl 6.4	Combined bending and Compression  Applied Bending Moment about X-X Axis			ОК
	$M_x$	-	0.114 kNm	
	Applied Bending Moment about Y-Y Axis $M_{y}$	-	0.000 kNm	
Cl 6.4.2	Local Capacity Check $\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$	1		

	DESIGN CALCULATION By Date	
	PART : Cold form -10.0m Span Truss-Bot. chord BNC 2014-03-	24
	0.078 + 0.020 + 0.000 - 0.098	Local Capacity OK
Cl 3.5.4	Net corss sectional area - A <sub>net</sub>	
	Net area after deduction in $3.5.4.5a$ ) = $bt - 6dt$ Net area after deduction in $3.5.4.5b$ ) = $bt - \left(9dt - \frac{8s_2^2t}{4g}\right)$	
	Number of Bolt Holes - 3 Holes Pattern - Zig-Zag	
	Hole Diameter . 8.00 mm	
	University of Moratuwa, Sri Lanka.  Electronic Thases & Dissertation www.lib.mrt.ac.lk	12
CI 7.2.2	Tension Capcity $P_{\scriptscriptstyle t} = A_{\scriptscriptstyle e}  p_{\scriptscriptstyle  y}$	
	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 <sub>2</sub> Gross sectional area of unconnected leg or legs	
Cl 7.2.3	b) If two component are parellel back to back	
	$A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $\mathbf{a_1} \qquad \text{as above}$	
	$a_1$ as above $a_2$ as above $a_1$ 232.80 mm <sup>2</sup> $a_2$ 110.52 mm <sup>2</sup>	

	DESIGN PART :	CALCULATIO Cold form -10.0		Truss-Bot. chord	By BNC	Date 2014-03-24	
		Connection typ		-	a)		
				A <sub>e</sub> _	328.22 mm <sup>2</sup>		
				- p <sub>t</sub> _	91.90 kN		
Cl 7.3	Members	s in Combined					
		Applied Tensile Applied Bendin		F <sub>t</sub> nt about X-X Axis	-	44.868 kN	
				M <sub>x</sub> nt about Y-Y Axis	-	0.089 kNm	
		Applied beliali	ig ivioillei	M <sub>y</sub>	-	0.098 kNm	
		$\frac{F_t}{P_t}$	$+\frac{M_{x}}{M_{cx}}$	$- + \frac{M_{y}}{M_{cy}} \le 1$			
		0.49 +	0.02	+ 0.06	-	0.56	Local Capacity OK
			Elec	versity of M tronic The v.lib.mrt.ac	ses & Dis	Sri Lanka. sertations	

DESIGN CALCULATI PART : Cold form -10	ON D.Om Span Truss-Bracing Member	By BNC	Date 2014-03-24	
DETAILED DESIGN				
	Tension	Compre	ession	
Frame Text	-	20	62	
Axial force	- 22.1	.05	-14.673 kN	
M <sub>xx</sub>	- 0.0	000	-0.012 kNm	
M <sub>yy</sub>	- 0.0	124	0.000 kNm	
Unrestrained length of so	ection (L) -	2.453	m	
Table	$9$ — Effective lengths, $L_{ m E}$ for compress	ion members		
	f 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 ar	nd partially restrained in direction at both end	ls	0.85L	
	nd restrained in direction at both ends		0.7L	
Effectively held in position ar effectively restrained in direct	nd restrained in direction at one end with the tion but not held in position	other end	1.2L	
Effectively held in position ar partially restrained in direction	nd restrained in direction at one end with the m but not held in	other end	1.5L	
Effectively held in position ar position or restrained in direct	nd restrained in direction at one end position ation at the other end	but not held in	2.0L	
.0	University of Morat Electronic Theses &			
Effective length Factor	(LWWW.lib.mrt.ac.lk 1		accord	
Material Properties	E - 20	5.0 kN/mm <sup>2</sup>		
	P <sub>y</sub> - 28	0.0 N/mm <sup>2</sup>		
TYPE OF SECT	•	ction		
	\$ 100 x 1.2 [C100x50x	20]		
Gross Section Property				
A _ 2.67 cr	$m^2$			
l <sub>xx</sub> <sub>-</sub> 44.31 cr	$m^4$			
I <sub>YY</sub> _ 11.49 cr	$m^4$			
d _ 100 m	ım			
t _ 1.2 m				
r <sub>xx</sub> _ 4.07 cr	m			
r <sub>yy</sub> _ 2.07 cr	n			
b <sub>2</sub> . 50 m	ım			
b <sub>3</sub> _ 20 m	ım			
Effective Length (L <sub>E</sub> )	_ 2.4	53 m		

	DESIGN CALCULATION	Ву	Date	
	PART: Cold form -10.0m Span Truss-Bracing Member	BNC	2014-03-24	
	Members in Compression			
Cl 6.2.2	Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 60.22	2		
	$\left(\begin{array}{c}L_{E}\\\hline r_{yy}\end{array}\right) \qquad - \qquad \qquad 118.25$	5		
	Maximum Slenderness limit			
	1) For members resisting loads other than wind load	-	180	
	2) For members resisting self weight and wind load	-	250 350	
	3) For members acting as ties with reversal stressing 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		2	
Annex B	University of Moratu Electronic Theses &	wa, Sri La Dissertatio	nka.	
	For Lipped Channel: $K_1$ WWW.lipsmrt.ac.lk $7 - 1000000000000000000000000000000000000$	5.43		
	$h = \frac{b_2}{b_1}$	0.51		
	$b_1$ $b_1$ - 98.8 mm $b_2$ - 50.0 mm $t_1$ - 1.16 mm $t_2$ - 1.16 mm			
	$P_{cr} = 0.904EK_1 \left(\frac{t}{h}\right)^2$	138.62	N/mm²	
	$P_{cr} = 0.904EK_{\rm I} \left(\frac{t}{b}\right)^{2}$ $\frac{f_{c}}{P_{cr}}$	2.02	> 0.123	
	For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{\it eff}}{b}$ = 1			
	For $\frac{f_c}{P_{cr}} > 0.123$ $\frac{b_{eff}}{b} = \left[1 + 14 \left\{ \left(\sqrt{\frac{f_c}{P_{cr}}}\right) - 0.35 \right] \right]$	} 4 ] -0.2		

DESIGN CALCULATION PART: Cold form -10.	<b>)N</b> Om Span Truss-Bracing	Memher	By BNC	Date 2014-03-24
ART . COID IOINI 10.	On Span Trass Bracing	Wichiber	БИС	2014 03 24
$rac{b_{\it eff}}{b}$		-	0.552	
$b_{\it eff}$		-	54.58	mm
	dth of Flange Element			
$K_2 = K_1 h^2$ $P_{cr} = 0.904 A$	$\left(\frac{t_1}{t_2}\right)^2$	-	1.39	
$P_{cr}=0.904L$	$EK_2\left(\frac{t}{b}\right)^2$	-	399.0	N/mm²
$rac{{{f}_{c}}}{{{P}_{cr}}}$		-	0.702	> 0.123
$\frac{b_{\it eff}}{b}$		-	0.890	
$\stackrel{\mathcal{b}}{b}_{_{\mathit{eff}}}$		-	44.49	mm
perry coefficie	WWW.lib.mrt PE nt y	ac.lk	59.51 38.6 0.20 52.9	kN kN
Buckling Resistance Under	Axial Load for section sy	mmetrical ab	out Both axes	or
Closed section $ P_{c} = \begin{pmatrix} \\ \\ \phi \end{pmatrix} $ $ P_{C} $	$\frac{P_E P_{CS}}{+\sqrt{\phi^2 - P_E P_{PC}}}$		30.60	kN
Determination Of Moment  a) Limiting Stress	for Stiffened Web in E			
$p_0 = \begin{cases} 1.13 - 1 \end{cases}$	$-0.0019 \frac{D_w}{t} \left(\frac{Y_s}{280}\right)$	$p_{\bar{y}}$		270.54 N/mm <sup>2</sup>

	DESIGN CALCULATION By Date	
	PART: Cold form -10.0m Span Truss-Bracing Member BNC 2014-03-24	
	b) Moment Capacity - $M_c$ i) Moment Capacity about X-X axise $M_{cx} = p_0  imes Z_{xr}$ - 2.237 kNm	
	ii) Moment Capacity about Y-Y axise  outstand in tension	
	$M_{cy} = p_0 \times Z_{y_1 r}$ - 1.023 kNm outstand in compression	
	$\boldsymbol{M}_{c} = \boldsymbol{p}_{0} \times \boldsymbol{Z}_{y2r} \qquad \qquad \text{0.871 kNm}$	
	Where  Z <sub>xr</sub> - Reduced section modulus for major axise bending	
	Z <sub>y1r</sub> - Reduced section modulus for minor axise bending (outstand in tension)  Z <sub>y2r</sub> - Reduced section modulus for minor axise bending (outstand in compression)	
	<b>6</b> (	
Cl 6.2.4	Buckling Resistance Under Axial Load for Single symmetrical section  University of Moratuwa, Sri Lanka.  i) Property Y-Y-Doutstand in tension less & Dissertations	
	$P_c = \frac{\text{www.clib.mrt.ac.lk}}{M_c + P_c e_s}$ 27.64 kN	
	ii) P' <sub>c</sub> (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 <sub>C</sub>	
	F <sub>C</sub> - 14.67 kN	Buckling Resistance OK
Cl 6.4	Combined bending and Compression	
	Applied Bending Moment about X-X Axis  M <sub>x</sub> - 0.012 kNm	
	Applied Bending Moment about Y-Y Axis  M <sub>y</sub> - 0.000 kNm	
CI 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 PART: Cold form -10.0m Span Truss-Bracing Member	By BNC	Date 2014-03-24	
	0.247 + 0.005 + 0.000 -	0.252		Local Capacity OK
CI 3.5.4	Net corss sectional area  5 holes in line  Total of 9 holes and 8 gas  b  s  Net area after deduction in 3.5.4.5a) = bt - 5dt  Net area after deduction in 3.5.4.5a		A <sub>net</sub>	
	Number of Bolt Holes Holes Pattern Hole Diameter  University of Morat Electronic Theses & www.lib.mrt.ac.lk	uwa, Sri La Dissertatio	3 Zig-Zag 8.00 mm 20 150 95.2 mm <sup>2</sup>	
Cl 7.2.2	Tension Capcity $P_{_{t}}=A_{_{e}}p_{_{y}}$			
	a) For single angle ties connected through one leg o For plain channel section connected only through For "T" sections connected only through the flang $A_e = \frac{a_1(3a_1+4a_2)}{(3a_1+a_2)}$	n web		
	$a_1$ . Net sectional area of connected leg $a_2$ . Gross sectional area of unconnected $b$	eg or legs		
Cl 7.2.3	b) If two component are parellel back to back			
		20 mm² 12 mm²		

	ESIGN CALCULATION  ART: Cold form -10.0m Span Truss-Bracing Member	•	Pate 2014-03-24
	Connection type -	a)	
	A <sub>e</sub> - 1	42.80 mm <sup>2</sup>	
	p <sub>t</sub> _	39.98 kN	
.3 <b>M</b>	embers in Combined Bending & Tension		
	Applied Tensile Load F <sub>t</sub> - Applied Bending Moment about X-X Axis	22.105 k	N
	M <sub>x</sub> - Applied Bending Moment about Y-Y Axis	0.000 k	Nm
	$M_{y}$ -	0.024 k	Nm
	$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$		
	0.55 + 0.00 + 0.02 -	0.58	Local Capacity OK
	University of Mor Electronic Theses www.lib.mrt.ac.lk	& Dissertation	
			ĺ

	DESIGN CALCULATION		Dv	Data			
	PART : Cold form -10.0m Span Truss-Top	chord	By BNC	Date 2014-03-24			
	PART . Cold form -10.0m span fruss-10p	Ciloru	DIVC	2014-03-24			
	DETAILED DESIGN						
	DETAILED DESIGN	Tension	Compression	,			
	   Frame Text	- 14	33	'			
	Axial force	- 10.957	-52.221	kN			
	M <sub>xx</sub>	- 0.749	-0.292	kNm			
	M <sub>yy</sub>	0.001	-0.017	kNm			
	,,						
	Unrestrained length of section (L)	- 1	.894	m			
	Table 9 — Effective length	${ m s}, L_{ m E}$ for compress	sion members				
	Conditions of restraint at ends (in plan	ne under consideration	n)	Effective length			
	Effectively held in position at both ends but not restra	ained in direction		1.0L			
	Effectively held in position at both ends and restraine	d in direction at one	end	0.85L			
	Effectively held in position and partially restrained in			0.85L			
	Effectively held in position and restrained in direction			0.7L			
	· 1		othon or J	-			
	Effectively held in position and restrained in direction effectively restrained in direction but not held in position.		e outer end	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	2.0L					
	position or restrained in direction at the other safey						
	( Electronic	Theses &	Dissertat	ions			
able.	www.lib.m						
09	Effective length Factor (L <sub>E</sub> )	_ 1.0	0 I				
	Material Properties E		0 kN/mm <sup>2</sup>				
	P <sub>v</sub>		0 N/mm <sup>2</sup>				
	ı y	- 280.	0 14/111111				
	TVDE OF SECTION	Cinala Casatia					
	TYPE OF SECTION	- Single C section S 150 x 1.8	on				
		[C150x65x25(	1.8)]				
	Gross Section Property	(	71				
	A _ 4.84 cm <sup>2</sup>						
	l <sub>XX</sub> <sub>-</sub> 165.4 cm <sup>4</sup>						
	l <sub>yy</sub> - 18.87 cm <sup>4</sup>						
	d _ 150 mm						
	t _ 1.8 mm						
	r <sub>xx</sub> <sub>-</sub> 5.85 cm						
	r <sub>yy</sub> <sub>-</sub> 1.97 cm						
	b <sub>2 -</sub> 65 mm						
	•						
	b <sub>3</sub> . 20 mm						
	$b_3$ . 20 mm Effective Length ( $L_E$ )	_ 1.89	4 m				

