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ANALYSIS OF TRUSS SYSTEM USING COLD-FORMED STEEL SECTIONS

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Abstract

The use of cold-formed steel sections for roof truss construction is quite new in Indonesia. This kind of construction is growing quite fast and significant due to the substitution of using woods or timber. The advantages of using cold-formed for roof truss system are that this is speedy, light, flexible, and easy to erect. These advantages have made cold-formed more popular than traditional wood truss systems. Therefore, a new cold-formed steel roof truss is introduced in the market. This paper is aimed to provide the information on the analysis and design of cold-formed section used to substitute a conventional roof truss system. The sections were designed and tested for the material capacities estimation and the tests were conducted in accordance to BS 5950 Part 5: 1998. The test results have shown good agreement with the theoretical estimation, therefore the designed section is accepted used for the roof truss system.

The effort to explore the usage of cold-formed steel structure is very great because it is a quite new technology. The test carried out the full scale test actually not only for trust but also for beams and column used as light construction. Some experiments have done for experiments of Truss System to conform the design and the performance of the system. The test has been carried out conforming to the following:

1. **Project** : Typical Long Span Truss for Roof of Structures

2. **Material Specification** :

   Truss member/rafter

   - Section : 102mm × 51mm × 1.6mm thick

     : C – Lipped Channel (MC 100.16)
   - Material : High Tensile Galvanized Steel

     : Yield Stress $450 N/mm^2$

     : Zinc Coating of Z 275

3. **Connection** : Class 2 – Self Tapping Screws

4. **Truss Spacing** : 1.5m

5. **Roof Pitch** : 25 °

6. **Design Code** :

- BS 5950 Part 5 - Code of Practice for Design of Cold Formed Section
- BS 6399 - Load Design
- BS 8110 - Structural use of concrete
- CP3 Chapter V (1972) - Wind loading (Adopt basic wind speed of V=30 m/s)

7. **Loadings** :

- Concrete Roof Tiles = 0.55 kN/m^2
- Batten = 0.05 kN/m^2
- Ceiling & Frame = 0.10 kN/m^2
- M & E Services = 0.15 kN/m^2
- Live Load = 0.25 kN/m^2
- Basic Wind Speed = 30 m/s

The aims of the test is to prove the design value compared with the testing result, to show that the Truss System has performed far higher than the design load without visible failure in accordance to the design specification.

1.0 Introduction

A series of theoretical and experimental investigations have been done to obtain the following objectives:

1. To conform the adequacy of intended use of the proposed system.
2. To define the actual behavior of the truss system in comparison of theoretical estimation.
3. To fit the requirements of Truss System Design.

To achieve the best reliable output, the methodology adopted should satisfy the requirements described as in the followings:

1. Determine the theoretical required strength of the truss system, which regarding to the axial force capacities, the deflection limitation, and the moment capacity of members.
2. Full scale test on the proposed system – in order to determine the ultimate strength in actual, therefore the results are compared with the theoretical estimations.
3. Modeling of the tested specimen – to conform the validity of theoretical estimation to represent the actual condition.

The Full-Scale Test was carried out based on join - research with Steel Technology Centre (STC) in the Laboratory of Structural and Material, Universiti Teknologi Malaysia.

