

## Cold-formed steel section building design

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**ASTARCT:** A relatively recent idea in India, cold formed steel sections are widely employed in industrial and non-industrial projects across the globe. In the 1990s, when the Indian economy opened up and a number of multi-nationals built up green-field projects, these ideas were brought to the Indian market. Local marketing agents and licenced builders have helped Global Cold formed steel create a footprint in India. There is no need to worry about compatibility issues since the whole building package is provided by a single vendor. There are several advantages to using the Cold formed construction technology, such as this: Because only bolted connections are employed, a structure may be dismantled, stored, or relocated to a new place and reassembled when no longer required. The stiff structure is sturdy despite the lack of field riveting or welding. The project may be completed in a shorter amount of time thanks to the use of a Cold formed system. In this study, an in-depth investigation of a cold-formed industrial building is conducted. Parametric investigations are taken into consideration as well. Hot Roll steel Industrial building and Cold formed Industrial building have been compared and a conclusion produced.

Stad Pro 2008 and the IS Code

### INTRODUCTION

GENERAL Function and construction cost efficiency are the two most important considerations in industrial building design. A warehouse or aircraft hangar may need a building with a bigger span than a factory or assembly plant. Conversely, a factory, assembly plant, maintenance facility, or packaging plant may require a building with a lower span. Structural designer's advice on optimal spans and the

selection of acceptable cross-sections profiles may have a significant impact on attaining overall

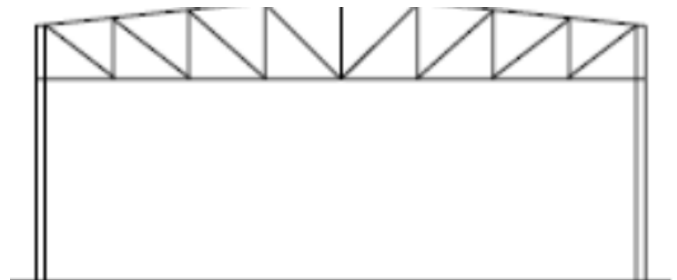
economic efficiency in the design process. The longitudinal dimensions of the structure, i.e. the bay lengths, are an area in which the structural designer may have a greater impact. As a general rule, bigger bays need fewer, heavier components like as columns and trusses as well as a higher number of these components at a lower mass per square foot. As fewer columns mean cheaper foundation expenses, that's a crucial factor to keep in mind while making this decision..

### CLASSIFICATION

- I. Hot-Rolled Steel Industrial building.
- II. Cold-Form Steel Industrial building.

#### HOT-ROLLED STEEL INDUSTRIAL BUILDING

Cross sections for a single-story Hot-Rolled Steel industrial structure are many, but experience has proven that just a few designs are the most practical and cost-effective options. Figure 1 depicts several of these cross-sections. The yield strength of the cross-sections utilised in industrial buildings made of hot-rolled steel is 250Mpa. The cross-section of a hot-rolled steel industrial building is shown in the image below..



## Cross-sections used in Hot-Rolled Steel Industrial building

### ORIGIN OF COLD-FORM STEEL CONCEPT

Historically, Cold-Form Steel (CFS) structures have been shown to satisfy a broad variety of structural and aesthetic criteria via planned assembly of structural parts. Developed in the United States in the 1960s, Cold-Form Steel construction was first introduced to India in the late 1990s. In order to address a need for low-cost, standardised housing during World War II, the best-known Pre-fabricated structure (i.e.) became a household name. These constructions can be made and destroyed using just hand tools, and they can be easily transported and reassembled in a new location. It was in the 1960s when the scientific phrase "Cold-Form Steel Buildings" was coined. The structures were "Cold-Form Steel" because they relied on conventional engineering designs for a restricted number of off-the-shelf configurations, like their forefathers. As long as the architectural specifications of the customer are followed, the structures may be referred to as Cold-Form Steel.

### COLD-FORM STEEL BUILDING PARTS

In addition to the main frame, there are also secondary frames.

- Bracing gusts of wind
- Cladding on the outside

Exterior cladding made of trapezoidal sheets. In the secondary frame, cold-formed Z or C sections are employed. The main framework is made up of I-Sections that have been constructed up. Rods with circular cross sections are used for