	DESIGN CALCULATION PART: Cold form -10.0m Span Truss-Top chord	By BNC	Date 2014-03-24	
	Members in Compression			
Cl 6.2.2	Slenderness ratio $\left(\begin{array}{c} L_{E} \\ r_{xx} \end{array}\right)$ -	32.40		
	$\left(\begin{array}{c}L_{E}\\\hline r_{yy}\end{array}\right)$	95.93		
	Maximum Slenderness limit			
	<ol> <li>For members resisting loads other than wind load</li> <li>For members resisting self weight and wind load</li> <li>For members acting as ties with reversal stressing</li> </ol>	- - -	180 250 350	
	load pattern with respect to Max.Slenderne	ess limit -	2	Max. slenderness OK
Cl 6.2.3	Singly Symetrical Section			
Annex B	Effective Cross-sectional area a) Effective Breadth of Web Element of Modern Electronic These www.lip.gnrt.ac.l $7 - \frac{7}{0.15 + h} - 1.43$ .	s & Dissertat		
	$h = rac{b_2}{b_1}$ $b_1$ - 148.2 mm $b_2$ - 65.0 mm $t_1$ - 1.76 mm $t_2$ - 1.76 mm	- 0.44		
	$P_{cr} = 0.904EK_1 \left(\frac{t}{b}\right)^2$	- 144.75	N/mm <sup>2</sup>	
	$rac{f_c}{P_{cr}}$	- 1.93	> 0.123	 
	For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{\it eff}}{b}$ = 1			
	For $\frac{f_c}{P_{cr}} > 0.123$ $\frac{b_{eff}}{b} = \left[1 + 14 \left\{ \sqrt{\frac{f_c}{P_{cr}}} \right\} \right]$	$-0.35$ $^{4}$ $^{-0.2}$		

DESIGN CALCULATION		Ву	Date	
PART : Cold form -10.0m Span Truss-Top of	chord	BNC	2014-03-24	
h				
$\frac{b_{ ext{eff}}}{b}$	-	0.565		
$b_{\ e\!f\!f}$	-	83.67	mm	
b) Effective Breadth of Flange Elemer	nt			
$K = \frac{1}{2} \left( t_1 \right)^2$				
$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ $P_{cr} = 0.904 EK_2 \left(\frac{t}{b}\right)^2$	-	1.07		
$P = 0.904 EK \left(\frac{t}{t}\right)^2$			2	
$I_{cr} = 0.904 EK_2 \left(\frac{b}{b}\right)$	-	543.5	N/mm <sup>2</sup>	
		0.545	. 0.122	
$\frac{f_c}{P_{cr}}$	=	0.515	> 0.123	
- cr				
$b_{\it eff}$	_	0.955		
$rac{b_{ ext{ iny eff}}}{b}$	-	0.555		
$b_{e\!f\!f}$	-	62.10	mm	
eff				
Q - Factore Representing Reduced	Cross Section			
Q	-	0.801		
$A_{eff}$	-	387.68	mm <sup>2</sup>	
University (	of Morati	uwa Sr	i Lanka	
Electronic 7			lamons	
www.lib.em	rt.ac.lk	106.4	kN	
perry coefficient ŋ	-	0.15		
ф	-	115.6		
Buckling Resistance Under Axial Load for section Closed section	symmetrical ab	out Both axes	s or	
Closed Section				
$P = \begin{pmatrix} P_E P_{CS} \end{pmatrix}$				
$P_{c} = \left(\frac{P_{E}P_{CS}}{\phi + \sqrt{\phi^{2} - P_{E}P_{PC}}}\right)$				
(7 · V7 - E-PC)				
P <sub>C</sub>	-	73.10	kN	
			KIV	
Determination Of Moment Capacity				
a) Limiting Stress for Stiffened Web in				
$\int_{\mathbb{R}^{n}} D \left( Y \right)$	)1/2			
$p_0 = \begin{cases} 1.13 - 0.0019 \frac{D_w}{t} \left( \frac{Y_s}{280} \right) \end{cases}$	$\frac{1}{2}$ $p_{\bar{y}}$	27	71.06 N/mm <sup>2</sup>	
1 (280	<i>y</i> )			

	DESIGN CALCULATION	By	Date	
	PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24	
	b) Moment Capacity	$M_c$		
	i) Moment Capacity about X-X axise		784 kNm	
	$M_{cx} = p_0 \times Z_{xr}$	5.	704 KIVIII	
	ii) Moment Capacity about Y-Y axise outstand in tension			
	$M_{cy} = p_0 \times Z_{y1r}$ outstand in compression	- 1.	770 kNm	
	$M_c = p_0 \times Z_{y2r}$	- 1	309 kNm	
	Where			
	Z <sub>xr</sub> - Reduced sec axise bendin	tion modulus for maj g	or	
	<b>,</b>	tion modulus for min		
		tion modulus for min		
	<b>,</b>	g (outstand in compr		
Cl 6.2.4				
	Buckling Resistance Under Axial Load for Stingle Symmet	rical séculon a Sri	Lanka.	
	Flectronic These			
	i) P tabout Y-Y) Outstand in tension	11	attons	
	$P'_{c} = \frac{w_{c} P_{c} \text{lib.mrt.ac.}}{M_{c} + P_{c} e_{s}}$	.lk		
	$P_c = \frac{c c}{M + P \rho}$	- 63	3.39 kN	
	ii) P' <sub>c</sub> (about Y-Y) Outstand in compressio	n		
	M/P			
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c}+P_{c}}$	- 60	).55 kN	
	$M_c + P_c e_s$			
	Basic Requirment F <sub>c</sub> < (P <sub>c</sub> ) <sub>min</sub>			
	Applied Axial Load-Compression	_	F <sub>C</sub>	
	F <sub>C</sub>	- 52	2.22 kN	Buckling Resistance
Cl 6.4		32		OK
	Combined bending and Compression			
	Applied Bending Moment about X-X Axis			
	$M_x$	- 0.3	292 kNm	
	Applied Bending Moment about Y-Y Axis	_	047 111	
	$M_{y}$	- 0.0	017 kNm	
Cl 6.4.2				
	Local Capacity Check $\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$			

	DESIGN CALCULATION	Ву	Date	
	PART : Cold form -10.0m Span Truss-Top chord	BNC	2014-03-24	
	0.481 + 0.050 + 0.013 -	0.544	4	Local Capacity OK
Cl 3.5.4	Net corss sectional area	-	A <sub>net</sub>	
	5 holes in line Total of 9 holes and 8 gam	ge spaces in zig-zag lin	e	
	Net area after deduction in 3.5.4.5a) = $bt - 5dt$ Net area after deduction in 3.5.4. Figure 1 — Nomenclature for staggered holes with example 1.5.4.		)	
	Number of Bolt Holes	_	3	
	Holes Pattern	-	Zig-Zag	
	Hole Diameter	-	8.00 mm	
	University of Moratu Electronic Theses & www.lib.mrt.ac.lk	iwa, Sri L Dissertat	igska.	
CI 7.2.2	Tension Capcity $P_{_{t}}=A_{_{e}}p_{_{y}}$			
	a) For single angle ties connected through one leg on For plain channel section connected only through For "T" sections connected only through the flange $A_e = \frac{a_1(3a_1+4a_2)}{(3a_1+a_2)}$	web e		
	a <sub>1 -</sub> Net sectional area of connected leg			
	a <sub>2</sub> - Gross sectional area of unconnected le	g 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 <sub>1</sub> _ as above			
	a <sub>2</sub> _ as above	2		
	a <sub>1</sub> - 232.80 a <sub>2</sub> - 110.53			

	DESIGN CALCULATION	. Curan Turran Tananhand	Ву	Date 2014 03 24	
	PART : Cold form -10.0m	Span Truss-Top chord	BNC	2014-03-24	
	Connection type	-	a)		
		A <sub>e</sub> _	328.22 mm <sup>2</sup>		
		p <sub>t</sub> _	91.90 kN		
Cl 7.3	Members in Combined B	ending & Tension			
	Applied Tensile Lo	oad F <sub>t</sub> Moment about X-X Axis	-	10.957 kN	
		M <sub>x</sub> Moment about Y-Y Axis	-	0.749 kNm	
	Applied Bending	M <sub>y</sub>	-	0.001 kNm	
	$\frac{F_t}{P_t}$ +	$\frac{M_{x}}{M_{cx}} + \frac{M_{y}}{M_{cy}} \le 1$			
	0.12 +	0.13 + 0.00	_	0.25	Local Capacity OK
	( <b>(</b> ) E	Iniversity of Melectronic These www.lib.mrt.ac.	es & Disse		

DESIGN CALCULATION		Ву		Date
PART : Cold form -12.0m Span Truss-Bottom	n chord	BNC		2014-03-2
DETAILED DESIGN  Frame Text	Tension - 687 - 146.877		ression 682 2.785	kN
M <sub>xx</sub> -				
$M_{yy}$ - Unrestrained length of section (L) - Table 9— Effective lengths, $L$		1.010	0.000 hers	kNm m
Conditions of restraint at ends (in plane un				Effective length
Effectively held in position at both ends but not restrained	d 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 dire				0.85L
Effectively held in position and restrained in direction at I				0.7L
Effectively held in position and restrained in direction at effectively restrained in direction but not held in position	one end with the	e other end	d	1.2L
Effectively held in position and restrained in direction at a partially restrained in direction but not held in	one end with the	e other end	d	1.5L
Effectively held in position and restrained in direction at oposition or restrained in direction at the other end	one end positior	but not h	ield in	2.0L
TYPE OF SECTION -	- Double C se	ction		
	D 200 x 2.0			
[2/C200	x75x20 -Back	to Back	Double	Section]
Gross Section Property  A		to Back	Double	Section]

	DESIGN CALCULATION	Ву	Date	
	PART : Cold form -12.0m Span Truss-Bottom chord	BNC	2014-03-22	
	Members in Compression  Slenderness ratio $\left(\begin{array}{c} L_E \\ \overline{r_{xx}} \end{array}\right)$ - 13.13 $\left(\begin{array}{c} L_E \\ \overline{r_{yy}} \end{array}\right)$ - 35.93			
	Maximum Slenderness limit  1) For members resisting loads other than wind load  2) For members resisting self weight and wind load  3) For members acting as ties with reversal stressing	- - -	180 250 350	
	load pattern with respect to Max.Slenderness lin	nit -	2	Max. slenderness OK
Cl 6.2.3	Singly Symetrical Section			
Annex 3	Effective Cross-sectional area  a) Effective Breadth of Web Element of Mora $f$ Electronic Theses & www.lib.mrt.ac.lk  For Lipped Channel: $K_1 = 7 - \frac{1.8h}{0.15 + h} - 1.43h^3$	tuwa, Sri l 280 Disserta 4.88	Lanka. N/mm² tions	
	$h = \frac{b_2}{b_1}$ $b_1$ - 198.0 mm $b_2$ - 150.0 mm $t_1$ - 1.96 mm $t_2$ - 1.96 mm	0.76		
	$P_{cr} = 0.904EK_1 \left(\frac{t}{b}\right)^2$ $\frac{f_c}{P_{cr}}$	88.54 3.16	N/mm <sup>2</sup> > 0.123	
	For $\frac{f_c}{P_{cr}} < 0.123;$ $\frac{b_{eff}}{b} = 1$ For $\frac{f_c}{P_{cr}} > 0.123$ $\frac{b_{eff}}{b} = \left[1 + 14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right) - 0.5\right\}\right]$	$35$ $^{4}$ $^{-0.2}$		

