INSTITUTION OF ENGINEERS MALAYSIA

DESIGN REPORT

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DECLARATION

I declare that all informations in this Design Report are the result of my own writing based on the working experiences.

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1. EXECUTIVE SUMMARY

The design project submitted in this report was analysis and design checking and also a forensic investigation of Larkin Stadium, Johor Bahru. The works involved in this project were mainly focused on steel roof truss to sustain the designed load after experienced corrosion to some of the members and failure at some of the joints. The scope of this project was to investigate the truss through forensic point of view, reanalyse and redesign works and finally make a decision whether the roof truss should be repaired or rebuilt.

Larkin Stadium was built in 1964 to serve and cater mainly for sport activities in Johor. The stadium is managed by Stadium Board Authority and maintained by Public Work Department (JKR). In 1992 the stadium was upgraded where a roof was installed on top of the grandstand area. The roof is of space truss system. In July 2009, the stadium personnel noticed that two members from the top-chords do not attached to the joints. They then alert JKR for investigation.

JKR engaged consultant from Universiti Teknologi Malaysia to carry out structural engineering assessment on the roof. This final report presents the structural engineering assessment carried out by the team. The main objective of the structural assessment is to determine the structural integrity and stability of the roof. The methodology of work constitute of three categories namely visual inspection and condition survey, alignment survey, and structural analysis and design check. The assessment was to look at the condition of all of the members including joints and support plate. Any weakness such as defect, deterioration, and buckled members were recorded. The alignment survey can observe deflection of the roof truss and analysis and design check was to check the whole system capacity at present condition with two defect members and compare with complete system in responding to several loading combination.

Visual inspection found many roof structural members especially those located at the top were not in good condition. This also applies to joints (ball). About 59% were found to be corroded and the paint peeled off from the members. Purlins were the worst. There are even members that lost their section. The bottom part of the roof is much better with around 8% corroded. Alignment survey show that the left-hand side of the roof was deflected 30.31 mm relative to the right-hand side. This finding is close to what being found from analysis and design check. Interestingly, there are several members were found to be underdesigned and slender for ultimate load situation. As a conclusion, the roof needs to be repair immediately. The underdesigned members and the buckled members should be replaced with suitable size. The corrosion product need to be clean and new paint be applied.

The design calculations were carried out for top, bottom chord and diagonal members. The code and specifications used in this design were based on BS 5950: 1990 (Code of Practice for the Structural Use of Steel). The specifications include the construction materials, quality control and testing requirements.

The preparation of Bill of Quantity (BQ) covers the structural works of the truss. However, it can be used as a guideline to prepare for the document tender.

Typical calculation for quantity of material is shown in taking-off forms. This taking-off was carried out for the designed members only. The repetitive works were discarded from this calculation.

The whole design works were translated into three drawings and detailing as attached in Appendix.

Prepared by
Yusof Ahmad, November 2011
2. DESIGN EXPERIENCES

2.1 Introduction of Project

The whole roof truss structure was treated as a three dimensional roof truss system. The coordinates of each joint of the truss were obtained from survey using an equipment called Total Station. The survey was done on 17 Dec 2009. The roof sheeting was made by zincalum which supported by a series of purlins from lipped channel section. From the visual inspection, all purlins were badly attacked by corrosion and loose the material almost more than 60% of its cross section. The size of the roof truss was 94.14 m length, 21.26 m wide and 2.25 height as shown in Figure 1. The structure was symmetry at the mid length. The front portion of the roof truss was cantilever for 13 m whereas the remaining were supported by reinforced concrete columns where the spacing was between 3 – 10 meters. The roof truss was attached to the concrete columns by means of plates and bolts. The truss members were made by tubular steel section where the details mechanical properties were unknown. This structure was built in 1990. All joints use ball and socket system where each member free to rotate. An engineering software called STAADPro (Structural Analysis and Design for Professional) was used to analyze and design this structure. This software was licensed to Department of Structures and Materials, Faculty Civil Engineering, UTM, JB.

Figure 1  The layout of the roof truss
2.2 Loading

The loading used in this analysis was

a) Unfactored dead load (DL)
   i) Selfweight of roof truss : automatically included in STAADPro
   ii) Sheeting and insulation : 0.20 kN/m²
   iii) Purlins : 0.07 kN/m²
      \[ G_k = 0.27 \times 3.05 \times 3.05 = 2.5 \text{ kN} \]

b) Unfactored live load (LL)
   i) Imposed loads : 0.30 kN/m²
      \[ Q_k = 0.3 \times 3.05 \times 3.05 = 2.8 \text{ kN} \]

c) Unfactored wind load (WL)
   i) Wind load : \[ q = 0.613 K_1 K_2 K_3 K_4 V^2 \]
      \[ = 0.613 \times 1.1 \times 1.1 \times 1.1 \times 1.5 \times 22^2 \]
      \[ = 592 \text{ N/m}^2 \]
      \[ = 0.6 \text{ kN/m}^2 \]
      \[ W_k = 0.6 \times 3.05 \times 3.05 = 5.6 \text{ kN} \]

d) Load combinations
   i) Load combination 1 : 1.0LL
   ii) Load combination 2 : 1.4DL + 1.6LL
   iii) Load combination 3 : 1.2DL + 1.2LL + 1.2WL

2.3 Computer Analysis

Two models were considered in this analysis. The first model was the complete structure without any defects. The second model was the structure with the defect where two members were taken out from the model. The two members were identified at site during first investigation where both members were detached from its ball and socket. Both members were the main top chord of truss structure.