2.0 Theoretical Design Capacity of Sections

2.1 MC100 - 16

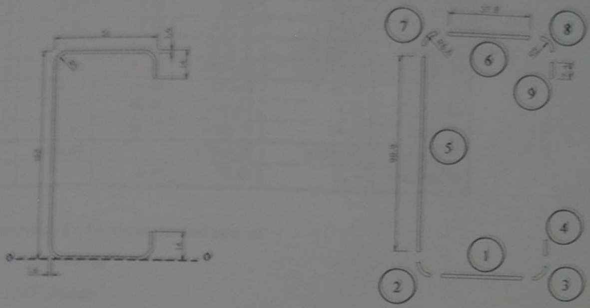

Figure 2-1 Detail and element labeling of MC10016

2.1.1 Bending Capacity

$h \quad = B_2/B_1 = 102/51$

$\quad = 2$

$K_s \quad = 4.163 > 4$ for stiffened element **Ok**

$p_{cr} \quad = 1379.86 N/mm^2$

$f_c/p_{cr} \quad = 450/1379.86$

$\quad = 0.3261 > 0.123$

Therefore,

$\frac{b_{eff}}{b} \quad = 0.99$

$b_{eff} \quad = 0.99b = 0.98 \times 37.8$

$\quad = 37.4mm$

Table 2-1 Second moment of area calculation based on effective width on compression flange

Element	A (mm^2)	y (mm)	Ay (mm^3)	$I_{element}$ (mm^4)	Ay^2 (mm^4)
1	60.48	0.80	48.38	12.90	38.71
2	14.57	2.11	30.76	46.45	64.94
3	14.57	2.11	30.76	46.45	64.94
4	11.84	10.30	121.95	54.03	1256.11
5	142.21	51.00	7252.61	93363.61	369883.01
6	59.84	101.20	6055.81	12.77	612847.77
7	14.57	99.09	1443.68	46.45	143052.93
8	14.57	99.09	1443.68	46.45	143052.93
9	11.84	91.70	1085.73	54.03	99561.26
Total	344.49		17513.37	93683.12	1369822.59
		$\bar{y}$ =	50.84	I_{00} =	1463505.71

Second moment of area about neutral axis is:

$I_{NA} \quad = 573142.47\text{mm}^4$

$Z_T \quad = 11273.66\text{mm}^3$

$Z_C \quad = 11202.73\text{mm}^3$

$Z_C < Z_T$ the compressive stress will reach p_o before the tensile stress reaches yield.

Therefore,

$p_o \quad = 439.40\text{N/mm}^2$

The moment capacity is:

$M_c \quad = p_o Z_C = 439.40 \times 11202.73 \times 10^{-6}$

$\quad = 4.92\text{kNm} > M_x = 2.329\text{kN}$ Ok

2.1.2 Axial Compressive Capacity

$h \quad = B_2 / B_1 = 51 / 102$

$\quad = 0.5$

$$K_1 = 7 - \frac{1.8 \times 0.5}{0.15 + 0.5} - 1.43 \times 0.5^3$$

$\quad = 5.457 > 4$ for stiffened element Ok

$p_{cr} \quad = 327.75\text{N/mm}^2$

$f_y/p_{cr} \quad = 450 / 327.75$

$\quad = 1.373 > 0.123$

Therefore,

$$\frac{b_{eff}}{b} = 0.6704$$

$$b_{eff} = 0.6704b = 0.6704 \times 88.8$$
$$= 59.53\text{mm}$$

$$K_2 = 1.364 < 4$$
$$K_2 = 4$$

$$p_{cr} = 185000\,K\left(\frac{t}{b}\right)^2 = 185000 \times 4 \times \left(\frac{1.6}{37.8}\right)^2$$
$$= 1325.83\text{N/mm}^2$$

$$f_c/p_{cr} = 450 / 1325.83$$
$$= 0.3394 > 0.123$$

Therefore,

$$\frac{b_{eff}}{b} = \left[1 + 14\left\{(f_{cr} / p_{cr})^{1/2} - 0.35\right\}^4\right]^{-0.2} = \left[1 + 14\left\{(0.3394)^{1/2} - 0.35\right\}^4\right]^{-0.2}$$
$$= 0.992$$

$$b_{eff} = 0.992b = 0.992 \times 37.8$$
$$= 37.5\text{mm}$$

For the lips

$$K_2 = 0.425$$

$$p_{cr} = 185000\,K\left(\frac{t}{b}\right)^2 = 185000 \times 0.425 \times \left(\frac{1.6}{7.4}\right)^2$$
$$= 3675.67\text{N/mm}^2$$