DESIGN CALCULATION	Ву	Date	
PART : Cold form -12.0m Span Truss-Bottom chord			
<i>b</i>	- 0.44	2	
$egin{array}{c} b_{e\!f\!f} \ b \end{array}$	0.11	_	
$b^{ u}_{\it eff}$ -	- 87.5	2 mm	
b) Effective Breadth of Flange Element			
$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ $P_{cr} = 0.904 EK_2 \left(\frac{t}{b}\right)^2$	2.80		
$P = 0.904 EK_{\circ} \left(\frac{t}{t}\right)^{2}$	120	C N/m = 2	
· · ·	- 126.	6 N/mm²	
$rac{f_c}{P_{cr}}$	- 2.21	2 > 0.123	ı
$P_{cr}$			
$b_{\scriptscriptstyle \mathrm{off}}$	0.53	0	
$rac{b_{\mathit{eff}}}{b}$	- 0.52	ō	
$b_{eff}$	- 79.1	6 mm	
<i></i>			
Q - Factore Representing Reduced Cross Sec	ction		
Q - ractore representing reduced cross sec	- 0.73		
$\stackrel{ ext{A}}{A}_{eff}$	- 982.	2	
Anto TT 1		G : I 1	
University of Mo			
Electroffic These	s & 2D1	sertations	
perry coefficient www.lip.mrt.ac.l	0.03		
- ·	1220	5.7	
Buckling Resistance Under Axial Load for section symmetri	ical about B	oth axes or	
Closed section			
$P_{c} = \left(\frac{P_{E}P_{CS}}{\phi + \sqrt{\phi^{2} - P_{E}P_{PC}}}\right)$			
$\left(\phi + \sqrt{\phi^2 - P_E P_{PC}}\right)$			
P <sub>C</sub> -	265.	46 kN	
Determination Of Moment Capacity			
a) Limiting Stress for Stiffened Web in Bending			
$p_0 = \left\{ 1.13 - 0.0019 \frac{D_w}{t} \left( \frac{Y_s}{280} \right)^{1/2} \right\} p$	_	262.11 N/mm <sup>2</sup>	
$p_0 = \frac{1.13 - 0.0019}{t} \left(\frac{280}{280}\right)$	у	ZOZ.II N/IIIII	

	DESIGN	CALCULATION	Ву	Date	
	PART :	Cold form -12.0m Span Truss-Botton	n chord BNC	2014-03-22	
	b)	Moment Capacity	- <i>I</i>	$M_c$	
		i) Moment Capacity about X-X axise	_	19.554 kNm	
		$M_{cx} = p_0 \times Z_{xr}$		13.334 KMIII	
		ii) Moment Capacity about Y-Y axise outstand in tension			
		$M_{cy} = p_0 \times Z_{y1}$ outstand in compression		4.293 kNm	
		$M_c = p_0 \times Z_{y2r}$	-	4.293 kNm	
		Where			
		$Z_{xr}$ - Reduced axise be	d section modulus fo ending	or major	
		7-	d section modulus fo ending (outstand in t		
			d section modulus fo		
		· · · · · · · · · · · · · · · · · · ·	ending (outstand in c	ompression)	
Cl 6.2.4	Buckling F	Resistance Under Axial Load for Single sy	mmetrical section		
		i) P' <sub>c</sub> (about Y-Y) Outstand in tensio	n		
		University o	f Moratuwa	, Sri Lanka.	
		Electronic T	heses & Dis	S265.26 k011S	
		Marty Alb. mr			
		WWW W ZIID. IIII	c.do.iik		
		ii) P' <sub>c</sub> (about Y-Y) Outstand in comp	ression		
		M D			
		$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$	-	265.46 kN	
		$M_c + P_c e_s$			
		5 (p)			
	Basic Requ			F	
		Applied Axial Load-Compression $F_C$	-	F <sub>C</sub> 2.79 kN	Buckling Resistance
		ı c	-	2.75 KIN	bucking Resistance
	a	11 11 12			ок
Cl 6.4	Combine	ed bending and Compression  Applied Bending Moment about X-X	Δvic		
		M <sub>x</sub>	-	1.763 kNm	
		Applied Bending Moment about Y-Y	Axis		
		$M_{y}$	-	0.000 kNm	
		E M M			
Cl 6.4.2	Local Capa	city Check $\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M}{M}$	$\frac{y}{2} \le 1$		
		$cs$ $rac{1}{1}cx$ $rac{1}{1}cx$	cy		

	DESIGN CALCULATION	By Date	
	PART: Cold form -12.0m Span Truss-Bottom chord	BNC 2014-03-2	2
	0.010 + 0.090 + 0.000 -	0.100	Local Capacity OK
Cl 3.5.4	Net corss sectional area	- A <sub>net</sub>	
	5 holes in line Total of 9 holes and	d 8 gange spaces in zig-zag line	
	Net area after deduction in 3.5.4.5a) = $bt-5dt$ Net area after deduction in Figure 1 — Nomenclature for staggered holes with	• • •	
	Number of Bolt Holes	- 3	
	Holes Pattern	- Zig-Zag	
	Hole Diameter	_ 8.00 mm	
	University of Mor Electronic Theses www.lib.mrt.ac.lk	& Dissertations mm <sup>2</sup>	
Cl 7.2.2	Tension Capcity $P_{\scriptscriptstyle t} = A_{\scriptscriptstyle e}  p_{\scriptscriptstyle y}$		
	<ul> <li>a) For single angle ties connected through one le For plain channel section connected only through the fl</li> </ul>	ough web	
	$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$	$(a_2)$	
	<ul> <li>a<sub>1</sub> . Net sectional area of connected leg</li> <li>a<sub>2</sub> . Gross sectional area of unconnected</li> </ul>		
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 <sub>1 -</sub> as above		
	$a_2$ $_{\scriptscriptstyle\perp}$ as above		
		4.00 mm <sup>2</sup> 2.00 mm <sup>2</sup>	

	DESIGN	CALCULATIO	N			Ву	Date	
	PART :	Cold form -12.0	m Span Tr	russ-Bottom cho	rd	BNC	2014-03-22	
		Connection typ	e	-	b)			
				A <sub>e</sub> _	1004.48	$mm^2$		
				-				
				p <sub>t</sub> _	281.25	KIN		
Cl 7.3	Member	s in Combined	Bending	& Tension				
		Applied Tensile		F <sub>t</sub> t about X-X Axis	-		146.877 kN	
				$M_x$	-		2.302 kNm	
		Applied Bendin	g Moment	t about Y-Y Axis M <sub>y</sub>	_		0.000 kNm	
		$\frac{F_t}{R}$	$+\frac{M_{x}}{M}$	$+ \frac{M_{y}}{M_{cy}} \le 1$				
							0.64	Local Canacity
		0.52 +	0.12	+ 0.00	-		0.64	Local Capacity OK
			Unive	ersity of N	lorat	uwa	, Sri Lanka.	
							sertations	
		The same of the sa	www.	.lib.mrt.ac	.lk			

	DESIGN OF COLD FORM STEEL S	LCTIONS
	DESIGN CALCULATION By	Date
	PART: Cold form -12.0m Span Truss-Bracing member BNC	2014-03-22
	DETAILED DESIGN  Tension Compression  Frame Text - 634	on 632
	Axial force - 53.956 -47.7	702 kN
	M <sub>xx</sub> - 1.578 0.0	064 kNm
	M <sub>yy</sub> - 0.000 -0.0	005 kNm
	Unrestrained length of section (L) - 1.032	m
	Table 9 — Effective lengths, $L_{ m E}$ for compression members	l
	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	University of Moratuwa, Sri Electronic Theses & Dissert	
	Material Properties E - 205.0 kN/mm <sup>2</sup>	
	$P_{y}$ - 280.0 N/mm $^{2}$	
	TYPE OF SECTION - Single C section S 200 x 2.0 [C200x75x20]	
	Gross Section Property	
	A _ 6.73 cm <sup>2</sup>	
	$I_{XX}$ _ 398.7 cm <sup>4</sup>	
	$I_{YY}$ - 34.03 cm <sup>4</sup>	
	d _ 200 mm	
	t _ 2 mm	
	r <sub>xx</sub> <sub>-</sub> 7.70 cm	
	r <sub>yy -</sub> 2.25 cm	
	b <sub>2 -</sub> 75 mm	
	b <sub>3</sub> . 20 mm	
	Effective Length (L <sub>E</sub> ) _ 1.032 m	

	DESIGN OF COLD FOR			l
	DESIGN CALCULATION	Ву	Date	
	PART: Cold form -12.0m Span Truss-Bracing member	BNC	2014-03-22	
	Members in Compression			
Cl 6.2.2	Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - 13.43	1		
	$\left(\begin{array}{c}L_{E}\\r_{yy}\end{array}\right) \qquad - \qquad \qquad 45.89$	9		
	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	
	5/10/ Hembers deting as ties with reversal stressing	-	330	
	load pattern with respect to Max.Slenderness lin	nit -	2	Max. slenderness OK
Cl 6.2.3	Singly Symetrical Section			
Annex B	Effective Cross-sectional are University of Moral a) Effective Breadth of Web Element Theses & Www.lib.mrt.ac.lk	tuwa, Sri l ¿ <sub>28</sub> disserta 5.63	∟anka. ti <sub>0</sub> ns²	
	For Lipped Channel: $K_1 = 0.15 + h$			
	$h = rac{b_2}{b_1}$ - 198.0 mm $b_2$ - 75.0 mm $t_1$ - 1.96 mm $t_2$ - 1.96 mm	0.38		
	$P_{cr} = 0.904EK_1 \left(\frac{t}{b}\right)^2$	102.29	N/mm <sup>2</sup>	
	$rac{f_c}{P_{cr}}$	2.74	> 0.123	
	For $\frac{f_c}{P_{cr}}$ < 0.123; $\frac{b_{\it eff}}{b}$ = 1	0.0		
	For $\frac{f_c}{P_{cr}} > 0.123$ $\frac{b_{eff}}{b} = \left[1 + 14 \left\{ \left(\sqrt{\frac{f_c}{P_{cr}}}\right) - 0.3 \right\} \right]$	$\begin{bmatrix} 35 \end{bmatrix}^4 \begin{bmatrix} -0.2 \\ 1 \end{bmatrix}$		
,				

DECICAL CALCULATION			L SECTIONS	
DESIGN CALCULATION		Ву	Date	1
PART : Cold form -12.0m Span Truss-Bracing	member	BNC	2014-03-22	
$b_{\it eff}$	-	0.475		
$\stackrel{b}{b}_{_{e\!f\!f}}$	-	93.97	mm	
b) Effective Breadth of Flange Element				
$K_2 = K_1 h^2 \left(\frac{t_1}{t_1}\right)^2$	-	0.81		
$P_{cr} = 0.904 EK_2 \left(\frac{t}{L}\right)^2$	-	506.3	N/mm²	
$f_c$	-	0.553	> 0.123	
$rac{b_{\it eff}}{b}$	-	0.944		
$b_{\ e\!f\!f}$	-	70.77	mm	
University of	f Morat			
Buckling Resistance Under Axial Load for section sy Closed section	mmetrical al	bout Both a	axes or	
$P_{c} = \left(\frac{P_{E}P_{CS}}{\phi + \sqrt{\phi^{2} - P_{E}P_{PC}}}\right)$ $P_{C}$	-	128.68	kN	
Determination Of Moment Capacity a) Limiting Stress for Stiffened Web in B $p_0 = \begin{cases} 1.13 - 0.0019 \frac{D_w}{t} \left( \frac{Y_s}{280} \right) \end{cases}$			262.11 N/mm²	
	$\frac{b_{eff}}{b}$ $b) \text{ Effective Breadth of Flange Element}$ $K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ $P_{cr} = 0.904  EK_2 \left(\frac{t}{b}\right)^2$ $\frac{f_c}{P_{cr}}$ $\frac{b_{eff}}{b}$ $b_{eff}$ $Q - \text{Factore Representing Reduced Cr}$ $Q - \text{Factore Representing Reduced Cr}$ $Electronic T \\ \text{www.lip}_{e} \\ \text{mrt}$ $perry coefficient \\ \psi$ $d$ $Buckling Resistance Under Axial Load for section sy Closed section P_c = \left(\frac{P_E P_{cs}}{\phi + \sqrt{\phi^2 - P_E P_{Pc}}}\right) P_c Determination Of Moment Capacity$	$\frac{b_{eff}}{b} \frac{b}{eff} \qquad -$ b) Effective Breadth of Flange Element $K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2 - P_{cr} = 0.904  EK_2 \left(\frac{t}{b}\right)^2 - \frac{f_c}{P_{cr}} - \frac{b_{eff}}{b} - \frac{b_{eff}}{b} - \frac{c}{b_{eff}} $	$\frac{b_{eff}}{b} = 0.475$ $\frac{b_{eff}}{b} = 0.475$ b) Effective Breadth of Flange Element $K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2 = 0.81$ $P_{cr} = 0.904 E K_2 \left(\frac{t}{b}\right)^2 = 0.553$ $\frac{f_c}{P_{cr}} = 0.904 E K_2 \left(\frac{t_1}{b}\right)^2 = 0.944$ $\frac{b_{eff}}{b} = 0.944$ $\frac{b_{eff}}{b} = 0.944$ $\frac{d_{eff}}{b} = 0.70.77$ Q - Factore Representing Reduced Cross Section $\frac{d_{eff}}{d} = 0.70.77$ Q - Factore Representing Reduced Cross Section $\frac{d_{eff}}{d} = 0.70.77$ Clare Representing Reduced Cross Section $\frac{d_{eff}}{d} = 0.70.77$ Electronic Theses & Diss www.libe mrt.ac.lk 646.5 $\frac{d_{eff}}{d} = 0.05$ $d_{ef$	$\frac{b_{eff}}{b} = 0.475$ $\frac{b_{eff}}{b} = 0.93.97  \text{mm}$ b) Effective Breadth of Flange Element $K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2 = 0.81$ $P_{cr} = 0.904 E K_2 \left(\frac{t}{b}\right)^2 = 506.3  \text{N/mm}^2$ $\frac{f_c}{P_{cr}} = 0.553 > 0.123$ $\frac{b_{eff}}{b} = 0.944$ $b_{eff} = 0.944$ $b_{eff} = 0.70.77  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.777} = 0.777  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - \text{Factore Representing Reduced Cross Section}$ $Q = \frac{0.727}{0.727} = 0.727  \text{mm}$ $Q - Factore Representing Reduced C$