Since the structure is symmetry, half of the structure was modeled in the computer. The isometric view of the first model of roof structure is shown in Figure 2. The
pin supports shown in the figure represent the column points. The supports at the right part of the structure were roller support representing the restraint in X direction only where the rest of degrees of freedom were released to simulate the symmetrical behavior of the structure. Figure 3, 4 and 5 show the plan, front and side view, respectively. Total number of nodes and members used to model this structure was 240 and 867, respectively. The purlins were not modeled in this analysis as it was not a subject in this computer analysis.

The member identification numbering systems for all bottom chord, top chord and diagonal members were mapped in Figure 6, 7 and 8, respectively. When refer to the results given by STAADPro, these figures would guide the reader about the exact location of the truss members.

Figure 9, 10, 11, and 12 are the sections used for the truss structure (shown by the highlighted members) using steel Circular Hollow Section (CHS) of $D = 48 \text{ mm}$, $T = 3.0 \text{ mm}$; $D = 60 \text{ mm}$, $T = 3.0 \text{ mm}$; $D = 76 \text{ mm}$, $T = 3.0 \text{ mm}$; and $D = 114 \text{ mm}$, $T = 4.0 \text{ mm}$; respectively. Since the material was steel, an assumption was made that the Young’s modulus was 205 kN/mm$^2$, the density was 7850 kg/m$^3$ and the Poisson’s ratio was 0.3.

The second model was similar to first model except two members were removed from the structure. The layout in Figure 13 shows the model without the two members.

The loads were directly applied to the top layer of all truss joints. Figure 14, 15 and 16 show the unfactored dead load, live load and wind load, respectively. Linear static analysis was implemented using STAADPro to obtain the results.

### 2.4 Design Calculations

Transformer load

Transformer + oil = 152 ton

Factored dead load is

$$G_k = 152 \times 9.81 \times 1.4 = 2090 \text{ kN}$$

Converting to uniformly distributed load to be applied on two main beams ($L = 5 \text{ m}$)

$$W_k = \frac{2090}{2} = 209 \text{ kN/m}$$

Selfweight for all members will be calculated automatically by computer software
Since the transformer has four wheels, the load should be converted to point loads and these loads become live load. Thus the factored live load is
\[ Q_4 = \frac{152 \times 9.81 \times 1.6}{4} = 597 \text{ kN} \]

For maximum effect to the bending moment, the load is arranged as shown in the following diagram

Member properties for right columns – column 13 and 14
\[ A = 30.4 \times 2 = 60.8 \text{ cm}^2 \]
\[ I_y = [152 + 30.4(10.36^2)] \times 2 = 68.3 \times 10^6 \text{ mm}^4 \]
\[ I_z = 19.55 \times 10^6 \times 2 = 39.1 \times 10^6 \text{ mm}^4 \]

Member properties for horizontal bracings – bracing 15 and 33
\[ A = 2.28 \times 2 \times 10^3 = 4.56 \times 10^3 \text{ mm}^2 \]
\[ I_y = [852 + 22.8(9.6^2)] \times 2 = 59.6 \times 10^6 \text{ mm}^4 \]
\[ I_z = 1.14 \times 10^6 \times 2 = 2.28 \times 10^6 \text{ mm}^4 \]

Member properties for main beams – beam 1
\[ A = 2(300 \times 25) + (15 \times 650) = 24.75 \times 10^3 \text{ mm}^2 \]
\[ I_y = \left(\frac{300 \times 25^3}{12} + 300 \times 25 \times 337.5^2\right) \times 2 + \frac{15 \times 650^3}{12} = 2052 \times 10^6 \text{ mm}^4 \]
\[ I_z = \left(\frac{25 \times 300^3}{12}\right) \times 2 + \frac{650 \times 15^3}{12} = 112 \times 10^6 \text{ mm}^4 \]

Member properties for secondary beams – beam 5, 7 and 8
\[ A = 300 \times 20 + 400 \times 20 + 10 \times 500 = 19 \times 10^3 \text{ mm}^2 \]
\[ I_y = \left(\frac{300 \times 20^3}{12} + 300 \times 20 \times 260^2\right) + \left(\frac{400 \times 20^3}{12} + 400 \times 20 \times 260^2\right) + \frac{10 \times 500^3}{12} \]
\[ = 1051 \times 10^6 \text{ mm}^4 \]
\[ I_z = \frac{20 \times 300^3}{12} + \frac{20 \times 400^3}{12} + \frac{500 \times 10^3}{12} = 151 \times 10^6 \text{ mm}^4 \]

Design of left columns – column 11 and 12

Results from computer analysis (QSE)

Maximum axial force, \[ F = 548 \text{ kN} \]

Maximum shear force, \[ V = 7.5 \text{ kN} \]
Maximum bending moment about x-axis,  \( M_x = 4.3 \text{ kNm} \)