$$f_c/p_{cr} = 450 / 3675.67$$
$$= 0.122 < 0.123$$

Therefore,

$$\frac{b_{eff}}{b} = 1.0$$

$$b_{eff} = 7.4\text{mm}$$

$$A_{eff} = (2 \times 37.5 + 59.53 + 2 \times 7.4 +) \times 1.6 + 4 \times 14.57$$
$$= 297.208\text{mm}^2$$

$$P_{cs} = A_{eff}\,Y_s = 297.208 \times 450 \times 10^{-3}$$
$$= 133.74\text{kN} > F_c = 18.923\text{kN}$$

Ok

2.1.3 Axial Tensile Capacity

Bolt hole size $= 2 \times 5 \times 1.6$

$= 16\text{mm}^2$

A_n $= 345.02 - 16$

$= 329\text{mm}^2$

Pt $= 148.05\text{kN}$

2.2 MB 263306

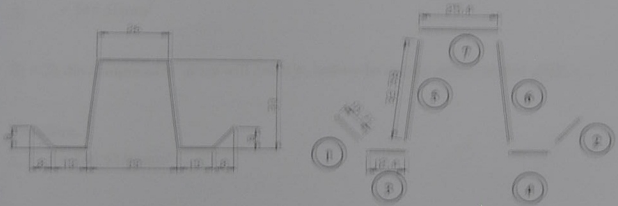

Figure 2-2 Detail and idealized model of SB 263306

* The model idealized using mid-line dimension approach

2.2.1 Bending Capacity

Loading on Purlin

Critical loading $= 2.34\text{kN/m}$

Truss spacing $= 1.5\text{m}$

Purlin spacing $= 0.3\text{m}$

Load on purlin $= 2.34/1.5 \times 0.3 = 0.468\text{kN/m}$

Moment on purlin $= 0.468 \times 1.5^2/8 = 0.132\text{kNm}$

h $= 1.28$

K_1 $= 4.4077 > 4$ for stiffened element Ok

Table 2-2 Second moment of area calculation based on effective width on compression flange

Element	A (mm^2)	y (mm)	Ay (mm^3)	$I_{element}$ (mm^4)	Ay^2 (mm^4)
1	6.43	4.00	25.70	30.81	102.82
2	6.43	4.00	25.70	30.81	102.82
3	7.44	0.30	2.23	0.22	0.67
4	7.44	0.30	2.23	0.22	0.67
5	19.55	16.50	322.54	1709.88	5321.94
6	19.55	16.50	322.54	1709.88	5321.94
7	10.80	32.70	353.16	0.32	11548.33
Total	77.63		**1054.12**	**3482.16**	**22399.19**
		$\bar{y}$ =	13.58	I_{oo} =	25881.35

Second moment of area about neutral axis is:

$I_{NA} \quad = 11567.43\text{mm}^4$

$Z_T \quad = 851.85\text{mm}^3$

$Z_C \quad = 595.61\text{mm}^3$

$Z_C < Z_T$ the compressive stress will reach p_o before the tensile stress reaches yield.

Therefore,

$p_o \quad = 540.94\text{N/mm}^2$

The moment capacity is:

$M_c \quad = p_o Z_C = 540.94 \times 595.61 \times 10^{-6}$

$\quad = 3.22\text{kNm} > 0.132\text{kNm}$ Ok

2.3 The Truss System

The truss system is design to resist the following loads:

- Concrete Roof Tiles = 0.55 kN/m^2
- Batten = 0.05 kN/m^2
- Ceiling & Frame = 0.10 kN/m^2
- M & E Services = 0.15 kN/m^2
- Live Load = 0.25 kN/m^2
- Basic Wind Speed = 30 m/s

The analysis has been done using computer program Multiframe - 3D. The loading diagram of the analysis model is depicted in the following figure:

Figure 2-3 Moment diagram of the design model

The extreme values of the truss are shown in the following tables:

Table 2-3 Deflection

	dx (mm)	dy (mm)	dz (mm)
Max	2.399	0.002	0
Min	-2.399	-5.602	0

Table 2-4 Load case 1.4DL + 1.6LL

	Px' (kN)	Vy' (kN)	Vz' (kN)	Tx' (kNm)	My' (kNm)	Mz' (kN)
Max	25.512	6.687	0	0	0	1.851
Min	-25.512	-6.133	0	0	0	-1.851

Table 2-5 Load case 1.2DL + 1.2LL + 1.2DWL

	Px' (kN)	Vy' (kN)	Vz' (kN)	Tx' (kNm)	My' (kNm)	Mz' (kN)
Max	30.792	8.041	0	0	0	2.329
Min	-30.792	-7.344	0	0	0	-2.329

Therefore, the members should able to resist the worst loading condition where:

Table 2-6

Px' (kN)	Vy' (kN)	Vz' (kN)	Tx' (kNm)	My' (kNm)	Mz' (kN)
30.792 (C)	8.041	0	0	0	2.329
-30.792 (T)	8.041	0	0	0	2.329

The deflection limitation of cantilever = Length/180 (Reference: Table 3 in BS 5950: Part 5)

$\delta_{lim} = 1411/180 = 7.84mm > \delta_{max} = 5.602mm$ **Ok**

Noticed that the 30.792kN is occurred at the end of horizontal truss member as shown in the figure below:

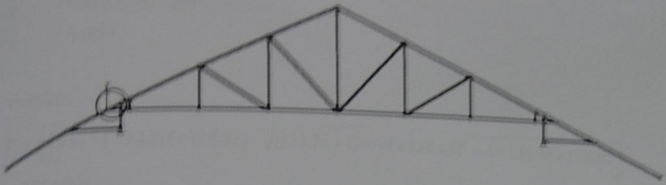

Figure 3-4 The critical axial compressive stressed member

Due to the member is connected to other member in a short length, it may be assumed adequate where the short strut capacity P_{cs} = 53.80kN is more than 30.792kN. Therefore the critical compressive loaded members are the two shown in the figure below:

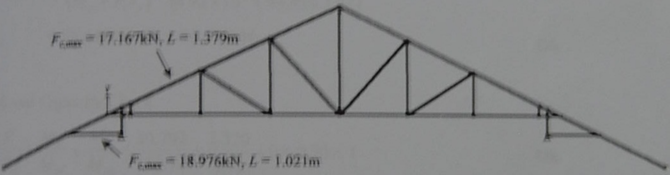

Figure 3-5 Two members to be checked their axial compressive capacity

Determination of P_c for Case where L = 1.379m and F_c = 17.167kN

I_x = 572732.10mm^4

r_x = 40.74mm

I_y = 120795.88mm^4

r_y = 18.71mm

Let $L_{EX} = L_{EY} = L = 1.379$,

$$P_{Ex} = \frac{\pi^2 \times 205 \times 572732.10}{1379^2} = 609.36\text{kN}$$

$$P_{Ey} = \frac{\pi^2 \times 205 \times 120795.88}{1379^2} = 128.52\text{kN}$$

$$\frac{L_{Ex}}{r_x} = \frac{1379}{40.81} = 33.79 < 250$$

$$\frac{L_{Ey}}{r_y} = \frac{1379}{18.71} = 73.70 < 250$$ **L_{EY}/r_y is more critical**

Ultimate loads

Since $L_{Ey}/r_y > 20$

$\eta = 0.002(73.70 - 20)$

$= 0.1074$

Therefore,

$$P_c = \frac{1}{2}\left[(133.74 + 1.1074 \times 128.52) - \sqrt{(133.74 + 1.1074 \times 128.52)^2 - 4 \times 133.74 \times 128.52}\right]$$

$= 94.85\text{kN}$

$M_c = 4.92\text{kNm}$

$e_s = 3.796\text{mm}$

$$P_c' = \frac{M_c P_c}{(M_c + P_c e_s)} = \frac{4.92 \times 10^3 \times 94.85}{(4.92 \times 10^3 + 94.85 \times 3.76)}$$

$= 88.44\text{kN} > F_c = 30.792\text{kN}$ Ok

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{30.792}{133.74} + \frac{2.329}{4.92} + 0 = 0.70 < 1$$ Ok

Overall Buckling Check

No buckling effect considered since the member is fully restraint by purlins.