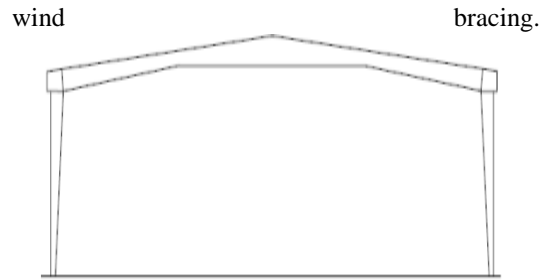


Figure 1.2 (a): Standard Cold-Form Steel portal frames

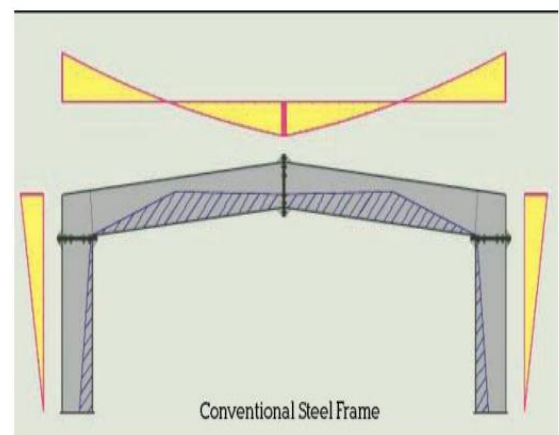


Figure 1.2(b): Conventional frame

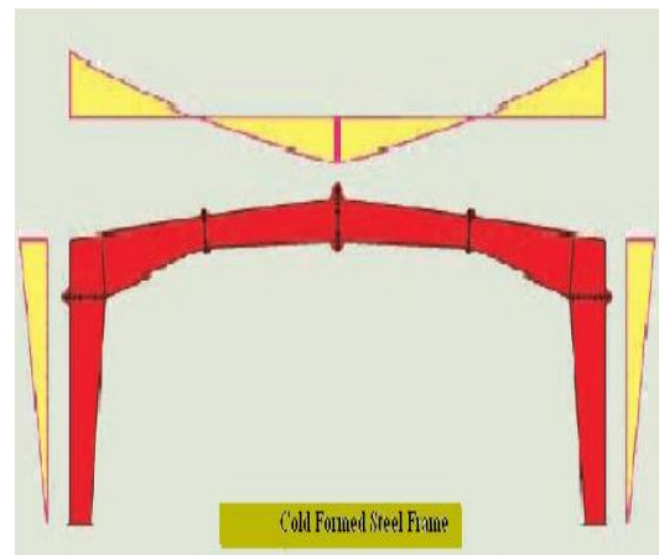


Figure 1.2(c): Cold form steel frame

Figure shows a typical single-span frame of this type. Column-rafter junction and apex high moments may be handled by the deeper section due to the rafters and columns being tapered to fit the overall form of the gravity bending moment diagram. A frame's width may be determined by using uniform flange and web thicknesses. Tapered, welded structure is more expensive to build, but the lower material content more than makes up for it. Only 75% of a comparable Hot-Rolled Steel rolling-steel portal frame is needed to make this structure. Webs may be as thin as 5mm, while flanges can be as thick as 8mm. Non-Hot-Rolled Steel design fabrication techniques need the employment of computer-aided design and detailing routines and automated shop assembly methods for the fabrication of these thin-webbed sections.

## MAJOR COMPONENTS OF COLD-FORM STEEL INDUSTRIAL BUILDING

Cold-Form Three product categories are used in steel construction: As the principal structural framing elements, the "I" section is composed of hot rolled sections with a yield strength of 345Mpa. cold formed secondary parts such purlins, eave braces and side braces with yield strength of 345Mpa are included in this category. c) Roof and wall cladding sheets with a yield strength of 345Mpa are available.]

## COMPARISON BETWEEN HOT ROLLED AND COLD FORMED STEEL SECTIONS

- Insect and fungal infections are no longer a concern, thus rotting and decomposed food is no longer an issue. Profile consistency and accuracy: The process's nature and how it's manufactured Using cold-rolling, the desired profile may be maintained and reproduced in a considerably tighter tolerance for as long as necessary. Furthermore, the cold rolling technique has less tool wear and is well-

suited to automated operation, both of which help to maintain precision. Cold-rolling allows for almost any cross-sectional form to be manufactured.Z-section with lips, for example, or C-section with lips. Pre-galvanized or pre-coated options are available: Galvanization or plastic coating may be applied to the steel.

- Both corrosion resistance and aesthetic appeal may be achieved by using different types of materials. It's best for construction on the ground: Cold-formed steel may outperform hot-rolled steel in several applications.
- In view of its ease of fabrication, which requires nothing more than people or a very light machine, It is because of the cold-forming process that local work is introduced that yield strength is increased.
- hardening in the region of created corners in the strip being made. An increase in final yield strength of up to 25% may be expected as a consequence of this local hardening. reducing the amount of stuff used: It is possible to employ an extremely thin material in contrast to a thicker one.
- Material consumption is substantially less than that of smallest hot rolled sections for a given strength or stiffness requirement. Structural characteristics may be achieved with minimal material weight by controlling the material thickness or cross sectional geometry.

## PLANNING OF COLD-FORM STEEL BUILDING FOR INDUSTRIAL PURPOSES

- **GENERAL** An industrial building's design is based on its functional needs, or the activities that will be carried out there. For an industrial building, it is important to take into account factors such as large primary frame areas, large height and wide doors and windows; large primary frame spans; and lighting and sanitary arrangements in order

to minimise the weight of the primary frames and other structural components. Prior to the design phase, the location of a planned plant is almost always predetermined. Preliminary plans should be discussed with the designer ahead of time. This provides the designer with the chance to choose an appropriate location while taking future advancements into account. The following are some of the elements that have a role in the choosing of a location: The location should be near a major thoroughfare.

It includes infrastructure like water and electricity; topography; and water drainage.

- The foundation design in relation to the soil condition
- There should be enough storage space for raw materials and completed goods.
- There should be enough room for transportation facilities to supply raw materials and collect the finished goods. Products that have been completed.
- Wastewater treatment plants.