Maximum bending moment about y-axis,  \( M_y = 3.8 \text{ kNm} \)

Try UC section of 200 x 200 x 56.2 kg/m, grade 43. The section properties are

Length of column,  \( L_{\text{max}} = 3.964 \text{ m} \)

Cross sectional area,  \( A = 7.15 \times 10^3 \text{ mm}^2 \)

Moment of inertia about x-axis,  \( I_x = 49.8 \times 10^6 \text{ mm}^4 \)

Moment of inertia about y-axis,  \( I_y = 17.0 \times 10^6 \text{ mm}^4 \)

Plastic moment of inertia about x-axis,  \( S_x = (200 \times 12 \times 94) + (12 \times 88 \times 44) = 544 \text{ cm}^3 \)

Plastic moment of inertia about y-axis,  \( S_y = (12 \times 94 \times 47) = 212 \text{ cm}^3 \)

Gyration radius about x-axis,  \( r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{49.8 \times 10^6}{7.15 \times 10^3}} = 83.4 \text{ mm} \)

Gyration radius about y-axis,  \( r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{17 \times 10^6}{7.15 \times 10^3}} = 48.7 \text{ mm} \)

Elastic moment of inertia about x-axis,  \( Z_x = \frac{I_x}{y} = \frac{49.8 \times 10^6}{100} = 498 \text{ cm}^3 \)

Elastic moment of inertia about y-axis,  \( Z_y = \frac{I_y}{y} = \frac{17 \times 10^6}{100} = 170 \text{ cm}^3 \)

For  \( T = 12 \text{ mm} < 16 \text{ mm} \); from Table 6;  \( P_y = 275 \text{ N/mm}^2 \)

\[ \frac{b}{T} = 100 / 12 = 8.3 < 8.5 \varepsilon \]

\[ \frac{d}{T} = 176 / 12 = 14.7 < 39 \varepsilon \]

From Table 7, section is classified as plastic

Check for axially loaded members with moments

Compression members should be checked for

a) Local capacity at point of maximum bending moment

b) Overall buckling

a) Local capacity check

The following equation should be satisfied (from 4.8.3.2b)

\[
\left( \frac{M_x}{M_{rx}} \right)^{z_1} + \left( \frac{M_y}{M_{ry}} \right)^{z_2} \leq 1.0
\]

Where  \( M_x = 3.8 \text{ kNm} \)

\( M_y = 4.3 \text{ kNm} \)
\[ z_1 = 2.0 \]
\[ z_2 = 1.0 \]

\( M_{r_x} \) and \( M_{r_y} \) are reduced plastic moment capacities due to axial load.

\[
n = \frac{F}{A_{p}p_y} = \frac{548 \times 10^3}{7.15 \times 10^3 x 275} = 0.278
\]

To find \( S_{r_x} \)
\[ K_1 = S_x = 544 \text{ cm}^3 \]
\[ K_2 = \frac{A^2}{4t} = \frac{(7.15 \times 10^3)^2}{4 \times 12} = 1065 \]
\[ S_{r_x} = K_1 - K_2n^2 = 544 - 1065(0.278)^2 = 461.7 \text{ cm}^3 \]

To find \( S_{r_y} \)
\[ K_1 = S_y = 212 \text{ cm}^3 \]
\[ K_2 = \frac{A^2}{4D} = \frac{(7.15 \times 10^3)^2}{4 \times 200} = 64 \]
\[ S_{r_y} = K_1 - K_2n^2 = 212 - 64(0.278)^2 = 207 \text{ cm}^3 \]

The reduced plastic moment capacities about both axes are

\[ M_{r_x} = S_{r_x}p_y = (461.7 \times 10^3)(275 \times 10^{-6}) = 127 \text{ kNm} \]
\[ 1.2Z_xp_y = 1.2(498)(265 \times 10^{-3}) = 158 \text{ kNm} \]
\[ \Rightarrow S_{r_x}p_y < 1.2Z_xp_y \]

Thus, \( M_{r_x} = 127 \text{ kNm} \)
\[ M_{r_y} = S_{r_y}p_y = (207 \times 10^3)(275 \times 10^{-6}) = 57 \text{ kNm} \]
\[ 1.2Z_yp_y = 1.2(170)(275 \times 10^{-3}) = 56 \text{ kNm} \]
\[ \Rightarrow 1.2Z_yp_y < S_{r_y}p_y \]

Thus, \( M_{r_y} = 56 \text{ kNm} \)

From 4.8.3.2b
\[
\left( \frac{M_x}{M_{r_x}} \right)^2 + \left( \frac{M_y}{M_{r_y}} \right)^2 \leq 1.0
\]
\[
\left( \frac{3.8}{127} \right)^2 + \left( \frac{4.3}{56} \right)^2 = 0.077 \leq 1.0
\]

Therefore the local capacity check is satisfied

b) Overall buckling check
The following condition should be satisfied

\[
\frac{F}{A_y p_c} + \frac{m_x M_x}{M_h} + \frac{m_y M_y}{p_y Z_y} \leq 1.0
\]