Determination of P_c for Case where $L = 1.021m$ and $F_c = 18.976kN$

Let $L_{EX} = L_{EY} = L = 1.021$,

$P_{Ex} = \dfrac{\pi^2 \times 205 \times 572732.10}{1021^2} = 1111.61\text{kN}$

$P_{Ey} = \dfrac{\pi^2 \times 205 \times 120795.88}{1021^2} = 234.45\text{kN}$

$\dfrac{L_{Ex}}{r_x} = \dfrac{1021}{40.81} = 25.02 < 250$

$\dfrac{L_{Ey}}{r_y} = \dfrac{1021}{18.71} = 54.57 < 250$ L_{Ey}/r_y is more critical

Ultimate loads

Since $L_{Ey}/r_y > 20$

$$\eta = 0.002(54.57 - 20) = 0.0691$$

Therefore,

$$P_c = \frac{1}{2}\left[(133.74 + 1.0691 \times 234.4) - \sqrt{(133.74 + 1.0691 \times 234.45)^2 - 4 \times 133.74 \times 234.45}\right]$$

$$= 117.47\text{kN}$$

$$M_c = 4.92\text{kNm}$$

$$e_x = 3.796\text{mm}$$

$$P_c'' = \frac{M_c P_c}{(M_c + P_c e_x)} = \frac{4.92 \times 10^3 \times 117.47}{(4.92 \times 10^3 + 117.47 \times 3.76)}$$

$$= 107.79\text{kN} > F_c = 17.167\text{kN}$$ Ok

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{18.976}{133.74} + 0 + 0 = 0.142 < 1$$ Ok

Overall Buckling Check

$$C_{bx} = 1.0$$

$$\frac{F_c}{P_c} + \frac{M_x}{C_{bx} M_{cx}\left(1 - \frac{F_c}{P_{Ex}}\right)} + \frac{M_y}{C_{by} M_{cy}\left(1 - \frac{F_c}{P_{Ey}}\right)}$$

$$= \frac{18.976}{117.47} + 0 + 0 = 0.162 < 1$$ Ok

2.4 Comments on the Design

The design capacities of the sections are satisfying all design requirements.

Ultimate loads

Since $L_{Ey}/r_y > 20$

$$\eta = 0.002(54.57 - 20) = 0.0691$$

Therefore,

$$P_c = \frac{1}{2}\left[(133.74 + 1.0691 \times 234.4) - \sqrt{(133.74 + 1.0691 \times 234.45)^2 - 4 \times 133.74 \times 234.45}\right]$$

$$= 117.47\text{kN}$$

$M_c \quad = 4.92\text{kNm}$

$e_s \quad = 3.796\text{mm}$

$$P_c' = \frac{M_c P_c}{(M_c + P_c e_s)} = \frac{4.92 \times 10^3 \times 117.47}{(4.92 \times 10^3 + 117.47 \times 3.76)}$$

$$= 107.79\text{kN} > F_c = 17.167\text{kN}$$ **Ok**

Local Capacity Check

$$\frac{F_c}{P_{cs}} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{18.976}{133.74} + 0 + 0 = 0.142 < 1$$ **Ok**

Overall Buckling Check

$C_{bx} \quad = 1.0$

$$\frac{F_c}{P_c} + \frac{M_x}{C_{bx} M_{cx}\left(1 - \frac{F_c}{P_{Ex}}\right)} + \frac{M_y}{C_{by} M_{cy}\left(1 - \frac{F_c}{P_{Ey}}\right)}$$

$$= \frac{18.976}{117.47} + 0 + 0 = 0.162 < 1$$ **Ok**

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The design capacities of the sections are satisfying all design requirements.

