EXPERIMENTAL INVESTIGATION ON THE ANALYSIS AND DESIGN OF TRUSS WITH LIGHT GAUGE COLD FORMED STEEL SECTION

K.Jothibaskar[1], Dr. D.Shobarajkumar[2]
[1]Assistant Professor, Knowledge Institute of Technology-Salem
[2]Associate Professor, Government College of Engineering -Salem

Abstract
The main purpose of this study is to analyze the steel roof truss with cold formed steel section under the normal permeability condition of wind according to Indian Standard Code IS: 875(Part 3)-1987, in which, intensity of wind load is calculated by considering a class of structure, terrain, height and structure size factor, topography factor, permeability conditions and compare the results so obtained with the calculations. By using cold formed system economy is achieved and completion of project is done in minimized time. In this project the detailed analysis of an industrial building with cold formed concept is carried out. This work is also extended by taking the parametric studies too. The numerical study of the cold form steel truss is carried out by Abacus 6.10. Design for maximum limit strength of truss is calculated using Indian Standard (IS 801-1975).
Key words: ABAQUS, COLD FORMED STEEL, TRUSS, FEM, DSM

1.INTRODUCTION

A roof truss is basically a framed structure formed by connecting various members at their ends to form a system of triangles, arranged in pre-decided pattern depending upon the span, type of loading and functional requirements .In industrial buildings, steel trusses are commonly used.

A-shaped truss: this is a type of truss that has a certain general shape resembling the letter “A”.

The steel truss has been designed as simply supported on columns. The analysis of A-type truss has been done on the basis of relevant Indian standards for the following different parameters:

• Span length of A-type trusses (metres)= 15
• Spacing between trusses (metres) = 4.0
• Roof slope=1 in 3,
• Column height = 8(metres)
• Wind zones = i, ii and iii
• Permeability = Normal

The design of industrial building is governed mainly by functional requirements and the need for economy of construction. In cross-sections these buildings will range from single or multibay structures of larger span when intended for use as warehouses or aircraft hangers to smaller span buildings as required for factories, assembly plants, maintenance facilities, packing plants etc. The main dimensions will nearly always be dictated by the particular operational activities involved, but the structural designer’s input on optimum spans and the selection of suitable cross-sections profile can have an important bearing on achieving overall economy. An aspect where the structural designer can make a more direct contribution is in lengthwise dimensions i.e. the bay lengths of the building. Here a balance must be struck between larger bays involving fewer, heavier main components such as columns, trusses, purlins, crane beams, etc. and smaller bays with a large number of these items at lower unit mass. An important consideration in this regard is the cost of foundations, since a reduction in number of columns will always result in lower foundation costs.

2.ANALYSIS as per IS: 875(Part 3)-1987.

The steel trusses have been analyzed as simply supported on columns. The support at one end is assumed to be hinged and the other end on rollers for the purpose of analysis. The truss has been analyzed for dead load, live load and wind load according to IS: 875(Part 3)-1987.

METHOD

Wind load calculation according to IS: 875(Part 3)-1987

Design Wind Speed (V_z) - Design Wind Speed depends upon a) Risk level (b) Terrain roughness, height and size of structure; and c) Local topography. It can be mathematically expressed as follows:

$V_Z = V_b k_1 k_2 k_3$

Where,

V_z = design wind speed at any height z in m/s

V_b=basicwind speed in m/s

k_1= probability factor (risk coefficient) given in Table 1 of IS: 875(Part 3)-1987,

k_2= terrain, height and structure size factor and

k_3= topography factor.

Basic Wind Speed (V_b) - Basic wind speeds have been worked out for a 50 year return period.

TERRAIN, HEIGHT AND STRUCTURE SIZE FACTOR (k_2)

The buildings/structures are classified into the following three different classes depending upon their size:

Class A- Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) less than 20 m.

Class B - Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) between 20 and 50 m. Class C - Structures and/or their components such as cladding, glazing, roofing, etc, having maximum dimension (greatest horizontal or vertical dimension) greater than 50 m.

Terrain: Category 1 - Exposed open terrain with few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5m.

Category 2 - Open terrain with well scattered Obstructions having heights generally between 1.5 to 10 m.

Category 3- Terrain with numerous closely spaced obstructions having the size of building-structures up to 10 m in height with or without a few isolated tall structures.

Category 4 - Terrain with numerous large high closely spaced obstructions.

TOPOGRAPHY FACTOR (k_3) –

The effect of topography will be significant at a site when the upwind slope (I) is greater than about 30, and below that, the value of k_3 may be taken to be equal to 1.0.