#### **PRIMARY COLD-FROM STEEL FRAME**

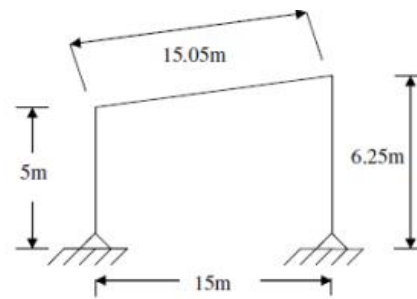
If a Cold-Among Steel building system is chosen for the project, the following step is to pick from the several varieties of Cold-From Steel main frame. In Cold-from-Steel structures, the basic frame, or "backbone," is critical to a successful application of the design procedures that follow. Main framing is influenced by a variety of elements, some of which are as follows: The building's width, length, and height are listed below.

- Slope of the roof
- Clear spans are required.
- The building's occupants' opinions on the exposed steel columns' acceptability.
- Materials to be used for the roof and walls.
- There are now five main varieties of Cold-Formed Steel frames on the market: Beam with a curved profile. Rigid single-span structure. Multi-span rigidity.
- A frame for a lean-to.
- Trusses with a single span and a continuous span.

Girts and eave struts are both considered part of the "frame width" measurement. Column interior faces are separated by a "clear span," which is measured in millimetres. Between the bottom of the column base plate and the eave strut, "eave height" is measured. It is the distance from the floor to the lowest point of a construction known as "clear height."

#### **SECONDARY FRAMING**

The Cold-From-Steel construction systems' secondary structural components connect the major building frames. In addition to supporting the roof and walls and transporting external loads to the main structure, they have a multifaceted function. As flange bracing for main frame and as part of the building's lateral load-resistant system, secondary structural components are sometimes referred to. As part of horizontal roof diaphragms, purlins and girts are typically used as secondary parts of the wall bracing assemblies, respectively. Known as the eave strut, eave purlin, or eave girt, this third form of secondary structure serves as both a girt and purlin for the roof. There are structural similarities among girts, purlins,



**Figure 3.1.1: Plan of CFS**

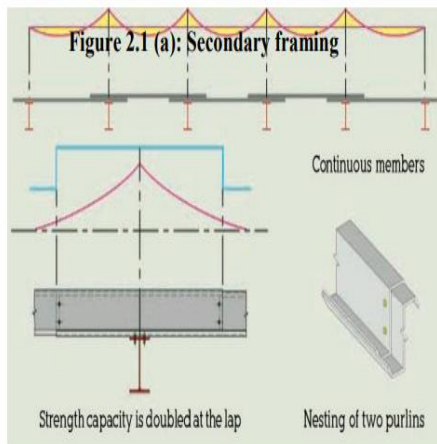
$$\tan \theta = \frac{1}{12}$$

$$\theta = \tan^{-1}\left(\frac{1}{12}\right)$$

$$= 4.76$$

$$\cong 5^\circ$$

15m



Overlapping of secondary members

### CASE STUDY 3.1 COLD FORMED STEEL CONCEPT FOR INDUSTRIAL BUILDING 3.1.1

#### INTRODUCTION

Spans of 10 to 18 metres are common in Cold-formed steel Industrial buildings. Industrial building slopes are offered in 1:10, 1:12 and 1:20 profiles. Building Parameters for Cold-Formed Steel In the city of Nagpur, India Cement Godown Building dimensions: 15m wide. a 50-meter-long building 5 m eave height, c/c of main frames: 6.25 m Purlins may be spaced up to 1.5 metres apart. At 1:12 Aria, the slope of the roof Areas Protected: 15 m 50 m