For both axes, \( \beta = 0 \), thus \( m_x = m_y = 0.57 \)

Determining \( M_h \) from 4.3.7.3:

\[
M_h = S_x p_h
\]

\[
\lambda_{LT} = n u v \lambda
\]

where \( n = 1.0, u = 0.848 \) and \( x = 15.8 \) (from section table)

Effectiv elength, \( L_{ex} = L_{ev} = 1.0L \). Thus \( L_{ev} = 1.0 \times 3.964 = 3.964 \) m

\[
\lambda = L_{ev} / r_y = 3964 / 48.7 = 81.4
\]

\[
\lambda / x = 81.4 / 15.8 = 5.15
\]

\( N = 0.5 \), thus \( v = 0.81 \)

\[
\lambda_{LT} = n u v \lambda = 1.0 \times 0.848 \times 0.81 \times 81.4 = 56
\]

For \( \lambda_{LT} = 56 \) and \( p_y = 275 \) N/mm\(^2\); \( p_h = 225 \) N/mm\(^2\)

\[
M_h = S_x p_h = (544 \times 10^3)(225 \times 10^{-6}) = 122 \text{ kNm}
\]

Determining \( p_c \):

For \( \lambda = 81.4 \) and \( p_y = 275 \) N/mm\(^2\), \( p_{cy} = 160 \) N/mm\(^2\)

Thus

\[
\frac{548 \times 10^3}{71.5 \times 10^2}(160) + \frac{(0.57)(3.8 \times 10^6)}{122 \times 10^6} + \frac{(0.57)(4.3 \times 10^6)}{275(170 \times 10^3)} = 0.55 \leq 1.0
\]

Section has sufficient resistance against overall buckling

Adopt UC 200 \times 200 \times 56.2 \text{ kg/m}.

Design of right columns – column 13 and 14

Results from computer analysis (QSE)

Maximum axial force, \( F = 547 \) kN

Maximum shear force, \( V = 7.5 \) kN

Maximum bending moment about x-axis, \( M_x = 3.8 \) kNm

Maximum bending moment about y-axis, \( M_y = 9.4 \) kNm

Try double structural C-section of 203 \times 76 \times 23.82 \text{ kg/m}, grade 43. The section properties are

Length of column, \( L_{max} = 3.964 \) m
Cross sectional area, $A = 60.8 \, \text{cm}^2$

Moment of inertia about $x$-axis, $I_x = 68.3 \times 10^6 \, \text{mm}^4$

Moment of inertia about $y$-axis, $I_y = 39.1 \times 10^6 \, \text{mm}^4$

Plastic moment of inertia about $x$-axis, 
$S_x = [203 \times 7.1 \times 121.1 + (69 \times 11.2 \times 80.5)2]2 = 599 \, \text{cm}^3$

Plastic moment of inertia about $y$-axis, $S_y = 226 \times 2 = 452 \, \text{cm}^3$

Gyration radius about $x$-axis, $r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{68.3 \times 10^6}{60.8 \times 10^2}} = 106 \, \text{mm}$

Gyration radius about $y$-axis, $r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{39.1 \times 10^6}{60.8 \times 10^2}} = 80 \, \text{mm}$

Elastic moment of inertia about $x$-axis, $Z_x = \frac{I_x}{y} = \frac{68.3 \times 10^6}{122} = 560 \, \text{cm}^3$

Elastic moment of inertia about $y$-axis, $Z_y = \frac{I_y}{y} = \frac{39.1 \times 10^6}{101.5} = 385 \, \text{cm}^3$

For $T = 12 \, \text{mm} < 16 \, \text{mm}$; from Table 6; $P_y = 275 \, \text{N/mm}^2$

$b / T = 76 / 11.2 = 6.8 < 8.5\ell$

$d / T = 203 / 7.1 = 28.6 < 39\ell$

From Table 7, section is classified as plastic

Check for axially loaded members with moments

Compression members should be checked for

a) Local capacity at point of maximum bending moment

b) Overall buckling

a) Local capacity check

The following equation should be satisfied (from 4.8.3.2b)

$$\left( \frac{M_x}{M_{rx}} \right)^{z_1} + \left( \frac{M_y}{M_{ry}} \right)^{z_2} \leq 1.0$$

Where $M_x = 3.8 \, \text{kNm}$

$M_y = 9.4 \, \text{kNm}$

$z_1 = z_2 = 1.0$

$M_{rx}$ and $M_{ry}$ are reduced plastic moment capacities due to axial load.
\[ n = \frac{F}{A_p P_y} = \frac{547 \times 10^3}{60.8 \times 10^2 \times 275} = 0.327 \]

To find \( S_{rx} \)
\[ S_{rx} = S_x - \frac{A^2n^2}{8t} \text{ (assume hollow section)} \]
\[ = 599 - \frac{60.8^2 \times 0.327^2}{8 \times 0.71} \]
\[ = 529 \text{ cm}^3 \]

To find \( S_{ry} \)
\[ S_{ry} = S_y - \frac{A^2n^2}{8t} \text{ (assume hollow section)} \]
\[ = 452 - \frac{60.8^2 \times 0.327^2}{8 \times 0.71} \]
\[ = 382 \text{ cm}^3 \]