The value of k_3 is confined in the range of 1.0 to 1.36 for slopes greater than 30. It may be noted that the value of k_3 varies with height above ground level, at a maximum near the ground, and reducing to 1.0 at higher levels.

Design Wind Pressure (P_z) - The design wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind velocity:

$P_z = 0.6\ V_z^2$

Where,

P_z = design wind pressure in N/m^2 at height z, and

V_z = design wind velocity in m/s at height z.

WIND PRESSURES AND FORCES ON BUILDINGS/STRUCTURES

Wind Load on Individual Members – For clad structures, it is necessary to know the internal pressure as well as the external pressure. Then the wind load, F, acting in a direction normal to the individual structural element or cladding unit is:

$F= (C_{pe} – C_{pi}).A.P_z$

Where

C_{pe} = external pressure coefficient,

C_{pi} = internal pressure coefficient,

A = surface area of structural element or cladding unit, and

P_z= design wind pressure

3.DESIGN EXAMPLE

Plan area = 15 .0 m X 40.0 m

Roof truss span = 15.0 m

Roof slope=1 in 3

Height of column = 8.0 m

Type of roofing = A.C. Sheeting

Location of shed = Salem

Type of truss = A-type

Permeability= Normal

Topography = θ less than 3^0

Spacing of trusses= 4m

1. Selection of configuration:

Let a pitch of 1/5 be provided

Height of truss = 1/5 x 15 = 3m

Slope of top chord = tan-1 (3/7.5)

= 21.8^0

If purlins are to be placed on top panel point only, panel length should be around 1.4m so that sufficient lap can be provided when 1.65m A.C sheets are used.

Length of top chord $7.5^2 + 3^2 = 8.078$m

If we select 6 panels, length of panel = 80.78/6 = 1.346 say 1.35m

2. Loads:

Dead load:

Wt.of sheeting including laps and connections

= 170 N/m^2

Wt.of purlins = 120 N/m^2

Self wt.of truss = 20 + 6.6L

= 20 + 6.6 x 15 = 120 N/m^2

Total dead load = 170 + 120 + 120

=410 N/m^2

Each purlin takes care of an area

= 1.35 + 4m^2

Load on each intermediate panel point

= 410 x 1.35 x 4

= 2214 N

= 2.214 KN

Load on shoe: Taking 450 mm roof projection load

= 410 x (1.35/2+ 0.45/2) x 4

= 1476 N = = 1.476 KN

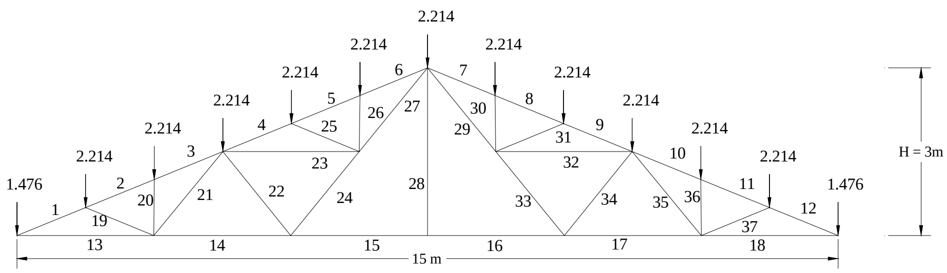


Figure 4.2:Dead load

Live load :

LL = 750 – (21.8 – 10) x 20

= 514 N/m

LL on intermediate panel point

= 514 x 3.14 x 4

= 2776 N

= 2.776 KN

LL on shoe = 514 x (1.35/2 + 0.45/2) x 4

= 1850 N

= 1.850 KN

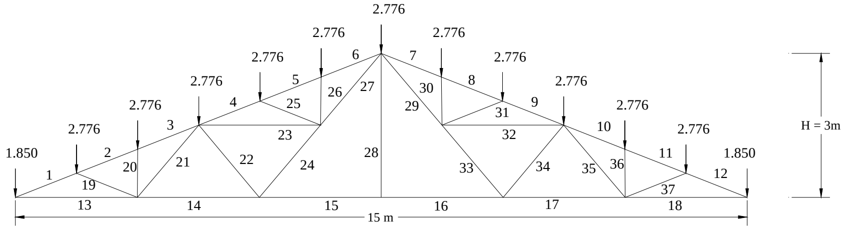


Figure 4.3 : Live load

Wind load:

Basic wind velocity near Salem = 47 m/sec.