	DESIGN OF COLU			1
	DESIGN CALCULATION	Ву	Date	_
	PART: Cold form -12.0m Span Truss-Bracing mem	ber BNC	2014-03-2	2
	b) Moment Capacity	-	$M_{c}$	
	i) Moment Capacity about X-X axise		c c	
	$M_{cx} = p_0 \times Z_{xr}$	-	9.777 kNm	
	ii) Moment Capacity about Y-Y axise			
	outstand in tension			
	$M_{cy} = p_0 \times Z_{y1r}$ outstand in compression	-	2.506 kNm	
		-	1.856 kNm	
	$M_c = p_0 \times Z_{y2r}$			
	Where			
	Z <sub>xr</sub> - Reduced secti		or major	
	axise bending			
	Z <sub>y1r</sub> - Reduced secti			
	axise bending			
	Z <sub>y2r</sub> - Reduced secti			
	axise bending	(outstand in	compression)	
CI				
CI 6.2.4				
0.2.4				
	Buckling Resistance Under Axial Load for Single symmetry	rical section	Sri Lanka	
	i) P. Cabout v-VEdocatacom tessibilities		ssertations	
	$P_c = \frac{w_{M}v_{c}P_{c}\text{lib.mrt.ac}}{v_{c}P_{c}}$	.lk	00.64.14	
		-	98.64 kN	
	$M_c + P_c e_s$			
	ii) P' <sub>c</sub> (about Y-Y) Outstand in compression	n		
			04.45.151	
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c}}$	-	91.18 kN	
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c} + P_{c}e_{s}}$			
	Basic Requirment F <sub>c</sub> < (P' <sub>c</sub> ) <sub>min</sub>			
	Applied Axial Load-Compression	-	F <sub>C</sub>	
	F <sub>C</sub>	-	47.70 kN	Buckling Resistance
				ок
CI 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	
CI				
CI 6.4.2	Local Capacity Check $\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le$	1		
0.4.2	$\frac{1}{P_{-}} + \frac{1}{M} + \frac{1}{M} = \frac{1}{M}$	1		
	cs = -cx = -cy			
-				•

	DESIGN CALCULATION PART: Cold form -12.0m Span Truss-Bracing member	Ву	Date 2014-03-22	
	0.348 + 0.007 + 0.003 -	0.358		Local Capacity OK
CI 3.5.4	Net corss sectional area	-	۹ <sub>net</sub>	
		4.5b) = $bt - \left(9dt - \frac{8s_b^{-2}t}{4g}\right)$ ample	3 Zig-Zag 3.00 mm <b>301 k</b> a.	
CI 7.2.2	Tension Capcity $P_{t} = A_{e} p_{y}$			
	a) For single angle ties connected through one leg of For plain channel section connected only through For "T" sections connected only through the flang $A_e = \frac{a_1(3a_1+4a_2)}{(3a_1+a_2)}$ $a_1 \qquad \text{Net sectional area of connected leg}$ $a_2 \qquad \text{Gross sectional area of unconnected leg}$	h web ge		
Cl 7.2.3	b) If two component are parellel back to back			
	$A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_1  \text{as above}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_2  \text{as above}$ $a_2  \text{as above}$			

	•		DESIGN OF COL	D FUNIVI 3	TEEL SECTIONS	<u> </u>
	DESIGN	CALCULATION		Ву	Date	
	PART :	Cold form -12.0m Spar	n Truss-Bracing mei	mber BNC	2014-03-22	
		Connection type	-	a)		
			A <sub>e</sub>	492.78 mm <sup>2</sup>	-	
				137.30 KW		
Cl 7.3	Member	s in Combined Bendi				
		Applied Tensile Load Applied Bending Mom		-	53.956 kN	
		Applied Bending Mom		-	1.578 kNm	
		F $M$	,	-	0.000 kNm	
		$\frac{Y_t}{P_t} + \frac{M}{M}$	$\frac{x}{cx} + \frac{M}{M}_{cy} \le 1$			
		0.39 + 0.16	+ 0.00	-	0.55	Local Capacity OK
		Elec	versity of M etronic Thes w.lib.mrt.ac	ses & Di	a, Sri Lanka. issertations	

DESIG PART :	N CALCULATIO	<b>N</b> Om Span Truss-Top ch	B word B	y NC	Date 2014-03-24
PART :	Cold form -12.0	om Span Truss-Top Cr	юги в	INC	2014-03-24
DETAILE	D DESIGN				
				Compres	
Frame T Axial for			- 702 1.147	1./	696 5.559 kN
M <sub>xx</sub>	ce		1.147 0.084		0.127 kNm
M <sub>yy</sub>			- 0.000		0.000 kNm
	ained length of sec	tion (L)		10	m
		(=)			
	Table 9	— Effective lengths, $L_{\mathbb{R}}$	for compression	members	
	Conditions of re	straint at ends (in plane und	er consideration)		Effective length
Effective	y held in position at bo	th ends but not restrained	in direction		1.0L
Effective	ly held in position at bo	th ends and restrained in o	lirection at one end		0.85L
Effective	y held in position and p	partially restrained in direc	tion at both ends		0.85L
Effective	y held in position and i	restrained in direction at b	oth ends		0.7L
		restrained in direction at or but not held in position	ne end with the other	er end	1.2L
	y held in position and r restrained in direction b	restrained in direction at or out not held in	ne end with the othe	er end	1.5L
	ly held in position and or or restrained in direction	restrained in direction at or n at the other end	ne end position but	not held ir	2.0L
ole		University of	f Moratuv	va, S	ri Lanka.
9		Electronic T			
	Marin Co.	www.lib.mrt		2000	
Materia	Properties	E E	- 205.0 k	N/mm <sup>2</sup>	
		$P_{y}$	- 280.0 N		
	TYPE OF SECTION	DN	- Double C sect	ion	
		[2/020	D 200 x 2.0 0x75x20 -Back to	n Back D	ouble Section1
		[2/ 0200	7X7 3X20 -Dack to	Dack D	ouble Section]
	ection Property 13.46 cm <sup>2</sup>				
Α .	797.3 cm				
l <sub>xx</sub> -	106.4 cm				
l <sub>YY</sub> -	200 mm				
t .	200 mm 2 mm				
r <sub>xx</sub>	7.70 cm				
r <sub>yy</sub> _	2.81 cm				
b <sub>2</sub> .	150 mm	1			
b <sub>3</sub> .	20 mm				
~5 -	20 11111				
Effective	e Length (L <sub>E</sub> )		_ 1.010 m	1	

	DESIGN CALCULATION	Ву	Date	
	PART : Cold form -12.0m Span Truss-Top chord	BNC	2014-03-24	
	Members in Compression  Slenderness ratio $\left(\frac{L_E}{r_{xx}}\right)$ - $\left(\frac{L_E}{r_{yy}}\right)$ -  Maximum Slenderness limit  1) For members resisting loads other than wind load  2) For members resisting self weight and wind load  3) For members acting as ties with reversal stressing  load pattern with respect to Max.Slenderne	13.12 35.92 - - - - ess limit -	180 250 350 2	Max. slenderness OK
Cl 6.2.3	Singly Symetrical Section			
Annex B	Effective Cross-sectional area  a) Effective Breadth of Webletonent of Model Effective Breadth of Webletonent of Breadth of Breadth of Webletonent of Breadth of Brea	s & Dissert k 280 - 4.88		
	$h = \frac{b_2}{b_1}$ $b_1 - 198.0 \text{ mm}$ $b_2 - 150.0 \text{ mm}$ $t_1 - 1.96 \text{ mm}$ $t_2 - 1.96 \text{ mm}$ $P_{cr} = 0.904EK_1 \left(\frac{t}{b}\right)^2$ $\frac{f_c}{P_{cr}}$	- 0.76 - 88.54 - 3.16	N/mm² > 0.123	
	For $\frac{f_c}{P_{cr}} < 0.123; \qquad \frac{b_{eff}}{b} = 1$ For $\frac{f_c}{P_{cr}} > 0.123 \qquad \frac{b_{eff}}{b} = \left[1 + 14\left\{\left(\sqrt{\frac{f_c}{P_{cr}}}\right)\right\}\right]$	$\left -0.35\right ^4$		

DESIGN CALCULATION		Ву	Date
ART : Cold form -12.0m Span Truss-Top chord		BNC	2014-03-24
$rac{b_{_{e\!f\!f}}}{b}_{_{e\!f\!f}}$	-	0.442	
$\overset{b}{b}_{_{\mathit{aff}}}$	-	87.52	mm
e,j,)			
b) Effective Breadth of Flange Element			
· · · · · · · · · · · · · · · · · · ·			
$K_2 = K_1 h^2 \left(\frac{t_1}{t_2}\right)^2$ $P_{cr} = 0.904 EK_2 \left(\frac{t}{b}\right)^2$	-	2.80	
$P = 0.904 E K_{2} \left(\frac{t}{t}\right)^{2}$		126.6	N/mm²
(cr)	-	126.6	N/IIIII
$f_c$		2.212	> 0.123
$rac{f_c}{P_{cr}}$		Z.Z.1Z	0.123
		0.536	
$rac{b_{e\!f\!f}}{b}$	-	0.528	
$\stackrel{\circ}{b}_{_{\it eff}}$	-	79.16	mm
$\begin{array}{ccc} & & & & & & & & \\ & & & & & & & \\ & & & & & & \\ & & & & & & \\ & & & & & \\ & & & & & \\ & & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & & & \\ & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ & & \\ &$		24444	kN kN
Buckling Resistance Under Axial Load for section symm Closed section ${\rm P_c}  =  \left(\frac{P_E P_{CS}}{\phi + \sqrt{\phi^2 - P_E P_{PC}}}\right)$ ${\rm P_C}$	etrical al	205 46	s or kN
Determination Of Moment Capacity  a) Limiting Stress for Stiffened Web in Bend	$\left\{ \mathcal{P}_{y} ight.$		

	DESIGN CALCULATION	Ву	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 $M - p \times Z$	- 19.55	4 kNm	
	$M_{cx} = p_0 \times Z_{xr}$			
	ii) Moment Capacity about Y-Y axise outstand in tension			
	$M_{cy} = p_0 \times Z_{y1r}$ outstand in compression	- 4.29	3 kNm	
	$M_c = p_0 \times Z_{y2r}$	- 4.29	3 kNm	
	Where			
		tion modulus for major		
	axise bendin Z <sub>y1r</sub> - Reduced sec	g tion modulus for minor		
	,	g (outstand in tension)		
	,	tion modulus for minor		
	axise bendin	g (outstand in compres	sion)	
Cl 6.2.4				
	University of M Buckling Resistance Under Axial Load for Single symmetry	loratuwa, Sri J	_anka.	
	Electronic Thes	es & Disseria	tions	
	i) P tabout Y-W Outstand in tension ac	.lk		
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c}P_{c}}$	- 265.4	6 kN	
	$P'_c = \frac{M_c P_c}{M_c + P_c e_s}$			
	ii) P' <sub>c</sub> (about Y-Y) Outstand in compression	on		
	. <i>M P</i>	- 265.4	6 kN	
	$P'_{c} = \frac{M_{c}P_{c}}{M_{c} + P_{c}e_{s}}$	203.4	O KIV	
	Basic Requirment $F_c < (P'_c)_{min}$ Applied Axial Load-Compression		- c	
	F <sub>C</sub>	- 145.5		Buckling Resistance
Cl 6.4				ок
	Combined bending and Compression			
	Applied Bending Moment about X-X Axis $\mathrm{M}_{\mathrm{x}}$	- 0.12	7 kNm	
	Applied Bending Moment about Y-Y Axis	-1		
	M <sub>y</sub>	- 0.00	0 kNm	
CI 6.4.2				
	Local Capacity Check $oldsymbol{F} oldsymbol{M}$			
	Local Capacity Check   $\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le$	≤1		
	$P_{cs} = M_{cx} = M_{cy}$			
				I