3.0 The Full Scale Test

3.1 Test Configuration

To define the actual behavior of the truss system, we are expecting that the full scale test will leads to discover the blind dot of idealized modeling, e.g. the geometrical difference in construction, connectivity of the joints, and the slenderness of the members.

Four hydraulic jacks ware set on a load distributor so that the load could be assumed evenly distributed through the four legs, onto the truss. The load cells are set between each hydraulic jack and load distributor for reading the load values. The stoppers are fixed on each leg to avoid sliding. Lateral restraint also provided to the truss.

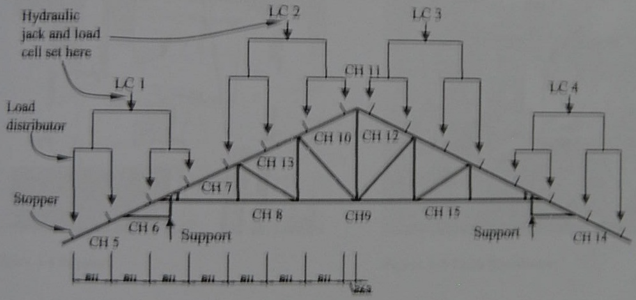

Figure 3-1 Test Specimen Rig

15 channels have been established to log both the loading and deflection data. The deflection reading was logged **two minutes** after load increment in order to get the stabilized reading.

The assumptions made under this test configuration are that:

- the load distributor distribute the load applied by hydraulic jack evenly to every legs,
- the reactions are symmetry on both site, therefore the reading on CH 5 is expected similar to the reading on CH 15; and CH 10 is similar to CH 12,
- the differences of geometry are ignored, and
- the friction of lateral restraining supports onto the truss is ignored.

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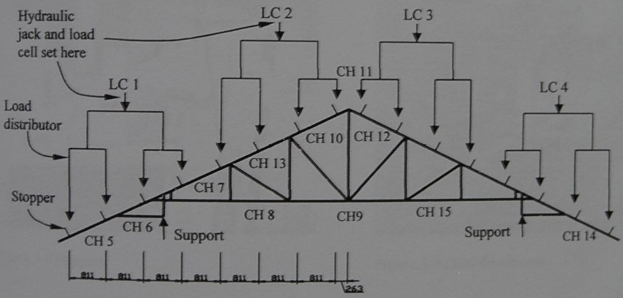

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- the friction of lateral restraining supports onto the truss is ignored.

The figures below show the testing process:

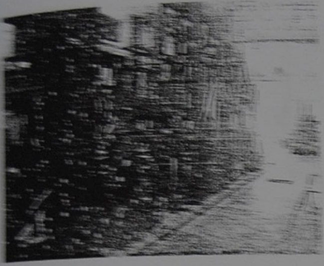

Figure 3-2 Setting of the specimen

Figure 3-3 Another view of the specimen

Figure 3-4 Support

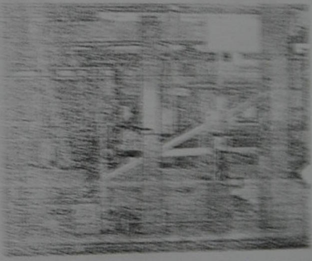

Figure 3-5 Load distributor

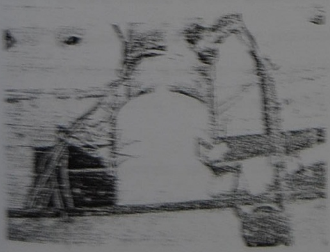

Figure 3-6 Hydraulic jack and load cell

Figure 3-7 The fixing of load distributor

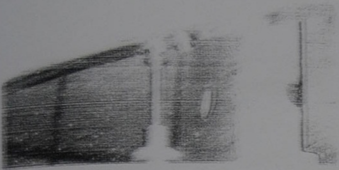

Figure 3-8 Dial gauge and the monitoring point

Figure 3-9 Dial gauge at the cantilever (wing)