The reduced plastic moment capacities about both axes are
\[ M_{rx} = S_{rx}P_y = (529 \times 10^3)(275 \times 10^{-6}) = 145 \text{ kNm} \]
\[ 1.2Z_{x}P_y = 1.2(560)(275 \times 10^{-3}) = 185 \text{ kNm} \]
\[ \Rightarrow S_{rx}P_y < 1.2Z_{x}P_y \]
Thus, \( M_{rx} = 145 \text{ kNm} \)
\[ M_{ry} = S_{ry}P_y = (382 \times 10^3)(275 \times 10^{-6}) = 105 \text{ kNm} \]
\[ 1.2Z_{y}P_y = 1.2(385)(275 \times 10^{-3}) = 127 \text{ kNm} \]
\[ \Rightarrow S_{ry}P_y < 1.2Z_{y}P_y \]
Thus, \( M_{ry} = 105 \text{ kNm} \)

From 4.8.3.2b
\[ \left(\frac{M_s}{M_{rx}}\right)^2 + \left(\frac{M_s}{M_{ry}}\right)^2 \leq 1.0 \]
\[ \left(\frac{3.8}{145}\right)^2 + \left(\frac{9.4}{105}\right)^2 = 0.116 \leq 1.0 \]

Therefore the local capacity check is satisfied

b) Overall buckling check

The following condition should be satisfied
\[
\frac{F}{A_p p_t} + \frac{m_x M_x}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1.0
\]

For critical case, thus \( m_x = m_y = 1.0 \)

Determining \( M_b \) from 4.3.7.3:

\[
M_b = S_v p_h
\]

\[
\lambda_{LT} = n u v \lambda
\]

where \( n = 1.0, u = 0.911 \) and \( x = 16.5 \) (from section table for single C)

Effective length, \( L_{ex} = L_{cy} = 1.0L \). Thus \( L_e = 1.0 \times 3.964 = 3.964 \) m  

\[
\lambda = \frac{L_e}{r_y} = 3964 / 80 = 49
\]

\[
\frac{\lambda}{x} = 49 / 16.5 = 3.0
\]

\( N = 0.5 \), thus \( v = 0.91 \)

\[
\lambda_{LT} = n u v \lambda = 1.0 \times 0.911 \times 0.91 \times 49 = 41
\]

For \( \lambda_{LT} = 41 \) and \( p_y = 275 \) N/mm\(^2\); \( p_b = 238 \) N/mm\(^2\)

\[
M_b = S_v p_h = (599 \times 10^3)(238 \times 10^{-6}) = 142 \text{ kNm}
\]

Determining \( p_c \):

For \( \lambda = 49 \) and \( p_y = 275 \) N/mm\(^2\), \( p_{cy} = 220 \) N/mm\(^2\)

Thus

\[
\frac{547 \times 10^3}{(60.8 \times 10^3)(220)} + \frac{(1.0)(3.8 \times 10^6)}{142 \times 10^6} + \frac{(1.0)(9.4 \times 10^6)}{(275)(385 \times 10^3)} = 0.53 \leq 1.0
\]

Section has sufficient resistance against overall buckling

Adopt UC 203\( \times \) 76\( \times \) 23.82 kg/m double channel.

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**Design of beams – member 15, 22 and 28**

Results from computer analysis (QSE)

**Maximum axial force**, \( F = 109 \) kN (tension)

Since both ends are pinned, there ini no moment induced. Therefore the beams will be designed as axially loaded tension members.

Try UC section of 152\( \times \) 152\( \times \) 23 kg/m, grade 43.

Tension capacity, \( P_t \) should be taken from \( P_t = A_t p_y \)

From section properties, \( A_g = 2.92 \times 10^3 \) mm\(^2\)

The effective area, \( A_e = 2.92 \times 10^3 - 20 \times 5.8 = 2804 \) mm\(^2\)

Thus, the tension capacity is
\( P_t = A \cdot p_y = 2804 \times 275 \times 10^{-3} = 771 \text{ kN} > 109 \text{ kN} \)

Tension capacity is satisfied.

Design of bracing – bracing member 16 – 21, 23 – 27, and 29 – 32

All these members will be designed as axially loaded column with pinned ends.

Results from computer analysis (QSE)