**Figure 3.1.1: Plan of CFS**

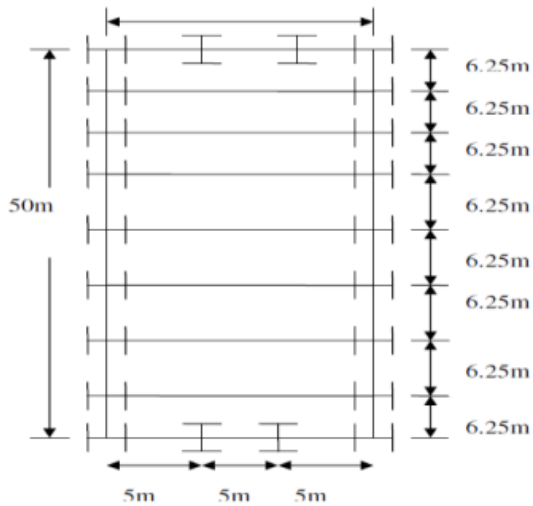
$$\tan \theta = \frac{1}{12}$$

$$\theta = \tan^{-1}\left(\frac{1}{12}\right)$$

$$= 4.76$$

$$\cong 5^\circ$$

15m

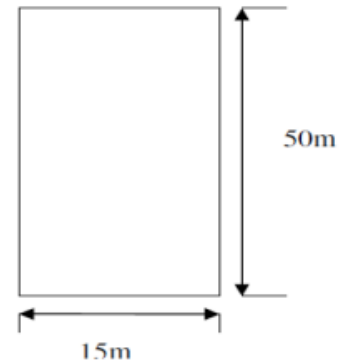


**Figure 3.1.2: Elevation of CFS**

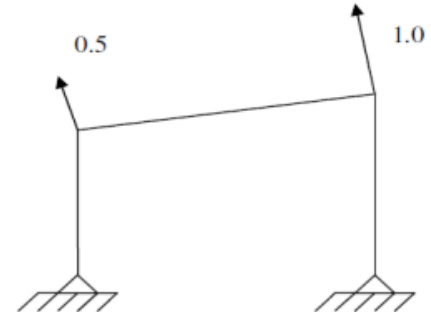
**LOADING**

INFRASTRUCTURED WORKLOAD  
 Table 2 of the IS states that for a slope of less than 10 degrees, the UDL on the roof measured on the plan area equals 75 kilogrammes per square metre. Cold Formed Steel Sections for Wind Load Building Design B www.irjes.com Page 6 of 6 v2 = vb k1 k2 k3 according to IS 875 (part3) value 5.3 assuming vb = Nagpur as a location Table 1 of IS 875 specifies that the basic wind speed (vb) is 44 m s k1 = 1 coefficient (part3) Table 2 of IS 875 shows that k2 = 0.88. (part3) topographical factor k3 is equal to one. Design wind speed v2: 44 1 0.88 1 = 38.72 m s; pz = 0.6 vz 2 = 0.6 38.72 2 = 900 m s 3.1.4 is the specified

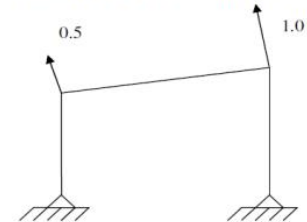
design wind pressure. To calculate the external pressure coefficient (CpE) Roof According to IS: 875, Table 6 (Part3) H is 5m; w is 15m in this example The roof angle is 5 0.



**Figure 3.1.3: Building Plan**



**Figure 3.1.3: Building Plan**



**Figure 3.1.4: Wind Across Length Of Building (Roof)**



**Figure 3.1.5: Wind Along Length Of Building (roof)Wall**

Referring to Table 4 of IS: 875(Part3)  $l/w = 5/15 = 1/3 < 1/2$   $l/w = 50/15 = 3.33 < 4$

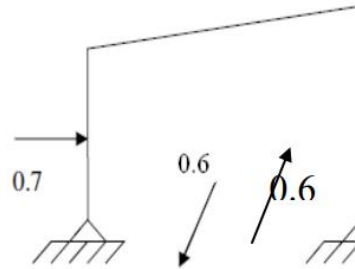


Figure 3.1.6: Wind across Length Of Bu

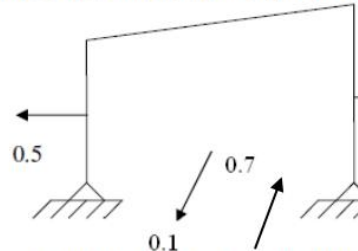


Figure 3.1.7: Wind Along Length Of Bu

PURLIN DESIGN considers apertures to be less than 5% of the total area to have an internal pressure coefficient of 0.2 Unit wt. /m of sheeting at  $0.06 \text{ kn/m}^2 = 0.06 \cdot 1.5 = 0.09 \text{ kn/m}^2$  dead load. Purlin's unit wt./m or self wt. is  $0.07 \text{ KN/m}$ . There is a total dead load of  $0.16 \text{ KN/m}$  on each Purlin metre. impedance on purlin =  $0.75 \text{ K N/m}^2$  b) Imposed Load  $1.125 \text{ KN/m}$  c) Wind Load Maximum Wind Load per metre on each Purlin =  $(1 + 0.2) \cdot 1.161 \cdot 1.5 = 1.62 \text{ KN/m}$  d) Total Imposed Load The Combination is loaded. Load Combination 1: Dead and Imposed Loads:  $0.16 + 0.125 = 1.285 \text{ KN/m}$  Dead weight plus wind weight equals  $1.46 \text{ KN/m}$ . Select z from the drop-down menu.

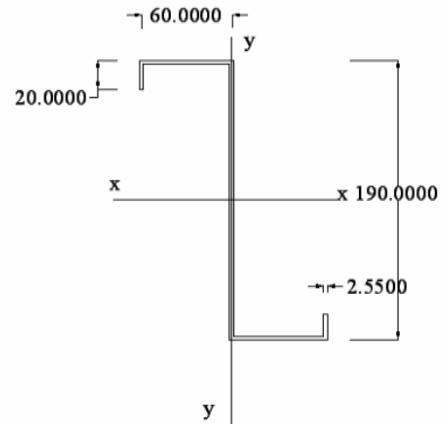


Figure 3.1.8: Z section

Table 3.1.1: Property of the Z-section selected is as shown below

Area	Thickness	Wt/m	$I_{xx}$	$I_{yy}$	$Z_{xx}$	$Z_{yy}$
8.37 sq.m	2.55 mm	6.57 kg/m	439 cm <sup>4</sup>	58.4 cm <sup>4</sup>	46.21	10.01

Checking the above section based on Section 9 of BS: 5950 Part5-1998 1. Overall Depth  $< 100 t$   $190 \text{ mm} < 100 \times 2.55$   $190 < 255$  (Ok) 2. Overall Width of Compression Flange/ Thickness i.e.  $B/t < 35$   $60/2.55 = 23.52 \text{ mm} < 35 \text{ mm}$  (O.K.) 3. Width of Lip  $> B/5$   $20 \text{ mm} > 60/5$   $20 \text{ mm} > 12 \text{ mm}$