$k_1 = 1.0$

k_2 for category 2, class B building with height 8m,is 0.98

$k_3 = 1.0$

Design wind speed = V_z

$= 1.0 \times 0.98 \times 1.0 \times 50$

$= 47$ m/sec

Design wind pressure

$P_d = 0.6 \times 492$

$= 1440$ N/m^2

Wind pressure Coefficient :

h/w = 10/15 = 2/3

Thus 1/2 < h/w 3/2

From table

When Wind load angle 0^0, for rafter slope 21.8^0 [wind normal to ridge]

On windward side $C_{pe} = 0.7+(1.8 /10 \times 0.5) = -0.61$

On leeward side $C_{pe} = -0.5$

When wind angle 90^0, for rafter slope 21.8^0 [wind parallel to ridge]

On windward side $C_{pe} = - 0.8$

On leeward side $C_{pe} = - 0.6 – 1.8/10 \times 0.2$

$= -0.636$

Internal wind pressure coefficient :

For a building with medium permeability

$C_{pi} = 0.5$

Design wind pressure on wind ware side

$= (-0.8 – 0.5)P_d$

$= 1.3 \times 1440$

$= - 1872$ N/m^2

(a) Intermediate panels

$= - 1.872 \times 1.35 \times 4$

$= - 10.110$KN

(b) At crown joint = - 5.050 KN

(c) At shoe = - 1.872 {1.35 x 0.450/2} x 4

= - 6.74 KN

Design wind load on leeward side

$= (-0.636 – 0.5) \times 1440$ KN/m

$= -1635.8$ N/m^2

(a) Intermediate panels

$= - 1.636 \times 1.35 \times 4$

$= - 10.110$KN

(b) At crown joint = - 4.415 KN

(c) At shoe = - 1.636 {1.35 + 0.450/2} x 4

= - 5.9 KN

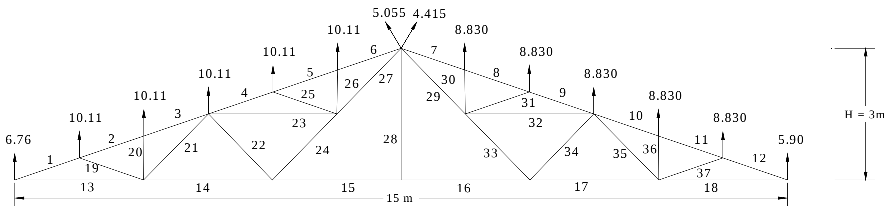


Figure 4.4 : Wind load

3.Analysis :

The truss is analysed for dead loads as show in figure.4.1and dead load force in various members are entered in table 4.1. Since live load are in direct preparation of live load in the ratio 4
514/410, the force in various members due to live load are found by ratio 514/410 and are listed in table 4.2. Live load figure 4.2.

Wind load analysis is carried out for the loads as shown in figure 4.3. And the member force are entered in tables.4.3

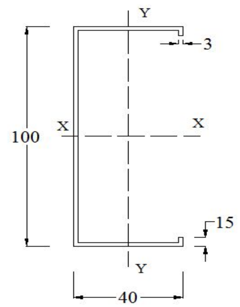
Sl.No	Group	Member	DL + LL in KN	DL + WL in KN
1	I	1	110.698	151.14
2		2	100.734	139.86
3		3	100.799	146.03
4		4	95.752	143.32
5		5	85.749	131.99
6		6	85.871	138.26
7		7	85.891	130.92
8		8	85.759	125.43
9		9	95.762	134.76
10		10	100.842	136.89
11		11	100.769	131.49
12		12	110.762	140.91
13	II	13	102.758	136.52
14		14	84.151	104.14
15		15	56.266	55.643
16		16	56.266	55.643
17		17	84.166	96.442
18		18	102.829	123.73
19	III	27	36.638	63.702
20		24	21.905	38.093
21		29	36.671	53.622
22		33	21.924	32.059
23	IV	22	21.817	37.932
24		34	21.817	31.902
25	V	23	18.627	32.384
26		32	18.618	27.222
27		21	14.687	25.534
28		35	14.719	21.52
29	VI	19	9.943	17.326
30		20	7.549	13.109
31		25	10.009	17.409
32		26	7.553	13.124
33		37	9.985	14.612
34		36	7.551	11.035
35		31	10.009	14.638
36		30	7.578	11.082
37		28	0	0