- **Maximum axial force**, \( F = 94.8 \text{ kN} \) (compression)
- Try C-section of \( 152 \times 76 \times 17.88 \text{ kg/m} \), grade 43.
  - \( b / T = 8.47 < 8.5 \epsilon \)
  - \( d / T = 16.5 < 39 \epsilon \)
- Section is plastic
- From section properties, \( A = 22.8 \text{ cm}^2; \quad r_y = 6.11 \text{ cm}; \quad r_x = 2.23 \text{ cm} \)
- Member length, \( L = 6.4 \text{ m} \)
- **Compressive resistance of member**:
  - Effective length, \( L_{ex} = L_{ey} = 1.0L \). Thus \( L_e = 1.0 \times 6.4 = 6.4 \text{ m} \)
  - \( \lambda_y = L_e / r_y = 6400 / 22.3 = 287 > 180 \) \( \text{Cl. 3.6} \)
- Section is slender
- Thus the design strength should be reduced
- Stress reduction factor is given by Table 8, i.e.
  \[
  \frac{31}{(b / T \epsilon)} - 8 = \frac{31}{[76 / (9 \times 1)]} - 8 = 70
  \]
  \[
  \therefore \quad p_y = 275 - 70 = 205 \text{ N/mm}^2
  \]
- From Table 27(c), \( \lambda_y = 287 \) and \( p_y = 205 \text{ N/mm}^2 \)
  \[
  \therefore \quad p_{cy} = 21 \text{ N/mm}^2
  \]
- Compression capacity is
  \[
  P = A_p p_{cy} = 2280 \times 21 \times 10^{-3} = 48 \text{ kN} < F = 94.8 \text{ kN}
  \]
- Section fail against compression. Extra bracings are required or increase the member size.
- If the former is adopted,
  \[
  L_e = 3200 \text{ mm}; \quad \lambda_y = 3200 / 22.3 = 144 < 180
  \]
  \[
  \therefore \quad p_{cy} = 72 \text{ N/mm}^2
  \]
Compression capacity is

\[ P = A_p P_{cy} = 2280 \times 72 \times 10^{-3} = 164 \text{ kN} > F = 94.8 \text{ kN} \]

Compression capacity is sufficient.

Design of horizontal bracings – member 15 and 33

Since these members act as strut, its design is similar to axially loaded column.

Results from computer analysis (QSE)

Maximum axial force, \( F = 6.4 \text{ kN} \) (tension)

These members use double channels 152 x 76 x 17.88 kg/m. No design checking is required because the axial force is small.

Design of main beams – member 1

Results from computer analysis (QSE)

Maximum axial force, \( F = 26.3 \text{ kN} \) (tension)

Maximum shear force, \( V = 530 \text{ kN} \)

Maximum bending moment, \( M = 662 \text{ kNm} \)

Try UB section of 700 x 300, grade 43. The section properties are

Cross sectional area, \( A = 24.75 \times 10^3 \text{ mm}^2 \)

Moment of inertia about x-axis, \( I_x = 2052 \times 10^6 \text{ mm}^4 \)

Moment of inertia about y-axis, \( I_y = 112 \times 10^6 \text{ mm}^4 \)

\( u = 0.864; \quad x = 38.1 \)

Plastic moment of inertia about x-axis, \( S_x = (300 \times 25 \times 337.5 + 15 \times 325 \times 162.5)2 = 6647 \text{ cm}^3 \)

Elastic moment of inertia about x-axis, \( Z_x = 2052 \times 10^6 / 350 = 5863 \text{ cm}^3 \)

Gyration radius about y-axis, \( r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{112 \times 10^6}{24.75 \times 10^3}} = 67.3 \text{ mm} \)

For \( T < 16 \text{ mm} \); from Table 6; \( P_y = 275 \text{ N/mm}^2 \)

\( b / T = 150 / 25 = 6 < 8.5\varepsilon \)
\(d / T = 650 / 15 = 43.3 < 79\varepsilon\)

From Table 7, section is classified as plastic

Shear capacity check

Shear capacity of section is given by

\[ P_c = 0.6P_yA_t \quad \text{where} \quad A_t = tD \]

\[ = 0.6 \times 275 \times (15 \times 700 \times 10^{-3}) \]

\[ = 1732 \text{kN} \]

\(0.6P_c = 1039 \text{kN}\)

\(V = 530 \text{kN} < P_c = 1732 \text{kN}\)

Shear capacity is ok

Moment capacity check

Moment capacity is given by

\[ M_e = P_yS_x \leq 1.2P_yZ \]

\[ M_e = 265 \times 6647 \times 10^{-3} = 1761 \text{kNm} \]

\[ 1.2P_yZ = 1.2 \times 265 \times 5863 \times 10^{-3} = 1864 \text{kNm} \]

\[ \therefore \quad M_e = 1761 \text{kNm} \]

Since \(M = 662 \text{kNm} < M_e = 1761 \text{kNm}\); section is ok

Lateral torsional buckling check

The condition that must be satisfied is

\[ \vec{M} \leq M_b \quad \text{where} \quad \vec{M} = mM_A \]

For \(\beta = 0\); \(\therefore \quad m = 0.57\) (from Table 18)

\[ \vec{M} = 0.57 \times 662 = 377 \text{kNm} \]

Buckling resistance is given by \(M_b = p_bS_x\)

The effective length is \(L_e = 1.0L = 1.0 \times 5000 = 5000 \text{mm}\)

The slenderness ratio is \(\lambda_x = L_e / r_y = 5000 / 67.3 = 74.3\)

\(\lambda / x = 74.3 / 38.1 = 1.95\) and \(N = 0.5\)

From Table 14, \(v = 0.96\)

From Table 13, \(n = 1.0\)

\(\lambda_{LT} = nu\lambda = 1.0 \times 0.864 \times 0.96 \times 74.3 = 61.6\)

From Table 11, \(p_b = 200 \text{N/mm}^2\)

\[M_b = p_bS_x = 200 \times 6647 \times 10^{-3} = 1329 \text{kNm}\]
Since $\bar{M} = 377 < M_b = 1329$, section is sufficient against lateral torsional buckling