**Checking the above section based on S**

1. Overall Depth < 100 t  
 $190\text{mm} < 100 \times 2.55$   
 $190 < 255$  (Ok)
2. Overall Width of Compression Flange  
 $60/2.55 = 23.52\text{mm} < 35\text{mm}$  (O.
3. Width of Lip > B/5  
 $20\text{mm} > 60/5$   
 $20\text{mm} > 12\text{mm}$

**Checking the above section based on I**

- $W/t = (60 - 2 \times 5.55)/2.55 = 21.52$   
 5 Minimum Overall depth required as

$$f_y$$

$$= 15\text{m}$$

$4.8t = 4.8 \times 2.55 = 12.24 < 15\text{mm}$  (OK)

6 Calculation for laterally unbraced beam  
 Calculation of effective design stress according to IS: 801-1975

When you multiply 4.8 by 2.55, you get a result of 12.24 millimetres (OK) 6 Beams without lateral bracing: calculation In accordance with IS: 801- 1975, the calculation of the compressive element's effective design width  $w_t$   $\lim = 1435$  f ft. In this case,  $f = 0.75 \times 1600 \text{ kg/cm}^2 = (w_t) \lim = 1435$  We get  $W = 41,4 \times 2,55$ , which is equal to  $(60 - 22,55) 105,57 > 54,9$ . (OK) According to clause 6.3 (b) of IS: 801-1975, a complete flange is effective in compression.  $S_{xc}$   $d_{lyc}$  The member's unbraced length is 2.08 m. (considering sag rods at spacing 2.08m) Compression part of a section around the gravity axis of the whole section parallel to the web =  $I_{yc}$  = Moment of inertia of the compression portion of the section about the gravity axis

of the entire section

$$E = 2 \times 10^6$$

$$f_y = 3450$$

$$D = \text{Depth of the section} = 190\text{mm}$$

$$\frac{I^2_{sxc}}{d_{lyc}} = \frac{208^2 \times 46.21}{19 \times 29.4}$$

$$= 3579 \rightarrow (1)$$

$$\frac{0.18\pi^2 Ecb}{f_y} = \frac{0.18 \times \pi^2 \times 2 \times 10^6 \times 1}{f_y}$$

$$= 1030 \rightarrow (2)$$

$$\frac{0.9\pi^2 Ecb}{f_y} = \frac{0.9 \times \pi^2 \times 2 \times 10^6 \times 1}{f_y}$$

$$= 5150 \rightarrow (3)$$

(1) > (2)  
 (1) < (3)

Hence  $F_b = \frac{2}{3} f_y - \frac{f_y^2}{2.7\pi^2 Ecb} \left( \frac{I^2_{sxc}}{d_{lyc}} \right)$

$$F_b = \frac{2}{3} 3450 - \frac{3450^2}{2.7\pi^2 \times 2 \times 10^6 \times 1} (3579)$$

$$= 1500.7 \text{ kg/cm}^2$$

$$F_b = 150.7 \text{ N/mm}^2$$

Referring Clause no. 6.1 @ IS 801-1975

F = Basic design stress =  $0.6 f_y$   
 $= 0.6 \times 3450 = 2070 \text{ kg/cm}^2$

Hence safe =  $207 \text{ N/mm}^2$

Hence  $F_b = 150.07 \text{ N/mm}^2$

Since here Wind load condition is critical,

$$F_b = 1.33 \times 150.07 = 199.59 \text{ Mpa}$$

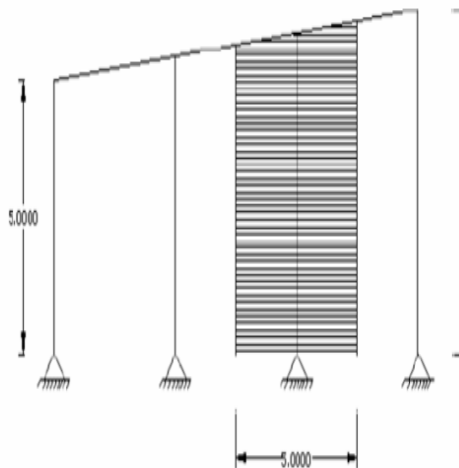
$$F_b \text{ act} = \frac{M}{Z_{xx}} = \frac{5.84 \times 10^6}{46.21 \times 10^3}$$

$$= 126.37 < 199.59 \text{ Mpa}$$

According to BS 5950, do a Deflection test. For the purlin, the permissible deflection is  $6250/240 = 26.06\text{mm}$ , which corresponds to  $6.25 \times 1.125 \times 6.25 = 7.03 \text{ KN}$ . No deflection was observed in this case when checking for deflection due to the imposition of the load on the purlin. b) Calculated Deflection due to the Imposed Load on Purlin  $EI: 5wl^3/384EI = 5wl + 7.03 + 10.3 + 6250 + 3.384 + 2 + 105 + 439$  According to Clause No. 6.4 of the IS: 801-1975, the shear stress in the web may be calculated using the following formula:  $(190 + 2 \times 2.55) = 178.9\text{mm}$  In order to meet the maximum permissible shear stress,  $f_v = 1275 \text{ kg/cm}^2$   $f_y = 0.04 f_y = 1380 \text{ kg/cm}^2$