Figure 3-10 The channels processor

Figure 3-11 Data Logger

3.2 Failure Mode of the Specimen

Since full lateral restrain has been provided to the specimen, the specimen deformed only in planar direction. The specimen was judged failed when the deflection reading suddenly increases significantly (yielded). There was **no visible failure** of any truss component observed.

The truss components include top chord, bottom chord, web members, apex, short strut members, joint, L-bracket and self tapping screws.

3.3 The Test Results

The truss specimen has been loading until failure. The result is plotted as below:

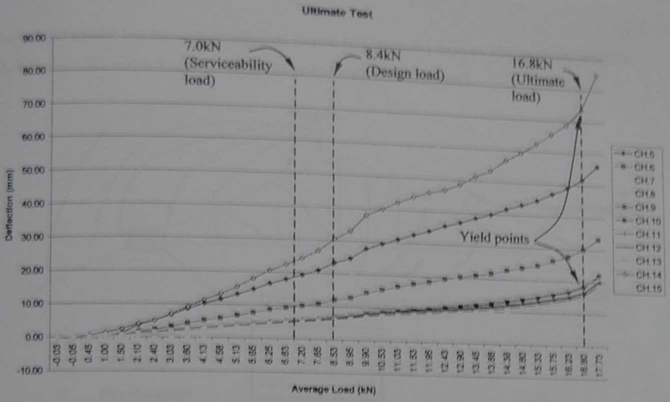

Figure 3-12 The test result

All channels show linear deformation until the second last reading. The ultimate yield point of the truss is on average loading **16.8kN** (for each hydraulic jack), which is exactly two times the design load. Four hydraulic jacks we used in the test were not applying the uniform load but the variation is within ±1.00. Therefore we use 'average load' to conclude the variations. Because of the difference of applied load, the readings of CH 5 and CH 14 at both ends which are expected equal to each other shown difference.

The rest channels in the truss body show the results which are very near to each other. The overall behavior of the truss is very near to the theoretical estimation (see session 3-12 above).

4.0 The Analysis Model of the Test Specimen (As-built Model)

An as-built model has been established as shown in Figure 4-1 below. The analysis program Multiframe - 3D which based on stiffness matrix calculation is used.

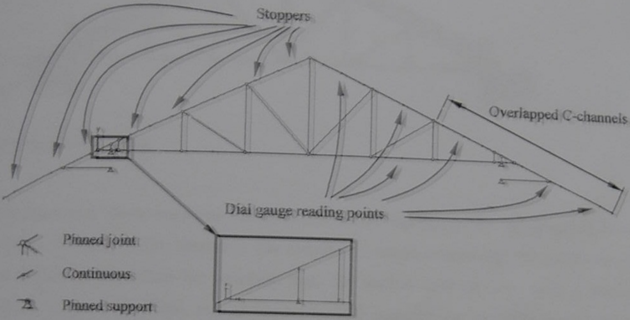

Figure 4-1 As-built model

The assumptions made in the model are:

- Plane frame analysis is utilized. We assume that the frame is not deforming laterally.
- All joints are pinned connection, except the joint jointing the wings (cantilever) to the truss.
- The supports are pin-jointed.

The ultimate load of the experiment has been applied in the analysis.