**Design of secondary beams – member 5, 7, 8**

**Results from computer analysis (QSE)**

- Maximum axial force, $F = 51.2 \text{ kN}$ (tension)
- Maximum shear force, $V = 532 \text{ kN}$
- Maximum bending moment, $M = 280 \text{ kNm}$

Try built-up section as shown from steel grade 43. The section properties are

- **Cross sectional area**, $A = 19 \times 10^3 \text{ mm}^2$
- **Moment of inertia about } x\text{-axis**, $I_x = 1051 \times 10^6 \text{ mm}^4$
- **Moment of inertia about } y\text{-axis**, $I_y = 151 \times 10^6 \text{ mm}^4$
- $u = 0.87; \ x = 34.1$

**Plastic moment of inertia about } x\text{-axis**, $S_x = (300 \times 20 \times 260 + 10 \times 250 \times 125) \times 2 = 3745 \text{ cm}^3$

- **Elastic moment of inertia about } x\text{-axis**, $Z_x = 1051 \times 10^6 / 270 = 3892 \text{ cm}^3$

**Gyration radius about } y\text{-axis**, $r_y = \sqrt{I_y / A} = \sqrt{151 \times 10^6 / 19 \times 10^3} = 89 \text{ mm}$

For $T < 16 \text{ mm}$; from Table 6; $P_y = 275 \text{ N/mm}^2$

- $b / T = 150 / 20 = 7.5 < 8.5t$

- $d / T = 500 / 10 = 50 < 79t$

From Table 7, section is classified as plastic

**Shear capacity check**

Shear capacity of section is given by

$$P_v = 0.6 P_y A_y \quad \text{where} \quad A_y = td$$

$$= 0.6 \times 265 \times (10 \times 500 \times 10^{-3})$$

$$= 859 \text{ kN}$$

$0.6P_v = 0.6 \times 859 = 515 \text{ kN}$

$V = 532 \text{ kN} < P_v = 859 \text{ kN}$, shear capacity is ok

Since $V = 532 \text{ kN} > 0.6P_v = 515 \text{ kN}$, thus the section is subjected to high shear load

**Moment capacity check**
Moment capacity should be calculated from
\[ M_c = P_y (S - S_v \rho_1) ≤ 1.2P_y Z \]
\[ \rho_1 = \frac{2.5F_v}{P_v} - 1.5 = \frac{2.5 \times 532}{859} - 1.5 = 0.048 \]
\[ S_v = 10 \times 250 \times 125 \times 2 = 625 \]
\[ M_c = 265(3745 - 625 \times 0.048) \times 10^{-3} = 984 \text{ kNm} \]
\[ 1.2P_y Z = 1.2 \times 265 \times 3892 \times 10^{-3} = 1237 \text{ kNm} \]
\[ \therefore M_c = 984 \text{ kNm} \]
Since \( M = 280 \text{ kNm} < M_c = 984 \text{ kNm} \), section is ok

Lateral torsional buckling check

The condition that must be satisfied is
\[ \bar{M} ≤ M_b \quad \text{where} \quad \bar{M} = mM_A \]
For \( \beta = 0 \); \( \therefore m = 0.57 \) (from Table 18)
\[ \bar{M} = 0.57 \times 280 = 160 \text{ kNm} \]
Buckling resistance is given by \( M_b = p_b S_x \)
The effective length is \( L_e = 1.0L = 1.0 \times 2800 = 2800 \text{ mm} \)
The slenderness ratio is \( \lambda_y = L_e / r_y = 2800 / 89 = 31.5 \)
\[ \lambda / x = 31.5 / 34.1 = 0.92 \quad \text{and} \quad N = 0.5 \]
From Table 14, \( v = 0.99 \)
From Table 13, \( n = 1.0 \)
\[ \hat{\lambda}_{LT} = mnu\lambda = 1.0 \times 0.87 \times 0.99 \times 31.5 = 27.1 \]
From Table 12, \( p_b = 265 \text{ N/mm}^2 \)
\[ M_b = p_b S_x = 265 \times 3745 \times 10^{-3} = 992 \text{ kNm} \]
Since \( \bar{M} = 160 < M_b = 992 \), section is sufficient against lateral torsional buckling
2.5 Section Design

The results from computer analysis software were divided into two parts. Part 1 was the results for the complete roof truss structure without defects whereas Part 2 was the results for the roof truss with defects.

**Part 1: Roof truss without defects (model 1)**

1) **Results of Load Combination 1**

Load Combination 1 was the case for unfactored live load (1.0LL) aiming for the determination of deflection under service load. The deformed shape of the truss from isometric view and side view is shown in Figure 17 and 18 respectively. Figure 19 shows the summary of the deflection and the maximum deflection in vertical direction was 17.5 mm which occurred at corner of the overhanging part. This value is not critical to the structure for serviceability limit state even though a value of L/300 is taken for the limiting value. Thus, this structure was satisfactory for deflection due to this load.