1275 1380 70.15 = 67.51N/mm<sup>2</sup> for the fv  
 Dead Load + Wind Load Shear Stress = 1.46  
 KN/m 20 N/mm<sup>2</sup> = 20 Mpa 67.51 51N  
 /mm<sup>2</sup> = 20 x 1.33N /mm<sup>2</sup> =20 x 89.78 Mpa  
 c) Actual Shear stress = 1.46 103 625 178  
 2.25 = 20 N /mm<sup>2</sup> Inspect the web for  
 combined shear and construction. Referring  
 to IS 801- 1975 clauses 6.4.2 and 6.4.3 To  
 calculate Fbw, divide 36560000 by the time  
 in seconds. = 365.60000 / (70.15) Fbw is  
 equal to 7429.3 kg/cm<sup>2</sup> For example, Fbw =  
 742.39 N/mm<sup>2</sup> and F= 0.6 3450=2072 kg/  
 cm<sup>2</sup> = 207 N/mm<sup>2</sup>. In this case, 2 fv/2Fv 2 /  
 1 = ( 126.37 199.59 ) 1 - 0.67 = 0.67 20  
 89.78



Wind force on the gable end side causes a bending moment on the end wall column, while the self-weight of the side sheeting, girt, etc. causes an axial compressive load. Consider a C/C spacing of 5m for the end wall columns. Dead Load (a) Assume that self-weight is equal to 0.16 KN/m owing to side sheeting and Girt. End wall Column Columns are loaded at each node (junction with a side girt) Side Sheets and Girt =0.8x3=2.4 KN Axial Compressive Load on End Wall Column of 0.16x5 =0.8KN. the number of Girts is 3, and Assume that the column's self-weight is 1.5 kg. The

maximum length of the end wall column is 5.84 metres.. 1.5+2.4=3.9KN Axial Compressive Load. 5x5.84x0.9x0.9=23.65KN is the wind load on the end wall column owing to the wind effect region. End wall column fastened on both ends, as an example. 17.26 kN/m<sup>2</sup> is the maximum value. Due to Wind Load, the Shear Force at the Ends (Supports) is 23.65/2 = 11.83KN.

**Choose Cross Section properties of C section**

Area	Unit/m	Ixx	Iyy	Zxx	Zyy	Rxx	Ryy
28.4Sq M	22.4 kg/m	1408cm <sup>2</sup>	432cm <sup>4</sup>	156.4cm <sup>3</sup>	54m <sup>3</sup>	7.04cm	3.9cm

Clause 6.6.3 of I. S. 801 1975 states: In this case, KL/Rx = (1 584) / 7.04 = 82.95 x 200 KL/YX = (1 150)/3.9 = 38.46 x 200 For Clause 6.6.1.1 of the IS: 801-1975 standard, "Fa1" is the maximum permissible average

compressive

stress.

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

$$= \sqrt{\frac{2\pi^2 \times 2 \times 10^6}{3450}}$$

$$= 107$$

Calculating of effective design width of compression element as per clause 5.2.1.1 of IS

801 – 1975

$$(w/t)_{Lim} = 1435/\sqrt{f} = 1435/\sqrt{20}$$

$$\text{Consider shera in compression } (w/t)_{Lim} = \frac{1435}{\sqrt{20}} = 321$$

OR

$$W = 321 \times 4$$

$$= 1284$$

$$Q = 1 \{ Q = \text{Effective Design Area or gross area} \}$$

$$\text{Max } \frac{kl}{r} = 82.95 < C_c = 107$$

As  $\frac{kl}{r} < C_c$ , According to clause no. 6.6.1.1 (b) IS 801 – 1975

$$\frac{\left[ 1 - \left( \frac{kl}{r} \right)^2 \right]}{2(C_c)^2} f_y$$

$$\frac{5}{3} + \frac{3}{8} \times \frac{(kl/r)}{C_c} - \frac{\left( \frac{kl}{r} \right)^3}{8(C_c)^3}$$

$$\frac{\left[ 1 - (82.95)^2 \right]}{2(107)^2} \times 3450$$

$$\frac{5}{3} + \frac{3}{8} \times \frac{82.95}{107} - \frac{(82.95)^3}{8(107)^3}$$

$$= 1276.39 \text{ kg/cm}^2$$

$$= 127.64 \text{ N/mm}^2$$

$$= \frac{3900}{28.4 \times 10^2} = 1.37 \text{ N/mm}^2$$

$$\text{Permissible axial comp stress} = \frac{3900}{28.4 \times 10^2} = 1.37 \text{ N/mm}^2$$

Calculation for permissible compressive Building stress As per clause 6.3 of IS 801 – 1975

$$F_b = 0.6 f_y$$

$$= 0.6 \times 3450$$

$$= 2070 \text{ kg/cm}^2$$

$$= 207 \text{ N/mm}^2$$

$$\text{Under wind load combination } F_b = 207 \times 1.33$$

$$= 275.31 \text{ N/mm}^2$$

$$(w/t)_{Lim} = 1435/\sqrt{f} \{ f = 0.75 \times 2000, = 1500 \text{ kg/cm}^2 \}$$

$$= 1435/\sqrt{1500}$$

$$= 37$$

Calculation for permissible compressive stress As per clause 6.3 of IS 801 – 1975