Cl 3.5.4  Net corss sectional area  Net corss sectional area  Net area after deduction in 30.4.6.3.—16.—26.  Net cors sectional area of section in this in 30.4.4.3		DESIGN CALCULATION By	Date	
Net corss sectional area  Net corss sectional area  Tital of the bedres read $x$ gauge equation in $x$ gauge from the $x$ gaug		PART : Cold form -12.0m Span Truss-Top chord BNC	2014-03-24	
Number of Bolt Holes  Number of Bolt Holes  Number of Bolt Holes  Hole Pattern  Hole Diameter  University of Moratuwa, Sri Lanka.  Electronic Thèses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  Tension Capcity $P_i = A_e p_y$ 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 $A_c = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a <sub>1</sub> Net sectional area of connected leg  a <sub>2</sub> Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_c = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ a <sub>1</sub> as above  a <sub>2</sub> as above  a <sub>1</sub> 717.33 mm²		0.529 + 0.006 + 0.000 -	0.536	
Not around addiction in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around after detection in 0.3.4.500 = $M - 501$ The around a many and are around after detection in 0.3.4.500 = $M - 501$ The around a many after detection in 0.3.4.500 = $M - 501$ The	Cl 3.5.4	Net corss sectional area	A <sub>net</sub>	
Set area after declaration in 3.3.4.5 $\alpha$ ) = $\delta t - 5dt$ (set area after declaration in 3.3.4.5 $\alpha$ ) = $\delta t - (5dt - \frac{86/2}{4g})$ Figure 1 — Nomenrelature for staggered holes with example  Number of Bolt Holes  Holes Pattern  Zig. Zag Hole Diameter  University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  Tension Capcity $P_t = A_t p_y$ 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 $A_r = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a1 Net sectional area of connected leg a2 Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_z = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ $a_1$ as above $a_2$ as above $a_1$ — 717.33 mm²		5 holes in line Total of 9 holes and 8 gauge space	es in zig-zag line	
Niel after abduction is $0.3.4.5a_1 = 4c - 5d$ . Not area after devalution in $0.3.4.5a_1 = 4c - (ast - \frac{86c^2}{4g^2})$ .  Figure 1— Nomencelature for straggered holes with example.  Number of Bolt Holes  Holes Pattern  Zig. Zag Hole Diameter  University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  Tension Capcity $P_i = A_r p_y$ 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 $A_r = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a <sub>1</sub> . Net sectional area of connected leg a <sub>2</sub> . Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_z = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ a <sub>1</sub> . as above  a <sub>2</sub> . as above  a <sub>1</sub> . 717.33 mm²				
Set area after declaration in 3.3.4.5 $\alpha$ ) = $\delta t - 5dt$ (set area after declaration in 3.3.4.5 $\alpha$ ) = $\delta t - (5dt - \frac{86/2}{4g})$ Figure 1 — Nomenrelature for staggered holes with example  Number of Bolt Holes  Holes Pattern  Zig. Zag Hole Diameter  University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  Tension Capcity $P_t = A_t p_y$ 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 $A_r = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a1 Net sectional area of connected leg a2 Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_z = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ $a_1$ as above $a_2$ as above $a_1$ — 717.33 mm²				
Net area after decluction in 9.5.4.50a $= bc - 5dt$ . Not area after the intrins in 13.4.510 $= bc - (bat - \frac{86.71}{4gt})$ Pigure 1 — Nomerical targeties for staggered holes with example.  Number of Bolt Holes Holes Pattern Hole Diameter  University of Moratuwa, Sri Lanka. Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  Tension Capcity $P_t = A_c P_y$ 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 $A_c = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a <sub>1</sub> Net sectional area of connected leg a <sub>2</sub> - Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_c = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ a <sub>1</sub> - as above a <sub>2</sub> - as above a <sub>1</sub> - 717.33 mm²			-	
Niet area after declicition in 3.5.4.5kg) = 6t - 76tt Ned series after declicition in 3.5.4.5kg) = $\alpha - \left( \cos t - \frac{8 e R^2}{4 g} \right)$ Figure 1 — Nomenclature for staggered holes with example  Number of Bolt Holes Holes Pattern Hole Diameter  University of Moratuwa, Sri Lanka. Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  Cl 7.2.2  Tension Capcity $P_r = A_r p_y$ 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 $A_r = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a <sub>1</sub> . Net sectional area of connected leg a <sub>2</sub> . Gross sectional area of unconnected leg or legs  Cl 7.2.3  b) If two component are parellel back to back $A_r = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ $a_1$ . as above $a_2$ . as above $a_3$ . as above $a_4$ . T77.33 mm²			-	
Net arou after deduction in 3.5.4.5%) = 6t - 5tt   Net arou after deduction in 3.5.4.5%) = 6t - 5tt   Net arou after deduction in 3.5.4.5%) = 6t - $(ast - \frac{3s_0 \pi}{4g})$   Figure 1 — Nomenclature for staggered holes with example   Number of Bolt Holes   1   3   4.5%   1.5%				
Number of Bolt Holes  Number of Bolt Holes  Holes Pattern  Hole Diameter  University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations mm²  Www.lib.mrt.ac.lk  Cl 7.2.2  Tension Capcity $P_i = A_e P_y$ 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 $A_c = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a1		/	$t - \left(3dt - \frac{8s_n^2t}{4n}\right)$	
Holes Pattern Hole Diameter  University of Moratuwa, Sri Lanka. Electronic Theses & Dissertations mm²  Www.lib.mrt.ac.lk  Cl 7.2.2  Tension Capcity $P_t = A_x p_y$ 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 $A_c = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a <sub>1</sub> . Net sectional area of connected leg a <sub>2</sub> . Gross sectional area of unconnected leg or legs  Cl 7.2.3  b) If two component are parellel back to back $A_c = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ a <sub>1</sub> . as above a <sub>2</sub> . as above a <sub>2</sub> . as above a <sub>3</sub> . T17.33 mm²			( 19/	
University of Moratuwa, Sri Lanka.  Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  C17.2.2  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 $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ a1  Net sectional area of connected leg  a2  Gross sectional area of unconnected leg or legs  C17.2.3  b) If two component are parellel back to back $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ a1  as above  a2  as above  a3  T17.33 mm²				
University of gMoratuwa, Sri Lanka. Electronic Theses & Dissertations mm² www.lib.mrt.ac.lk  Cl 7.2.2  Tension Capcity $P_i = A_e p_y$ 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 $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ $a_1  \text{Net sectional area of connected leg}$ $a_2  \text{Gross sectional area of unconnected leg or legs}$ Cl 7.2.3  b) If two component are parellel back to back $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_1  \text{as above}$				
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 $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ and $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ and $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ b) If two component are parellel back to back $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ and $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ and $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ and $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$		Electronic Theses & Dis	, Sri Lanka.	
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 $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ and $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ and $A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ b) If two component are parellel back to back $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ and $A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$		Members in Tension WWW.lib.mrt.ac.lk		
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 $A_{\epsilon} = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$ $a_1 \qquad \text{Net sectional area of connected leg}$ $a_2 \qquad \text{Gross sectional area of unconnected leg or legs}$ CI 7.2.3 b) If two component are parellel back to back $A_{\epsilon} = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$ $\frac{a_1}{(5a_1 + a_2)} = \frac{a_1}{(5a_1 + a_2)}$ $\frac{a_1}{(5a_1 + a_2)} = $	Cl 7.2.2			
For plain channel section connected only through web For "T" sections connected only through the flange $A_e = \frac{a_1(3a_1+4a_2)}{(3a_1+a_2)}$ $a_1  \text{Net sectional area of connected leg}$ $a_2  \text{Gross sectional area of unconnected leg or legs}$ CI 7.2.3 b) If two component are parellel back to back $A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_1  \text{as above}$ $a_1  \text{as above}$ $a_1  \text{as above}$ $a_1  \text{as above}$		Tension Capcity $P_{t} = A_{e} p_{y}$		
$A_e = \frac{a_1(3a_1+4a_2)}{(3a_1+a_2)}$ $a_1  \text{Net sectional area of connected leg}$ $a_2  \text{Gross sectional area of unconnected leg or legs}$ $\text{CI 7.2.3}$ $\text{b) If two component are parellel back to back}$ $A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_1  \text{as above}$		For plain channel section connected only through web		
cl 7.2.3 b) If two component are parellel back to back $A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ and $a_1$ as above $a_2$ as above $a_1$ as above $a_1$ as above $a_2$ as above $a_1$ as above $a_2$ as above $a_1$ as above $a_2$ as above $a_1$				
cl 7.2.3 b) If two component are parellel back to back $A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  -  \text{as above}$ $a_2  -  \text{as above}$ $a_1  -  717.33 \text{ mm}^2$		$A_e = \frac{a_1(3a_1 + 4a_2)}{(3a_1 + a_2)}$		
CI 7.2.3 b) If two component are parellel back to back $A_e=\frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  -  \text{as above}$ $a_2  -  \text{as above}$ $a_1  -  717.33 \text{ mm}^2$		a <sub>1</sub> . Net sectional area of connected leg		
b) If two component are parellel back to back $A_e=\frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1 = as \ above$ $a_2 = as \ above$ $a_1 = 717.33 \ \text{mm}^2$		a <sub>2</sub> _ Gross sectional area of unconnected leg or le	egs	
$A_e = \frac{a_1(5a_1+6a_2)}{(5a_1+a_2)}$ $a_1  \text{as above}$ $a_2  \text{as above}$ $a_1  \text{-}  717.33 \text{ mm}^2$	CI 7.2.3			
$a_1$ _ as above $a_2$ _ as above $a_1$ _ 717.33 mm $^2$		b) If two component are parellel back to back		
a <sub>2</sub> _ as above $a_1 = 717.33 \; \text{mm}^2$		$A_e = \frac{a_1(5a_1 + 6a_2)}{(5a_1 + a_2)}$		
a <sub>1</sub> _ 717.33 mm <sup>2</sup>		a <sub>1</sub> _ as above		
a <sub>2</sub> _ 292.00 mm <sup>2</sup>		a <sub>2</sub> _ 292.00 mm <sup>2</sup>		

DESIC PART	GN CALCULAT  : Cold form -1		Truss-Top chord	By BNC	Date 2014-03-24	
	Connection t		,			
	Connection	гуре	-	b)		
			A <sub>e</sub> _	987.35 mm <sup>2</sup>		
			p <sub>t</sub> _	276.46 kN		
3 Meml	bers in Combin	ed Bendir	ng & Tension			
	Applied Tens	sile Load	F <sub>t</sub>	-	1.147 kN	
	Applied Bend	ding Mome	ent about X-X Axis	3		
			~	-	0.084 kNm	
	Applied Bend	ding Mome	ent about Y-Y Axis M <sub>y</sub>	-	0.000 kNm	
			·		O.OOO KIVIII	
	<u>1</u>	$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}}$	$\frac{1}{x} + \frac{M_{y}}{M_{cy}} \le 1$			
	0.00 -	+ 0.00	+ 0.00	-	0.01	Local Capacity
						ОК
		Elect	ronic Thes	ses & Diss	Sri Lanka. sertations	
		Elect		ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		
		Elect	ronic Thes	ses & Diss		

## C.2 DESIGN OF HOT ROLLED SECTION



	DESIGN CALCULATION PART: Hot Rolled Section -10.0m Sp	pan Truss-Bott	om chord		By BNC	Date 2014-03-24	
Cl 4.7	Design of Members						
	_	- Compressior - Tension	1	-ve +ve			
	Steel Grade	- 275	5	N/mm <sup>2</sup>			
	E	- 205000	)	N/mm <sup>2</sup>			
	Analysis Sumary						
		1	Tension		Со	mpression	
	Member Assignment	2L3X3X1/4 Double Angl	e		2L3X3X1/ Double Ar		
	Frame Text Axial Forces	49	LAI		50	LAI	
	Axiai Forces M <sub>xx</sub>	139.62 0.241	kN kNm		113.40 -0.106	kN kNm	
	M <sub>yy</sub>	0.000	kNm			kNm	
	Unrestrain Length	1.667	m		1.667	m	
		•					
	Section Propeties  Cross Section Area	Α	_		1852	mm <sup>2</sup>	
	Second Moment of Inertia	I <sub>xx</sub>	-		1024000		
		I <sub>yy</sub>	-		1883000		
	Plastic section Modulus Universi		oratuw es & Di	a, Sri issert	L33000 L311k 40000 ations	mm <sup>3</sup> mm <sup>3</sup> mm <sup>3</sup>	
	www.lib	.int.ac.	lk -		25000		
	Radius of gyration	$R_{xx}$	-		23.51	mm	
		$R_{yy}$	-		31.89	mm	
	For single Angle / Double Angle on	-					
		t <sub>w</sub>	-		6.35		
		t <sub>f</sub>	-		6.35		
		t <sub>3</sub>	-		76.20 152.40		V ×9
		t <sub>2</sub> Y	-		11.5		^
		X	-		0.00		
	Product of Moment of Inerti	ia I <sub>xy</sub>	-			mm <sup>4</sup>	
		I <sub>max(u-u)</sub>	_		0	mm <sup>4</sup>	
		I <sub>min(v-v)</sub>	_			mm <sup>4</sup>	
		R <sub>uu</sub>	_		0.00		
		R <sub>vv</sub>	-		0.00		
CL 2.F.1							
Cl 3.5.1	Member Loacal Buckling Check						
		$\frac{b}{a}$	-		11.75		
		d					
		$\frac{a}{t}$	-		12		
		ε	-		1		
					_		