Maximum Tensile load = **110.698 kN**

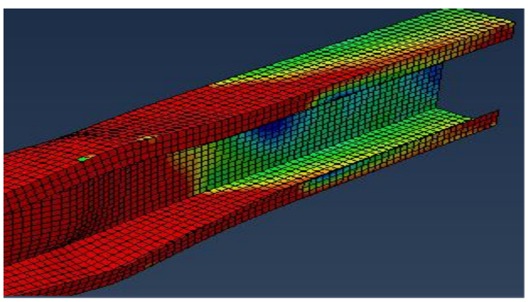
Maximum Compression load = **151.14 kN**

4.FINITE ELEMENT MODELLING &VALIDATION:

Finite element software, ABAQUS version 6.10 is used for the numerical study. The model is generated based on the centre line dimensions of the cross-sections. A linear buckling analysis is performed prior to the post-buckling analysing of the section, failure mode shape for the first mode is considered. Following this imperfection factor incorporated for nonlinear post-buckling analysis and the load versus end-shortening characteristics were plotted and the ultimate load carrying capacity of section is calculated. S4R5 shell elements in ABAQUS are used. Convergence studies have been carried out on the column in order to determine a suitable finite element model for the analysis. The aspect ratio (length to width) of 1.0 for the flange and web elements was used. The hinged end conditions were used in the verification model as well as in the parametric study models. The pin-end conditions of the columns were modeled with both the ends prevented from rotation about the z-axis. The translation in x, y and z direction was arrested at the unloaded end. But in the loaded end was prevented from translation in x and y directions. These boundary conditions were applied to the independent node of the rigid fixed Multi Point Constraint located at the upper and lower end of the model in fig. 2. Dependent nodes are connected to the independent node using rigid beams and all six structural degrees of freedoms are rigidly attached to each other. In this model, the independent node was located at the geometric centre of the cross-section. The displacement control load was applied in increments to the master node using the modified RISK method available in the ABAQUS library The geometric and material non linearity were considered in the model. In order to account for the Elasto-plastic behaviour, a bilinear stress-strain curve is adopted, having a modulus of elasticity (Et) of 200000 N/mm2 and yield stress 230 N/mm2 and Poisson ratio as 0.3 for the model. Elastic perfectly plastic material model is used for parametric study. The initial geometric imperfections were considered in the non-linear analysis. Eigen mode 1 is scaled by a factor (0.006*w for local, 1*t for distortional and l/500 for global) of the plate thickness of the sections was used in modelling the geometric imperfections of the columns in the verification model. Fine element method (ABACUS) procedure is validated through the results available in the literature [4]. Table 1 shows the comparison of FEA results with test results of literature [4]. The mean for PEXP to PFEM is 1.01 and Standard deviation is 0.02. Therefore good agreement is in-between them. In Fig. 3 shows the association of the deformed shape of the column. The simulation in ABACUS is same as that obtained from experimental results. Based on the comparison of ultimate load and deformed shape the numerical method is validated.



100CS40X15



The buckling load find from abaqus software
= 42.9kN

5.EXPERIMENTAL ANALYSIS

The laboratory testing of the C sections was carried out in accordance withthe code of practice. These test results need to be verified with the formula asdescribed in the code depending on the usage of the section in the roof truss system.

PROTOTYPE

It is possible to use prototype testing to reduce the risk that a design may not perform as intended, however prototypes generally cannot eliminate all risk. There are pragmatic and practical limitations to the ability of a prototype to match the intended final performance of the product and some allowances and engineering judgement are often required before moving forward with a production design. Scale 1:2.7

COMPRESSION TEST

The specimens represent the capacity of the sections which failed due to local buckling. The code specifies that the length of the specimen of this test may be adopted as three times the width of the longest element of the specimen. Therefore, the test was set as shown in Figure 2. Since the specimens were susceptible to local failure, a solid block of steel plate was installed at each end of the specimens in order to avoid the end crushing and promote the failure at the middle length. Three specimens were tested in this test

Sl.No	Specimen	Buckling load kN
1	100CS40X15	41.2 kN
2	100CS40X15	40.9 kN
3	100CS40X15	41.6 kN

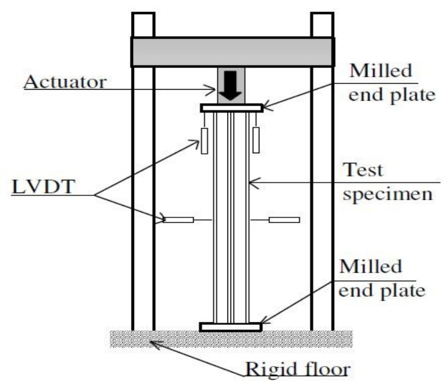


Figure 2

All specimens were fully deformed after test and the minimum failure load was 40.9 kN