Referring to Clause no.6.3 (a) of IS: 801-1975. L = 1.5m is Unbraced Length

$$S_{xc} = Z_{xx} 156.4 \text{ cm}^3$$

$$I_{yc} = I_{yy}/2 = 432/2$$

$$= 216 \text{ cm}^4$$

$$\frac{I^2_{sxc}}{d I_{yc}} = \frac{150^2 \times 156.4}{18 \times 216}$$

$$= 905$$

$$\frac{0.36 \pi^2 \times E \times CB}{f_y} \quad (1)$$

$$\frac{J_y}{3450} \times \frac{0.36 \pi^2 \times 2 \times 10^6 \times 1}{3450}$$

$$= 2060 \quad \longrightarrow \quad (2)$$

$$\frac{1.8 \pi^2 \times E \times CB}{f_y}$$

$$\frac{1.8 \pi^2 \times 2 \times 10^6 \times 1}{3450}$$

$$= 10299 \quad \longrightarrow \quad (3)$$

$$(1) < (2)$$

$$(1) < (3)$$

$$(1) < (3)$$

$$\text{Hence } F_b = 275.31 \text{ N/mm}^2$$

$$F_b = 275.31 \text{ N/mm}^2$$

$$F_{b,act} = \frac{22.28 \times 10^6}{156.4 \times 10^3}$$

$$= 142.46 < 275.31 \text{ Mpa}$$

**Unity Check**

Referring to Clause number 6.7.2 of IS: 801-1975

$$\frac{f_a}{F_{a1}} = \frac{1.37}{169.76} = 0.0080 <$$

$$\frac{f_a}{F_{a1}} = \frac{142.46}{275.76} = 0.52$$

$$= \frac{f_a}{F_{a1}} + \frac{F_b}{F_{b1}} < 1$$

$$= \frac{f_a}{F_{a1}} + \frac{F_b}{F_{b1}} < 0.52 <$$

Hence safe

**Check for Deflection**

$$\delta_{allow} = \frac{\text{span}}{120} = \frac{5840}{120} = 48.67 \text{ mm}$$

Calculated Deflection due to Wind load on Column

$$\delta_{Cal} = \frac{5wt^3}{384EI}$$

$$= \frac{5 \times 23.65 \times 10^3 \times 5840^3}{384 \times 2 \times 10^5 \times 1408 \times 10^4}$$

$$= 21.78 \text{ mm}$$

$$= 21.78 < \delta_{allow} = 48.67 \text{ mm}$$

Hence the column is found safe when checked for deflection

**MAIN FRAME DESIGN LOAD CALCULATION**

Doomed to Failure The dead weight (roof sheeting and purlins) on the structure is calculated to be  $0.17 \times 6.25 = 1.0625 \text{ KN/m}^2$ . There are  $0.17 \times 1.5 \times 6.25 = 1.59 \text{ KN}$  of column loads The total weight of the column's girt and sheeting is  $1.59 \times 3 = 4.78 \text{ KN}$ . Load Bearing Capacity Because the service load on the rafter is  $0.1 \text{ kn/m}^2$ , U.D.L =  $0.1 \times 6.25 = 0.625 \text{ KN/m}$  for loads on the rafter As 10 0 0's Imposed Weight U.D.L. =  $0.75 + 6.25 = 4.6875 \text{ KN/m}^2$  for a live load of 0.75 To calculate wind load, multiply 9 by 6.25 and divide by 9.9 to get a value of 70.875 kilopascals

(KN).

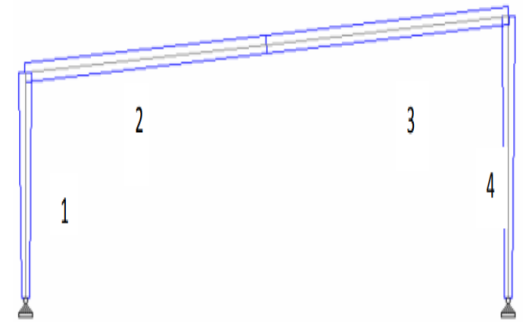


Table 3.1.3: properties of member of the frame

Member No	Depth at start node	Depth at end Node	Width of flange	Thickness of Flange	Thick ness of web
1	250mm	400mm	200mm	10mm	6mm
2	400mm	400mm	200mm	10mm	6mm
3	400mm	400mm	200mm	10mm	6mm
4	400mm	250mm	200mm	10mm	6mm

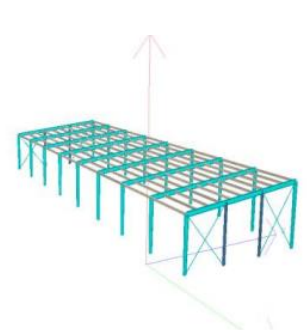


Figure 3.1.12: 3D view of CFS

**Check for Deflection**

$$\begin{aligned} \text{Max}^m \text{ permissible Def}^n \text{ due to wind load only for frame} &= h/60 \text{ to } h/100 \text{ (horizontal)} \\ &= \frac{5000}{60} \text{ to } \frac{5000}{100} \\ &= 83.33\text{mm to } 50 \text{ mm} \end{aligned}$$

Maximum permissible Vertical Deflection due to live load on frame

$$= \frac{\text{span}}{240} = \frac{5000}{240} = 62.5\text{mm}$$

The frame was checked for horizontal at node 2 & 4 for wind load & was found to be checked for vertical deflection at node 3 & was found safe.