2) **Results of Load Combination 2 and 3**

Load combination 2 and 3 was the case of 1.4DL + 1.6LL and 1.2DL + 1.2LL + 1.2WL, respectively, for the determination of member capacity under ultimate limit state. Figure 20 shows the typical axial force diagram for load combination where the blue and red color indicate the tension and compression members, respectively. The capacity of each member were checked against axial force either tension or compression. The new member sizes were proposed during design stage where the STAADPro would search the most optimum section of British Section from the library. BS 5950: 1990 was used for design works. All truss members were treated as having
unrestrained conditions. Appendix A reported the details of the results for all truss members. From the results, it was found that 7 members were failed in the ultimate limit state point of view. All the members that failed were diagonal members which attached to the column. These members are shown in Figure 21. In general, the size of the members were small i.e CHS: D = 60 mm and T = 3.0 mm. This section was insufficient to take the design loads as the axial forces were considerably high for these diagonal members.

**Part 2: Roof truss with defects (model 2)**

1) **Results of Load Combination 1**

The deformed shape of the truss from isometric view and the summary of the deflection under unfactored live load and the maximum deflection in vertical direction was 20.9 mm which occurred at a point of the overhanging part close to the defect area. This is shown in Figure 22. This value is not critical to the structure for serviceability limit state even though a value of L/300 is taken for the limiting value. Thus, this structure was satisfactory for deflection due to this load.

2) **Results of Load Combination 2 and 3**

Load combination 2 and 3 was the case of 1.4DL + 1.6LL and 1.2DL + 1.2LL + 1.2WL, respectively, for the determination of member capacity under ultimate limit state. Appendix A reported the summary of the results for member sizes of all truss members. From the analysis, 9 diagonal members failed and the details locations of these members are shown in Figure 23. The size for the members was CHS: D = 60 mm and T = 3.0 mm. From this figure it was found that almost the same members failed for truss without
defects (model 1) and truss with defect (model 2) except two members. Other members were considered critical in which the slenderness criterion was not satisfied. Figure 24, 25 and 26 show the slender members at bottom chord, top chord and diagonal.

*Design checking due to unfactored dead load*

The maximum deflection due to unfactored dead load for model 1 and 2 was 21.3 mm and 26.4 mm, respectively as shown in Figure 27 and 28. No member failed for both models due to this load. The details are shown in Appendix A. However, some members failed due to slenderness problem.

### 2.6 Conclusions of Computer Analysis

The conclusions that can be drawn from the computer analysis are:-

i) Generally, some of the members were underdesigned especially the diagonal members and as a result the impending failure tends to occur in future if further action is not taken.

ii) The two damage members need to be repaired as soon as possible to prevent over stress to other members as the axial forces would be transferred when the members were removed until the force equilibrium is achieved.

iii) The members that are marked failed either due to insufficient capacity or slenderness problem should be stiffened or replaced by bigger section proposed in Appendix A.

iv) The statements given in i), ii) and iii) above are based on ultimate limit state design. With the current situation, the structure can still withstand because the
actual load was unfactored dead load (1.0DL) which is much lesser compared to design load.

v) The deflection was not critical and the value was small which has been proved by survey works at site.
Figure 2  Isometric view of model 1 (without defects)
**Figure 3**  Plan view of model 1 (without defects)
Figure 4  Front view of model 1 (without defects)

Figure 5  Side view of model 1 (without defects)
Figure 6  Member numbering system for bottom chord of model 1 (without defects)
Figure 7  Member numbering system for top chord of model 1 (without defects)
Figure 8 Member numbering system for diagonal members of model 1 (without defects)
Figure 9  Members having CHS: $D = 48$ mm; $T = 3.0$ mm
Figure 10  Members having CHS: $D = 60$ mm; $T = 3.0$ mm
Figure 11  Members having CHS: $D = 76$ mm; $T = 3.0$ mm
Figure 12  Members having CHS: $D = 114$ mm; $T = 4.0$ mm
Figure 13  Layout of model 2: the two members were removed
Figure 14  Unfactored dead load (DL)
Figure 15  Unfactored live load (LL)
Figure 16  Unfactored wind load (WL)
Figure 17  Deformed shape of the truss due to load combination 1 for model 1 (isometric view)
Figure 18  Deformed shape of the truss due to load combination 1 for model 1 (side view)
Figure 19  Maximum deflection of the truss due to load combination 1 for model 1

Figure 20  Axial force diagram of part of the roof truss due to load combination 2
Figure 21  The expected diagonal members to be failed due to insufficient design capacity for model 1
Figure 22  Maximum deflection of the truss due to load combination 1 for model 2
Figure 23  The expected diagonal members to be failed due to insufficient design capacity when the two top chord members were taken out from the structure (model 2)
Figure 24  The expected bottom chord members to be failed due to slenderness when the two top chord members were taken out from the structure (model 2)
Figure 25  The expected top chord members to be failed due to slenderness when the two top chord members were taken out from the structure (model 2)
Figure 26  The expected diagonal members to be failed due to slenderness when the two top chord members were taken out from the structure (model 2)
Figure 27  Maximum deflection of the truss due to unfactored dead load for model 1
Figure 28  Maximum deflection of the truss due to unfactored dead load for model 2
Terima kasih.

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