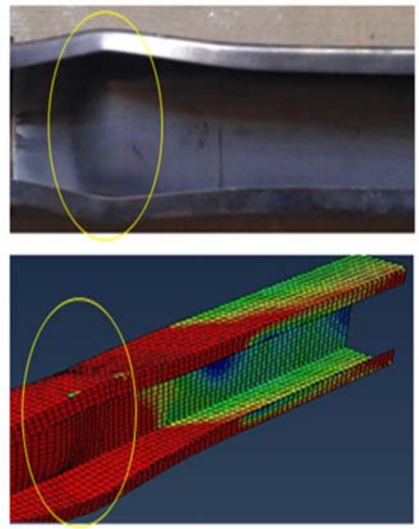
6.ANALYTICAL ANALYSIS

The capacities of cold-formed C-sections were estimated using themethods providedIS:1608-2005 (Part.1). Coupon tests were carried out to determinethe yield strength of the specimens which have thickness namely 3 mm. The determination of the yield strength is necessary because this value is applied into the capacities calculation of the tested sections. Three tests were conducted 3 mm thick plate.
The configuration of the test is shown in Figure.1

The design yield strength was adopted at around 85% of the characteristic yield strength as in accordance with the code.it was determined that the average yield strength for a 3 mm thick plate was 240 N/mm^2. The yield strength decreased as the thickness increased, which is in line with the expected values as described by IS:1608-2005(Part.1).The thinner specimen tends to possess a slightly finer grain structure as a result of faster cooling during the formation of the plate material. As a result, a higher yield strength will be achieved for thinner specimen.The capacities of the specimens are later calculated based on these yield strength values.

COMPARE THE ABAQUS RESULTS WITH EXPERIMENTAL RESULTS

Tested result –40.9 kN
Abaqus result – 42.9 kN



Sl.No	Specimen I D	Length (mm)	PFEM (kN)	PTEST	PDSM (kN)	PFEM/PDSM	PTEST/PDSM
1	BC100CS40X15	500	42.9	40.9	43.75	0.98	0.93

Figure 1

Area of Coupon= 12.5 X 3 = 37.5 mm^2
Area of Specimen = 630 mm^2
= 630 x 240 = 151.2 kN

8.CONCLUSIONS

The experimental testing of the proposed cold-formed steel C-section has been carried out successfully and the results showed good agreement with the theoretical values. From the study, further conclusions can be drawn as follows:

(1) The experimental tests results are only valid for the failure mode and capacities of the specimens specifically mentioned above. The results should not be generalised for all shapes of cold-form steel sections.

(2) The experimental results showed that the actual capacities of the specimens that represent the member of roof truss can be predicted and validated.

Therefore, it can be concluded that the capacities of the tested specimens can be used in the actual design of the member for roof truss system.

(3) No modification is needed in the application of the specimens to the roof trusssystem. The proposed C-section of locally produced cold-formedsteel section can be safely used in the roof truss system provided that the designstrength should not be greater than the capacity strength of the sections.

Although in this study the test results showed good agreement with the theoreticalvalues, further testing need

to be done to understand the global behavior of thewhole structure. It is suggested that a full-scale testing of the proposed roof trusssystem to be carried out by assembling the proposed sections and connectionstogether, in order to gain further understanding on the failure modes and capacitiesof the whole system.

REFERENCE:

1. Sanchita. S. Nawale, Sangram Chalukya, and Admane.S.V (2014) "**Comparative Analysis and Bending Behavior of Cold form Steel with Hot Rolled Steel Section"** *American Journal of Engineering Research (AJER).*

2.Roshan S Satpute and Valsson Varghese (2012) **"Building Design Using Cold Formed Steel Section"** *International Refereed Journal of Engineering and Science (IRJES).*

3.PadmaPriya R and Kandasamy S (2010) **"Study On The Behaviour Of Cold-Formed Steelangle Tension Members"** *Twentieth International Specialty Conference on Cold-Formed Steel Structures.*

4. Pradeep V, Papa Rao G (***2014)*** "**Comparative Study of Pre Engineered and Conventional Industrial Building"** *International Journal of Engineering Trends and Technology (IJETT).*

5. Mishra U.K and Nagender A.S.V. (2007) "**Computer aided analysis and design of multi-storeyed buildings"** *National Institute Of Technology Rourkela.*