	DESIGN CALCULATION			Ву	Date	
	PART : Hot Rolled Section -10.0m S	Span Truss-Bottom	chord	BNC	2014-03-24	
		,				
Table 11		15. ε		15		
				Section is No	n Slender	
	Design of Compression Members					
Cl 4 7 40 0		3		70.62		
Cl.4.7.10.2	D : 6: 11 (6: 1	λ	-	79.62 <sub>275</sub> N/	mm²	
	Design Strength of Steel	p <sub>y</sub>	-			
		$\lambda_0$	-	17.15		
		η -	-	0.34	2	
		P <sub>E</sub>	-	319.196 N/	mm	
		ф	-	351.93	2	
		$p_c$	-	162.00 N/	mm <sup>-</sup>	
	Compression Resistance					
	compression resistance					
Cl.4.7.4		$P_c$	-	$A_g p_c$		
			-	270.02 kN >	113.398 kN	ок
	$F_t$	$+\frac{M_x}{Z_x}+\frac{M_y}{Z_y}$	< n			
Annex: I.3	$A_{_{e}}$	$Z_x Z_y$	$= P_y$			
		•				
	61.2 + 5.55 +	3.22E-13	_	66 70 /	275 kN/mm2	ОК
	01.2 + 3.33 +	3.22E-13	-	00.78 <	273 KN/IIIII2	OK
	Univers	ity of Mor	atuwa	, Sri Lanka.	7	
Cl 3.4.1	Design of Tension Members Electron	io Thomas	0. D.	acortations	·	
	Electron	ne Theses	& DI	sseriations		
	Net corss sectional area	o.mrt.ac.lk	-	A <sub>net</sub>		
	5 holes in line	Tota	al of 9 holes	and 8 gauge spaces in zi	g-zag line	
		,	/			
		<i></i>	<u></u>			
		- <del></del>	= $d$	{		
	<b>-</b> ── \					
	b	<u>-&gt;</u>		_ <del>-</del>		
	5		s	1		
	-	- <del>Φ</del>	g	\		
	J	A-:->				
	- L+ / Y	+ +				
		<i>S</i> <sub>p</sub> −				
	Net area after deduction in $3.5.4.5a$ ) = $b$	bt - 5dt Net area af	ter deductio	on in $3.5.4.5b$ ) = $bt - 9dt$	$-\frac{8s_p^2t}{4}$	
		nclature for stagge			4g )	
	Number of Bolt Holes	;	-	3		
	Holes Pattern		-	Zig-Zag		
	Hole Diameter		-	8.00 mm		
	s <sub>p</sub>		-	20		
	g		-	60		
	A <sub>net</sub>		-	352.64 mr	n <sup>2</sup>	

DESIC PART	ON CALCULAT	TON Section -10.0m Span Truss-Bott	om chord	By BNC	Date 2014-03-24	
IANI				DITC	2014 03-24	
		Net sectional area of con	nected leg			
	a <sub>2</sub> .	$A_g - a_1$			2	
		a <sub>1</sub>	-	352.64 mm 1280.05 mm		
		$A_{e}$	-	1499.36	'	
		$a_2$	-	1499.56		
Cl 4.6.3		nection Type	-	Bolt connection	on	
		e Angle - For bolt connection P <sub>t</sub>	-	p <sub>y.</sub> (A <sub>e</sub> -0.5a <sub>2</sub> )		
		- For weld connection P <sub>t</sub>	_			
				. , 6		
		le Angle		/		
		- For bolt connection P <sub>t</sub>		. ,		
		- For weld connection $P_t$	-	p <sub>y.</sub> (A <sub>g</sub> -0.15a <sub>2</sub> )		
		$P_t$	-	248.93 kN		
				>	139.615 kN C	K
		Electronic These www.lib.mrt.ac.		ertations		

	DESIGN CALCULATION PART: Hot Rolled Section -10.0m	Span Truss-B	racing Member	By BNC	Date 2014-03-24	
Cl 4.7	Design of Members					
	Sign convention	- Compress	ion	-ve		
		- Tension		+ve		
	Steel Grade	- 2		N/mm <sup>2</sup>		
	E	- 2050	000	N/mm <sup>2</sup>		
	Analysis Sumary					
			Tension	Comp	ression	
	Member Assignment	2L2X2X1/- Double Ar		2L2X2X1/4 Double Angle		
	Frame Text	45		60		
	Axial Forces	63.17	kN	-40.66 kN		
	M <sub>xx</sub>	0.000	kNm	-0.146 kN	m	
	M <sub>yy</sub>	0.000	kNm	0.000 kN	m	
	Unrestrain Length	2.700	m	2.453 m		
	Section Propeties					
	Cross Section Area	Α	_	1219 mr	$n^2$	
	Second Moment of Inertia	I <sub>xx</sub>	-	287600 mr		
		I <sub>yy</sub>	_	564700 mr	$n^4$	
	Plastic section Modulus	S <sub>xx</sub>	_	14000 mr		
	Univer		Joratuwe	, Sri Land mr	$n^3$	
	Section Modulus Electro	ni¢ The	ses & Di	ssertations mr	n <sup>3</sup>	
	Radius of gyration WWW.li	b.mrt.a	c.lk	15.36 mr		
	Reduce of Spiration	R <sub>yy</sub>	-	21.52 mr		
	For single Angle / Double Angle	only				
		t <sub>w</sub>	-	6.35 mr	n	
		t <sub>f</sub>	-	6.35 mr	n	
		$t_3$	-	50.80 mr	n	
		t <sub>2</sub>	-	101.60 mr		X 🛪
		Υ	-	8.3 mr		Y
		Χ	-	0.00 mr		
	Product of Moment of Ine	rtia I <sub>xy</sub>	-	0 mr	n <sup>4</sup>	
		1		0 mr	$n^4$	
		I <sub>max(u-u)</sub>	-	0 mr		
		I <sub>min(v-v)</sub>	-			
		R <sub>uu</sub>	-	0.00 mr		
		$R_{vv}$	-	0.00 mr	n	
Cl 3.5.1						
	Member Loacal Buckling Check	h				
		$\frac{b}{t}$	-	7.75		
		$\stackrel{\iota}{d}$		0		
		$\overline{t}$	-	8		
		ε	-	1		

	DESIGN CALCULATION PART: Hot Rolled Section -10.0m S	nan Truss-Bracing Member	By Date BNC 2014-03-24	
	PART : Hot Rolled Section -10.011 3	pair it uss-bracing Member	DIVC 2014-03-24	
Table 11		15. ε	15	
			Section is Non Slender	
	Design of Compression Members			
Cl 4 7 40 2			161.10	
Cl.4.7.10.2	Design Strength of Steel	λ -	161.19 <sub>275</sub> N/mm²	
	Design strength of steel	$p_y$ - $\lambda_0$ -	17.15	
		η -	0.79	
		P <sub>E</sub> -	77.875 N/mm <sup>2</sup>	
		ф -	207.28	
		p <sub>c</sub> -	60.48 N/mm <sup>2</sup>	
	Compression Resistance			
Cl.4.7.4		P <sub>c</sub> -	$A_gp_c$	
	_	- M	66.35 kN > 40.656 kN	ОК
Annex: I.3	$\frac{F_t}{\Lambda}$	$+\frac{M_x}{Z_x} + \frac{M_y}{Z_y} \le p_y$		
Allilex: 1.5	$A_e$	$\mathbf{Z}_{x}$ $\mathbf{Z}_{y}$		
	33.4 + 18.3 +	2.70E-14 -	51.63 < 275 kN/mm2	ОК
		o.mrt.ac.lkal of 9 holes a	sertations	
		nclature for staggered holes w	rith example	
	Net area after deduction in 3.5.4.5a) = b Figure 1 — Nome  Number of Bolt Holes	t-5dt Net area after deduction nclature for staggered holes w	rith example	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern	t-5dt Net area after deduction nclature for staggered holes w	atth example  3  Zig-Zag	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter	t-5dt Net area after deduction nclature for staggered holes w	rith example	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern	t-5dt Net area after deduction nclature for staggered holes w	atth example  3  Zig-Zag  8.00 mm	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter  Sp	t – 5dt Net area after deduction nclature for staggered holes w - - - -	ith example  3 Zig-Zag 8.00 mm 20	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter  Sp g	t – 5dt Net area after deduction nclature for staggered holes w - - - -	7th example  3 Zig-Zag 8.00 mm 20 60	

DESIG	GN CALCULATION		Ву	Date	
PART	: Hot Rolled Section -10.0m Span Truss-	Bracing Member	BNC	2014-03-24	
	a <sub>1</sub> . Net sectional area of	f connected leg			
	$a_1$ . Net sectional area of	connected leg			
	$a_2 - b_2 = a_1$	-	191.35 r	mm²	
	$A_{e}$	<u>-</u>	796.18 r		
	$a_2$	-	1027.65		
	a <sub>2</sub>	-	1027.03		
l 4.6.3	Connnection Type	-	Bolt conne	ction	
	Single Angle		/4 0.5	,	
	- For bolt connection P <sub>t</sub>		•		
	- For weld connection P <sub>t</sub>	-	p <sub>y.</sub> (A <sub>g</sub> -0.3a <sub>2</sub>	2)	
	Double Angle				
	- For bolt connection P <sub>t</sub>				
	- For weld connection $P_t$	-	p <sub>y.</sub> (A <sub>g</sub> -0.15a	a <sub>2</sub> )	
	D		140 20 1	·NI	
	$P_{t}$	-		> 63.173 kN	OK
			ĺ	03.173 KIV	OK

	DESIGN CALCULATION PART: Hot Rolled Section -10.0m S	pan Truss-To	p chord		By BNC	Date 2014-03-24	
Cl 4.7	Design of Compression Members						
	_	- Compressio	on	-ve			
	Steel Grade	- Tension - 27	75	+ve N/mm <sup>2</sup>			
	E	- 20500		N/mm <sup>2</sup>			
	Analysis Sumary						
		1	Tension		Co	mpression	
	Member Assignment	2L3X3X1/4 Double Ang			2L3X3X1/4 Double Ar	4	
	Frame Text	56	,		54	.B.c	
	Axial Forces	-95.28	kN		-158.55	kN	
	$M_{xx}$	-0.286	kNm		0.151	kNm	
	M <sub>yy</sub>	0.000	kNm			kNm	
	Unrestrain Length	1.894	m		1.894	m	
	Section Propeties						
	Cross Section Area	Α	-		1852	mm <sup>2</sup>	
	Second Moment of Inertia	I <sub>xx</sub>	-		1024000	mm <sup>4</sup>	
		l <sub>yy</sub>	-		1883000	mm <sup>4</sup>	
	Plastic section Modulus Universion Medulus Electron		loratuw es & D	a, Sri issert	L33000 Lank 40000 atiggs	mm <sup>3</sup> mm <sup>3</sup> mm <sup>3</sup>	
	www.lib	.4mrt.ac	.lk -		25000		
	Radius of gyration	$R_{xx}$	-		23.51	mm	
		$R_{yy}$	-		31.89	mm	
	For single Angle / Double Angle or	nly					
		$t_w$	-		6.35	mm	
		$t_f$	-		6.35		
		t <sub>3</sub>	-		76.20	mm	
		$t_2$	-		152.40		X 🔻
		Υ	-		11.5		Y
	Product of Moment of Inerti	X	-		0.00	mm mm⁴	
	Product of Moment of merti	ia i <sub>xy</sub>	-		U		
		I <sub>max(u-u)</sub>	-		0	mm <sup>4</sup>	
		I <sub>min(v-v)</sub>	_			mm <sup>4</sup>	
		R <sub>uu</sub>	_		0.00		
		R <sub>vv</sub>	-		0.00		
Cl 3.5.1							
	Member Loacal Buckling Check	h					
		$\frac{b}{t}$	-		11.75		
		$\overset{\iota}{d}$			12		
		$\overline{t}$	-		12		
		ε	-		1		

	DESIGN CALCULATION			Ву	Date	
	PART : Hot Rolled Section -10.0m S	nan Truss-Ton chord	4	BNC	2014-03-24	
	The Noned Section 16.6115	pair riass rep enere	и	Dive	2011 03 21	
Table 11		15. ε		15		
				Section is	Non Slender	
	Design of Community Manusham					
Cl.4.7.10.2	Design of Compression Members	λ	_	86.39		
C1.4.7.10.2	Design Strength of Steel	p <sub>y</sub>	_		N/mm <sup>2</sup>	
	besign strength of steer	$\lambda_0$	_	17.15	,	
		η	_	0.38		
		P <sub>E</sub>	_	271.116	N/mm <sup>2</sup>	
		ф	_	324.68	,	
		p <sub>c</sub>	_	149.01	N/mm <sup>2</sup>	
		FC		1.5.01		
	Compression Resistance					
Cl.4.7.4		$P_c$	-	$A_g p_c$		
İ			- 248.	38 kN	> 158.554 kN	OK
Annex: I.3	$F_{t}$	$+\frac{M_x}{Z_x} + \frac{M_y}{Z_y} \le$	$\leq p$			
Alliex. 1.5	$A_{e}$	$Z_x$ $Z_y$	- P y			
	85.6 + 7.94 +	2.30E-12	-	93.55	< 275 kN/mm2	ОК
	Flectron www.lib	mrt.ac.lk		rtations B gauge spaces in		
					-	
	l		d	1		
	l → \   i		†	\	-	
	-Ψ		=			
	<b>→</b>   b   r-⊕-	<del>&gt;</del>			-	
			Ţ.s			
	- \ '	->0	g	\	-	
	J - <del>-</del>	Ψ.	_ <del>†</del>	)		
	- Lift				-	
		⊸ <sup>N</sup> p −				
	Net area after deduction in $3.5.4.5a$ ) = $bt$	t – 5dt – Net area after	r deduction in 9	3.5.4.5h) = ht _ [	$\left(9dt - \frac{8s_p^2t}{}\right)$	
				,	4g J	
	rigure 1 — Nomei	nclature for staggere	a notes with	example		
				•		
	Number of Bolt Holes		-	3 7ig 7ag		
	Holes Pattern Hole Diameter		-	Zig-Zag 8.00 mm		
	S <sub>p</sub>		_	20		
	g g		_	60		
	A <sub>net</sub>		_		mm <sup>2</sup>	
1	, whet			-201		