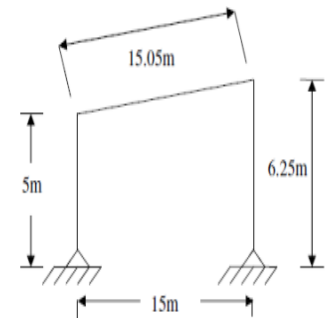
**The Weight of Cold Formed Industrial Building**

The total weight of Cold Formed Steel Industrial Building having area 15x50 m. & eave to be 15.92 Ton & cost of building is estimated 11.14 Lakh. The cost of cold formed steel

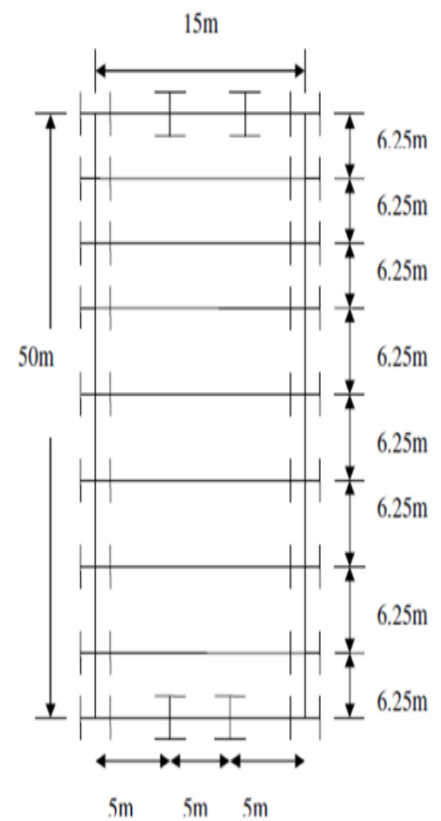
**3.2 ANALYSIS & DESIGNS OF HOT ROLL STEEL INDUSTRIAL BUILDING U**

**3.2.1 PARAMETERS OF HOT ROLLED INDUSTRIAL BUILDING**

Location	: Nagpur
Utility	: Cement Godown
Building Width	: 15m
Building Length	: 50 m
Eave Height	: 5m
C/C of Main frames	: 6.25 m
Maximum spacing of purlin	: 1.5 m
Slope of Roof	: 1:12
Structural material yield stress	: 250Mpa



**Figure 3.2.1: Elevation of HRS**



**Figure 3.2.2: Plan of HRS**

Figure 3.2.3: 3D View of HRS

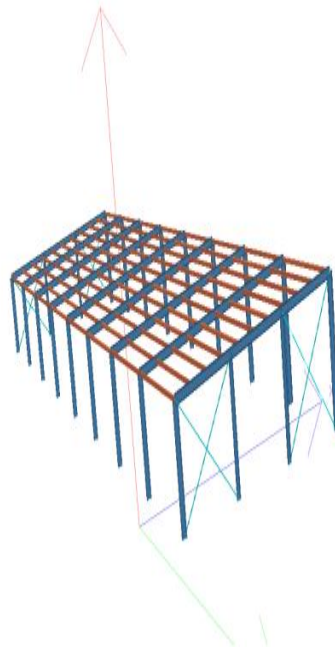


Table 3.2.1 Result of Conventional Industrial Building

Sr. No.	Member description	Section	Total wt (TN)
1	Rafter	ISMB-400	7.7
2	Purlins	ISJC-175	10.30
3	Main columns	ISMB-400	3.8
	Main columns	ISMB-350	1.59
4	Gable end wind column	ISMB-350	0.71
5	Tie runners	ISMB-250	0.821
Total weight			25.159

The total weight of Conventional Industrial Building having area of 15x50 m & eave height 5m was found to be 25.159 Ton & cost of building is estimated 15.09 Lakh. Cost of Hot Rolled steel is 60 Rs/Kg.

Chart plotted for the weight of Cold Formed Steel Industrial Building & Conventional Building by cost

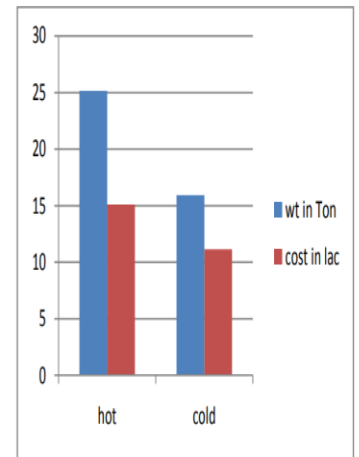


Figure 3.2.4: chart plotted between cost and weight of CFS and HRS

## RESULT AND CONCLUSION

### RESULT

Case 1: It was found that employing cold formed steel instead of hot rolled steel saved 9.239 T of material and saved 5.54 lakh in construction costs via section analysis and design. The distance between the c/c main frame is 7.14 m. It was found that by utilising cold formed steel instead of hot rolled steel, the material saved was 13.92 T and the cost was lowered to 8.35 lakh by using cold formed steel. The primary c/c frame is 6.67 metres apart.

### CONCLUSION

Cold-formed steel is more cost-effective in industrial construction than traditional steel, which is both more

expensive and more time-consuming. About a quarter of the materials and costs are saved. FUTURE PERSPECTIVE Cold-formed steel is used in the design of a multi-story structure to account for a variety of sectional characteristics. Different sections may be taken into account while designing various pieces, such as the eave strut, bracing system, sag rod, and foundation.

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