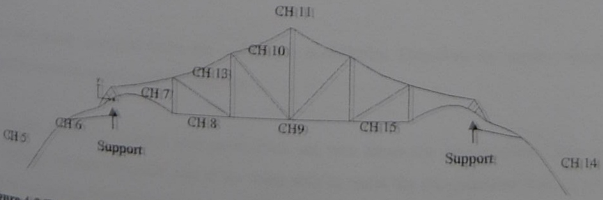

Figure 4-2 The deformed shape

The deform shape of the analyzed model is approximately similar to the actual condition. Notice on the deflection in the Figure 2-16, the ratio CH 5: CH 6: CH 7 is 6: 2: 1 and the experimental result is nearly 6: 3: 1. Moreover, the deflection of CH 13, CH 8, CH 9 and CH 15 are near to each others. Therefore we judge that the model is **valid to represent** the actual specimen.

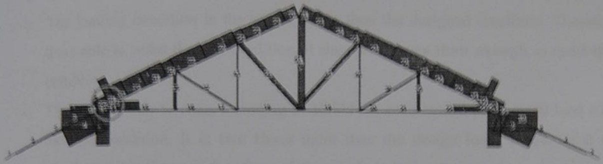

Figure 4-3 Axial force diagram

The maximum axial force occurs at the members which highlighted by the green circle in the above figure (70.40kN). The second is the member 7 which connecting the cantilever to the support (34.52kN). Therefore we conclude that, the section used at least have a short strut capacity P_{cs} value of 70.40kN; and **at least** 34.52kN capacity for an unrestrained length of 1.021m.

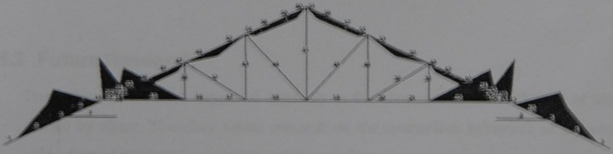

Figure 4-4 Bending moment diagram

The maximum moment from the analysis is 6.788kNm. Therefore the section should have the **least** moment resistance of 6.788kNm

Both the resultant moment and axial force are two times the design values (see session 2.3). Therefore we can conclude that if the truss able to resist the experimental load, it might able to resist the design load at 2 times magnitude.

8.0 Conclusion and Future Research

8.1 Conclusions

Some facts have been proved in this project, there are:

1. All the analysis models are **valid** to represent the actual behavior of the truss.
2. The loading condition in the test is worse than the designed condition. Therefore if the truss able to resist the test condition, it should be **more than enough** to resist the design condition.
3. The truss is able to resist a loading of 16.8kN at each loaded point (total load 67.2kN) in the test condition. It is **two times** more than the design load. Therefore it could be concluded that the capacity of truss is very **safe** to be implemented.
4. There are no visible failure observed during the yielding of the truss, therefore the components are expected have a **ductile behavior**. The ductile behavior is favorable in term of safety since it delays the significant collapse of the structure.
5. The failure of the truss happened at the connections and it was signed by shear failure of the screws.

From all stated facts above, we conclude that the truss is having the adequate strength to resist the design loads, and it is safe to be implemented in the intended uses.

8.2 Future Research

The failure of the truss happened suddenly at the connections by the failure of the screws caused by shear. Therefore future research on the connection behaviors of the truss using cold - formed sections is needed in order to analyze:

1. The behavior of thin cold-formed webs acted like the blades that can cut off the screws on shear acts and can cause sudden failure of the truss.
2. The effect of ex-centricity to the connections that can create torsional moment to the screws which could stimulate the failure of the connections.

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6.0 References

1. **BS 5950: Part 5: 1987** "British Standard: Structural Use of Steelwork in Building, Part 5. Code of practice for design of cold formed sections"

2. **Design of Cold Formed Steel Members**, Edited by J. Rhodes, 1991, London.

3. **Mechanical fasteners for use in steel sheeting and sections**. European Convention for Constructional Steelwork, ECCS-TC7, Publication No. 35, June 1983.

4. **The design and testing of connections in steel sheeting and sections**. European Convention for Constructional Steelwork, ECCS-TC7, Publication No. 21, May 1983.