DESIG	ON CALCULATION	В	y Date	
PART	: Hot Rolled Section -10.0m Span Truss-Top	chord B	NC 2014-03-24	
	Not sectional area of se	unnected lea		
	$a_1$ . Net sectional area of co $a_2$ . $A_g$ - $a_1$	onnected leg		
		_	352.64 mm <sup>2</sup>	
	$egin{aligned} a_1 \ A_e \end{aligned}$	-	1280.05 mm <sup>2</sup>	
	<b>~</b> e a <sub>2</sub>		1499.36	
	a <sub>2</sub>	-	1455.50	
1 4.6.3	Connnection Type	- B	olt connection	
	Single Angle			
	- For bolt connection P <sub>t</sub>		<sub>y.</sub> (A <sub>e</sub> -0.5a <sub>2</sub> )	
	- For weld connection $P_t$	- p	$_{y.}(A_{g}-0.3a_{2})$	
	Double Angle			
	- For bolt connection P <sub>t</sub>	- p	<sub>y.</sub> (A <sub>e</sub> -0.25a <sub>2</sub> )	
	- For weld connection P <sub>t</sub>		$_{y.}(A_g-0.15a_2)$	
	•	·	y.	
	$P_{t}$	-	248.93 kN	
			> 95.284 kN	ОК
	University of M	Corotuvuo Cri 1	Lonko	
	Electronic Thes		tions	
	www.lib.mrt.ac	.lk		
<b>I</b>				

	DESIGN CALCULATION PART: Hot Rolled Section -12.0m	Span Truss-Bo	ottom chord	By BNC	Date 2014-03-24	
Cl 4.7	Design of Compression Members					
	Sign convention	- Compressi	on -ve	<b>.</b>		
	Sign convention	- Tension	+ve			
	Steel Grade	- 2		mm <sup>2</sup>		
	Е	- 2050	00 N/	mm <sup>2</sup>		
	Analysis Sumary					
		Tens	sion(Max.Forces)		npression in Forces)	
	Member Assignment	2L3X3X5/1	.6	2L3X3X5/1		
		Double An	gle	Double An	gle	
	Frame Text	687		682		
	Axial Forces	155.82	kN		kN	
	M <sub>xx</sub>	0.195	kNm		kNm	
	M <sub>yy</sub>	0.000 1.010	kNm m		kNm m	
	Unrestrain Length	1.010	m	1.010	m	
	Section Propeties					
	Cross Section Area	Α	-	2290	mm <sup>2</sup>	
	Second Moment of Inertia	I <sub>xx</sub>	-	1257000	mm <sup>4</sup>	
		l <sub>yy</sub>	-	2346000	mm <sup>4</sup>	
	Plastic section Modulus . Univer	sit <sup>S</sup> <sub>w</sub> of N	Ioratuwa,	Sri Lank:	mm <sup>3</sup>	
	A CANADA	ni <u>ę</u> The ib. <b>z</b> nrt.ac	ses & Diss	ertations 31000		
	Radius of gyration	R <sub>xx</sub>	-	23.43		
		R <sub>yy</sub>	-	32.01		
	For single Angle / Double Angle	only				
		$t_w$	-	7.94	mm	
		$t_f$	-	7.94	mm	
		t <sub>3</sub>	-	76.20	mm	
		t <sub>2</sub>	-	152.40	mm	× 🛪
		Υ	-	12.1		Y
		Χ	-	0.00		N/
	Product of Moment of Ine	rtia I <sub>xy</sub>	-	0	mm⁴	
		I <sub>max(u-u)</sub>	-	0	mm <sup>4</sup>	
		I <sub>min(v-v)</sub>	-	0	mm <sup>4</sup>	
		R <sub>uu</sub>	-	0.00	mm	
		R <sub>vv</sub>	-	0.00		
Cl 3.5.1						
	Member Loacal Buckling Check	b				
		$\frac{b}{t}$	-	9.3494		
		$\frac{d}{t}$	_	9.5994		
		$\overline{t}$	-	9.3334		
		3	-	1		

	DESIGN CALCULATION			Ву	Date	
	PART: Hot Rolled Section -12.0m S	pan Truss-Bottom	n chord	BNC	2014-03-24	
Table 11		15. ε		15 Section is I	Non Slender	
Cl.4.7.10.2	Design of Compression Members  Design Strength of Steel	$\lambda$ $p_y$	-	60.17 275	N/mm²	
		$\lambda_0$ $\eta$ $P_E$ $\Phi$	- - -	17.15 0.24 558.82 483.02	N/mm²	
	Compression Resistance	p <sub>c</sub>	-	200.83	N/mm <sup>2</sup>	
Cl.4.7.4 Annex: l.3	·	$P_{c} + \frac{M_{x}}{Z_{x}} + \frac{M_{y}}{Z_{y}}$	_	A <sub>g</sub> p <sub>c</sub> 413.91 kN	> 3.752 kN	ОК
	1.64 + 13.2 +	1.68E-13	-	14.81	< 275 kN/mm2	ОК
Cl 3.4.1	Design of Tension Members					
	Mondo	ic Theses	& Di	a, Sri Amanka ssertations s and 8 gauge spaces in		
			-   d		-	
			s g		-	
		S <sub>p</sub>			-	
	Net area after deduction in $3.5.4.5a$ ) = $bt$	t-5dt Net area a	fter deduct	ion in $3.5.4.5$ b) = $bt - ($	$9dt - \frac{8s_p^2t}{4g}$	
	Figure 1 — Nome	nclature for stagg	ered holes	with example		
	Number of Bolt Holes Holes Pattern Hole Diameter		-	3 Zig-Zag 8.0 mm		
	s <sub>p</sub>		- - -	20 60	mm²	
	A <sub>net</sub>			440.02		

	DESIGN	CALCULAT			Ву	Date	
	PART :	Hot Rolled	Section -12.0m Span Truss-Bott	om chord	BNC	2014-03-24	
			Net sectional area of con	nected leg			
		a <sub>2</sub>	$A_g - a_1$			m m <sup>2</sup>	
			a <sub>1</sub>	-	440.82		
			$A_e$	-	1587.56	mm	
			$a_2$	-	1849.18		
1 4.6.3		Conn	nection Type	-	Bolt conne	ection	
			e Angle				
			- For bolt connection P <sub>t</sub>		,		
			- For weld connection $P_t$	-	р <sub>у.</sub> (А <sub>g</sub> -0.3а	2)	
		Doub	le Angle				
			- For bolt connection P <sub>t</sub>	-	p <sub>y.</sub> (A <sub>e</sub> -0.25	a <sub>2</sub> )	
			- For weld connection P <sub>t</sub>	-			
					,, ,		
			$P_{t}$	-	309.45	kN	
					:	> 155.817 kN	OK
		A CONTRACTOR OF THE PARTY OF TH	Electronic These www.lib.mrt.ac.		301 tativitS		

	DESIGN CALCULATION PART: Hot Rolled Section -12.0m Sp	pan Truss-Bra	cing Member	By BNC	Date 2014-03-24	
Cl 4.7	Design of Compression Members					
	Sign convention	- Compressio	n -ve	2		
		- Tension	+ve			
	Steel Grade	- 27		mm² mm²		
	E	- 205000	) N/	mm		
	Analysis Sumary					
			Tension	Compr	ession	
	Member Assignment	L3X2X1/4		L3X2X1/4		
	Frame Text	Angle 681		Angle 679		
	Axial Forces	48.59	kN	-47.60 kN		
	$M_{xx}$	0.217	kNm	0.011 kNr	n	
	M <sub>yy</sub>	0.000	kNm	0.000 kNr	n	
	Unrestrain Length	1.032	m	1.032 m		
	Section Propeties					
	Cross Section Area	Α	_	768 mm	12	
	Second Moment of Inertia	I <sub>xx</sub>	-	453700 mm		
		I <sub>yy</sub>	_	163200 mm		
	<b>Pl</b> astic section Modulus	S <sub>xx</sub>	-	16000 mm		
	LIniversi	twof M	oratuwa	Sri Laffka."	1 <sup>3</sup>	
	Section Modulus Electron	iç These	es & Diss	ertations mm	1 <sup>3</sup>	
	Radius of gyration WWW.lib	enrt.ac.	lk -	24.31 mm		
	3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3, 3	R <sub>yy</sub>	-	14.58 mm		
	For single Angle / Double Angle on	nlv				
		t <sub>w</sub>	-	6.35 mm	1	
		$t_f$	-	6.35 mm	1	
		$t_3$	-	76.20 mm	1	
		t <sub>2</sub>	-	50.80 mm		X 🛪
		Υ	-	25.2 mm	1	Ϋ́Υ
		Χ	-	12.50 mm		W
	Product of Moment of Inerti	ia I <sub>xy</sub>	-	235762 mm	ı <sup>4</sup>	
		I <sub>max(u-u)</sub>	-	585364 mm	1 <sup>4</sup>	
		I <sub>min(v-v)</sub>	-	31536.4 mm	14	
		$R_{uu}$	-	27.61 mm	1	
		$R_{vv}$	-	6.41 mm	1	
Cl 3.5.1						
	Member Loacal Buckling Check	b		2		
		$\frac{b}{t}$	-	8		
		$\underline{d}$	_	12		
		t		12		
		ε	-	1		
<u> </u>						

	DESIGN CALCULATION	By Date
	PART : Hot Rolled Section -12.0m Span Truss-Bracing Member	-
Table 11	15. ε	15 Section is Non Slender
Cl.4.7.10.2	Design of Compression Members $\begin{array}{cccccccccccccccccccccccccccccccccccc$	136.89 275 N/mm <sup>2</sup> 17.15 0.66 107.98 N/mm <sup>2</sup> 227.04 79.21 N/mm <sup>2</sup>
Cl.4.7.4	Compression Resistance P <sub>c</sub> -	A <sub>g</sub> p <sub>c</sub> 54.75 kN > 47.599 kN OK
Annex: I.3	$\frac{F_t}{A_e} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \le p_y$	
	62 + 1.26 + 1.37E-15 -	63.24 < 275 kN/mm2 OK
CI 3.4.1	Net area after deduction in 3.5.4.5a) = $bt - 5dt$ Net area after deduction. Figure 1 — Nomenclature for staggered holes. Number of Bolt Holes. Holes Pattern	Assertations  Is and 8 gauge spaces in zig-zag line  Solution in 3.5.4.5b) = $bt - \left(9dt - \frac{8s_p^2t}{4g}\right)$ So with example  3  Zig-Zag
	Hole Diameter - $s_p \qquad \qquad -$ $g \qquad \qquad -$ $A_{net} \qquad \qquad -$	8.0 mm 20 60 352.64 mm <sup>2</sup>
	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	

	SN CALCULAT			Ву	Date	
PART	: Hot Rolled	Section -12.0m Span Truss-Brac	ing Member	BNC	2014-03-24	
	2	Not costional area of som	nastad las			
		Net sectional area of con $A_g - a_1$	nected leg			
	a <sub>2</sub>	-		352.64 m	um²	
		a <sub>1</sub>	-	634.89 m		
		$A_e$	-		1111	
		$a_2$	-	415.36		
1 4.6.3	Conn	nection Type	-	Bolt connec	tion	
		e Angle				
		- For bolt connection $P_t$	-	p <sub>y.</sub> (A <sub>e</sub> -0.5a <sub>2</sub> )		
		- For weld connection $P_t$	-	$p_{y.}(A_g-0.3a_2)$		
	Doub	ole Angle		/4 0.25	,	
		- For bolt connection P <sub>t</sub>		. ,		
		- For weld connection $P_t$	-	p <sub>y.</sub> (A <sub>g</sub> -0.15a <sub>g</sub>	2)	
		$P_t$	-	117.48 kľ	N	
					48.59 kN	ОК
	Barrio	Electronic These				
	The state of the s	www.hb.hht.ac.	k			
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	DESIGN CALCULATION PART: Hot Rolled Section -12.0m S	Span Truss-To	op chord		By BNC	Date 2014-03-24	
CI 4.7	Design of Compression Members						
	Sign convention	- Compressi	on	-ve			
		- Tension		+ve			
	Steel Grade		75	N/mm <sup>2</sup>			
	E	- 2050	00	N/mm <sup>2</sup>			
	Analysis Sumary						
			Tension		Co	mpression	
	Member Assignment	2L3X3X5/1 Double An			2L3X3X5/ Double Ai		
	Frame Text	702			696		
	Axial Forces	-1.47	kN		-154.40	kN	
	M <sub>xx</sub>	-0.020	kNm		0.001	kNm	
	M <sub>yy</sub>	0.000	kNm		0.000	kNm	
	Unrestrain Length	1.010	m		1.010	m	
	Section Propeties						
	Cross Section Area	Α	-		2290		
	Second Moment of Inertia	$I_{xx}$	-		1257000		
	Plastic section Modulus  Electron	nis, The	ses & I	va, Sri Disserta	atis0006	mm <sup>3</sup>	
	Section Modulus WWW.11	o. <del>I</del> mrt.ac	c.lk -		23000	_	
		$Z_{yy}$	-		31000	mm³	
	Radius of gyration	$R_{xx}$	-		23.43	mm	
		$R_{yy}$	-		32.01	mm	
	For single Angle / Double Angle o	nly					
		$t_w$	-		7.94	mm	
		$t_f$	-		7.94	mm	
		t <sub>3</sub>	-		76.20	mm	
		$t_2$	-		152.40	mm	X 🐙
		Υ	-		12.1		Y
		Χ	-		0.00		W. T.
	Product of Moment of Iner	tia l <sub>xy</sub>	-		0	mm <sup>4</sup>	
		I <sub>max(u-u)</sub>	_		0	mm <sup>4</sup>	
			_				
			_				
			_				
Cl 3.5.1		•••			0.00		
	Member Loacal Buckling Check	1					
		<u>v</u>	-		9.3494		
		_	-		9.5994		
			_		1		
		C	_		1		
Cl 3.5.1	Member Loacal Buckling Check	$I_{\max(u-u)}$ $I_{\min(v-v)}$ $R_{uu}$ $R_{vv}$ $\frac{b}{t}$ $\frac{d}{t}$	- - - -		0.00 0.00	mm	

	DESIGN CALCULATION			Ву	Date	
	PART : Hot Rolled Section -12.0m S	pan Truss-Top ch	ord	BNC	2014-03-24	
Table 11		15. ε		15		
				Section is	Non Slender	
	Design of Compression Members					
	Design of Compression Wembers					
Cl.4.7.10.2		λ	_	60.17		
	Design Strength of Steel	p <sub>y</sub>	_	275	N/mm <sup>2</sup>	
		$\lambda_0$	_	17.15		
		η	_	0.24		
		P <sub>E</sub>	_	558.82	N/mm <sup>2</sup>	
		ф	_	483.02	,	
		p <sub>c</sub>	_	200.83	N/mm <sup>2</sup>	
		Mc		200.03	,	
	Compression Resistance					
1.4.7.4	·	$P_c$	-	$A_g p_c$		
			-	413.91 kN	> 154.401 kN	ОК
	$F_{t}$	$+\frac{M_x}{Z_x}+\frac{M_y}{Z_y}$	- < n			
Annex: I.3	$\overline{A_e}$	$\overline{Z}_{x}$ $\overline{Z}_{y}$	$\leq p_y$			
	67.4	4 5 4 5 4 3		67.40	. 275   N/	014
	67.4 + 0.06 +	1.54E-12	-	67.48	< 275 kN/mm2	OK
	I Inivers	ity of Mor	otuvio	, Sri Lank	0	
Cl 3.4.1	Design of Tension Members	ity of Moi	atuwa	i, on Lank	d.	
5.4.1	Design of Tension Members Electron	nic Theses	& Di	ssertations		
	Net corss sectional area 11	met oo 11	_			
			_	Anet		
	record and affailt	J.IIII t.ac.ik	_	A <sub>net</sub>		
	5 holes in line			A <sub>net</sub> and 8 gauge spaces i	n zig-zag line	
					n zig-zag line	
					n zig-zag line	
					n zig-zag line	
			tal of 9 holes		n zig-zag line	
			tal of 9 holes		n zig-zag line	
	b holes in line		tal of 9 holes		n zig-zag line	
			tal of 9 holes		n zig-zag line	
	b holes in line		dal of 9 holes		n zig-zag line	
	b holes in line		al of 9 holes		n zig-zag line	
	b holes in line		dal of 9 holes		n zig-zag line	
	b holes in line		dal of 9 holes		n zig-zag line	
	b holes in line		dal of 9 holes		n zig-zag line	
	b holes in line	- W-	al of 9 holes	and 8 gauge spaces I	-	
	b holes in line  b holes in line  b  b  s  Not area after deduction in 3.5.4.5a) = b	- W-	al of 9 holes	and 8 gauge spaces i	-	
	b holes in line  b holes in line  b  b  s  Not area after deduction in 3.5.4.5a) = b	5p st – 5dt Net area a	al of 9 holes	and 8 gauge spaces i	-	
	Net area after deduction in $3.5.4.5a$ ) = $b$	s <sub>p</sub> t - 5dt Net area a nelature for stagg	al of 9 holes	and 8 gauge spaces in an in 3.5.4.5b) = $bt$ — with example	-	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome	s <sub>p</sub> t - 5dt Net area a nelature for stagg	al of 9 holes	and 8 gauge spaces in an in 3.5.4.5b) = $bt$ — with example	-	
	Net area after deduction in $3.5.4.5a$ ) = $b$	s <sub>p</sub> t - 5dt Net area a nelature for stagg	al of 9 holes	on in 3.5.4.5b) = bt — with example  3 Zig-Zag	-	
	Not area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter	s <sub>p</sub> t - 5dt Net area a nelature for stagg	al of 9 holes	on in 3.5.4.5b) = bt — with example  3 Zig-Zag 8.0 mm	-	
	Net area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter  Sp	s <sub>p</sub> t - 5dt Net area a nelature for stagg	al of 9 holes	on in 3.5.4.5b) = bt - with example  3 Zig-Zag 8.0 mm 20	-	
	Not area after deduction in 3.5.4.5a) = b  Figure 1 — Nome  Number of Bolt Holes Holes Pattern Hole Diameter	5p t - 5dt Net area a nelature for stagg	al of 9 holes	and 8 gauge spaces II  on in 3.5.4.5b) = bt -  with example  3  Zig-Zag  8.0 mm  20  60	-	

DESI	GN CALCULAT	TON		Ву	Date	
PART	: Hot Rolled	Section -12.0m Span Truss-Top	chord	BNC	2014-03-24	
		Net sectional area of cor	nnected leg			
	a <sub>2</sub>	$A_g - a_1$			2	
		$a_1$	-	440.82		
		$A_{e}$	-	1587.56	mm¯	
		$a_2$	-	1849.18		
1 4.6.3	Conn	nection Type	_	Bolt conne	ction	
1 4.0.5		e Angle		Boil conne	ction	
		- For bolt connection $P_t$	-	p <sub>y.</sub> (A <sub>e</sub> -0.5a	2)	
		- For weld connection $P_t$	-	p <sub>y.</sub> (A <sub>g</sub> -0.3a	2)	
		le Angle - For bolt connection P <sub>t</sub>		» (A O 2E	2.)	
		- For weld connection P <sub>t</sub>	-	p <sub>y.</sub> (A <sub>e</sub> -0.25 p <sub>y.</sub> (A <sub>g</sub> -0.15		
		- For well conflection F <sub>t</sub>	-	р <sub>у.</sub> (А <sub>g</sub> -0.13	a <sub>2</sub> )	
		$P_t$	-	309.45 l	kN	
		•			> 1.466 kN	ОК
	And with	Electronic These www.lib.mrt.ac.		ocitations		
	Control of State of S	www.mo.mit.ac.	117			
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## **ANNEX: D - COST EVALUATION**



**ROOF COVERING - 12m Span Roof Structure** 

	COVERING - 12m Span Roof Structure  Description	Qty.	Unit	Rate	Amount
No.				LKR	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)	
	The following requirements are to be noted and the rates shall cover same. Electronic These Fine Stable and uniformly extraded singletlayer 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	es & Diss lk	Sri serta	Lanka. tion <sub>fote)</sub>	
	equivalent.			(Note)	
	Sheets to be of non colour  Steel roof framework & lipped channels provided separately.			(Note)	
				(Note)	

## **ROOF COVERING - 12m Span Roof Structure**

	Description	Qty.	Unit		Amount
No.				LKR	LKR
1.2 1.2.1	Roof Covering  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 <sup>2</sup>	3,500.00	1,018,500.00
	University of M Electronic These www.lib.mrt.ac.	es & Diss			
	Bill No. 1 - Roof Covering & Insulation Total Carried to Summary				1,018,500.00

	CTURAL STEEL WORK - 12m Span Roof S			ı	1
Item	Description	Qty.	Unit		Amount
No.				LKR	LKR
2.0	Structural Steel Work				
2.1	<u>Notes</u>				
2.1.1	The Bidder shall refer the following prior to				
	pricing the items in this trade			(Note)	
	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 transportation to locations  f. Method of handing & installation.				
2.1.2					
2.1.2	Rates shall include for all required plant and machinery required for hoisting and erecting.			(NI-4-)	
	inactificity required for horsting and creeting.			(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,		. Sr	i Lanka.	
	bolts, nuts, cleats, haunches, etc (please refer method of payment)  Liectronic Thes	es & Di	ccer	tations.	
	method of payment	11-	DOCI	(Note)	
2.1.5	Where members of steel structures are	.IK			
	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)	
210				(11010)	
2.1.8	Butt welds which will be visible in the				
	completed works should be dressed off smooth and flushed with adjacent surface.			(Note)	
	smooth and musica with adjacent surface.			(INOIE)	

	CTURAL STEEL WORK - 12m Span Roof S		1	Ī	1
Item	Description	Qty.	Unit		Amount
No.				LKR	LKR
	<u>Fabrication</u>				
2.1.9	All materials shall be straight and if necessary				
2.1.9	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.			OI ( )	
2 1 11				(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	Corotuny	Cn	i Lonko	
	be in accordance with Whest approved	toratuwa	1, 51	Lanka.	
	drawings/shop drawings to the satisfaction of	es & Di	sser	The state of the s	
	the Consultant www.lib.mrt.ac	.1k		(Note)	
2.1.13	During erection the steel work should be				
	securely bolted or otherwise fastened, and		İ		
	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			OI · · ·	
2115	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)	
				(1.000)	
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	Description Description	Qty.	Unit	Rate	Amount
No.	•			LKR	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.			(NI 4 )	
	paining, 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	Standard Stool Work				
2.2	Structural Steel Work University of M	loratuwa <sup>*</sup>	l, Sr	i Lanka.	
	( Electronic Thes		0		
2.2.1	Steel Top Member C200mm x 75mm x 20mm	.lk	ober.	accord .	
	(6.18kg/m) fixed in position, as specified	9 300.00			
	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
		512.00	111	3,300.00	1,072,000.00
	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)

	TURAL STEEL WORK - 12m Span Roof S			·	
Item	Description	Qty.	Unit		Amount
No.				LKR	LKR
2.0	Structural Steel Work				
2.1	<u>Notes</u>				
2.1.1	The Bidder shall refer the following prior to				
	pricing the items in this trade			(Note)	
	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.				
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, clears, haunches, etc (please refer method of payment)  Lectronic Thes	es & Di	a, Sr sseri	i Lanka. tations <sub>e)</sub>	
2.1.5	Where members of steel structures are	.lk			
	fastened together by means of reverts, bolts or				
	by welding and all such connection shall be finished neatly.				
	·			(Note)	
	<u>Welding</u>				
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)	
210				(14016)	
2.1.8	Butt welds which will be visible in the completed works should be dressed off smooth and flushed with adjacent surface.			(Note)	
L			l		

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

STRUCTURAL STEEL WORK - 12m Span Roof Structure (HRS)						
Item	Description	Qty.	Unit		Amount	
No.				LKR	LKR	
	<u>Fabrication</u>					
210	A 11 4 1					
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.			(Nata)		
	Erection			(Note)		
2.1.10						
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.			(Nota)		
2.1.12				(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 1 vites 1 approved	loratuwa 1	a. Sr	i Lanka.		
	drawings/shop drawings to the satisfaction of					
	the Compultor	Participation of the Control of the	D2CI	(Note)		
2.1.13	www.nb.nnt.ad	.lk		(11010)		
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.			(NI 4 )		
2114				(Note)		
2.1.14	No reverting, permanent bolting or welding					
	should be done until proper alignment has			OI ()		
2 1 1 5	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)		
				(11010)		
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	CTURAL STEEL WORK - 12m Span Roof S  Description	Qty.	Unit	Rate	Amount
No.	<b>F</b>	<b>C</b> -3, ·	01110	LKR	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)	
	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	University of M	loratuwa	ı, Sr	i Lanka.	
	( Electronic Thes	es & Di	sseri	tations	
2.2.1	Steel Top Member,75mm x 75mm x 8mm	.lk			
	angles (9.03kg/m) fixed in position, as	. IIX			
	specified including all items as defined in the				
222	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		277 000 00	277 000 00
	notes to the trade on.	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				2 142 000 00
	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

