Gable Frame Structure Planning Using LRFD Method In Pamekasan Factor Warehouse Project

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ABSTRACT

Currently, the use of steel as a building construction has been widely used as the main material for building structures. Steel frames come in a variety of profiles and sizes. The use of steel frames can be adjusted to the type of construction to be built. From the results of the planning of the WF steel roof structure on the factory warehouse construction project in Pamekasan, it was obtained planning data: Gording using Profile C 125x50x40x4.5. Trekstang uses 8 mm diameter, Wind ties use 10mm diameter steel, Rafter uses WF 350x350x19x19 profile, column uses WF 350x350x19x19 profile, 8 pieces A325 bolts with 22 mm diameter, Hoist Crane Beam uses IWF Bulit-Up beam with 500x500x18x22 profile, Base Plate uses a size of 500x500x8mm with a column of 600x600. Calculation using LRFD is very important to get a structure that is stable, strong enough, serviceable, durable, and economical. A structure is said to be stable if it is not easily overturned, tilted, or displaced during the design of the building.

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1. Introduction

Currently, the use of steel as a building construction framework has been widely used as the main material for building structures[1][2]. Considering the use of wood, which is considered expensive compared to steel, it becomes the best alternative to building frames[3][4].

The advantage of using steel frames for building construction is that it is easy to install, practical, strong and durable. Besides steel will not be affected by extreme weather changes. Steel can be designed to be not easy to rust, mushroom[5][6]. In addition, it can be made of steel that is waterproof, termite-resistant, not easily porous and has great yield strength[7][8]. Steel frames are not only used for roof construction, but can also be used to build large buildings, such as factories, warehouses, construction structures, etc.

Calculation using LRFD is very important to get a structure that is stable, strong enough, serviceable, durable, and economical. A structure is said to be stable if it is not easily overturned, tilted, or displaced during the design of the building.

Other mechanical properties of structural steel for planning purposes are determined as follows (SNI 1729-2015) [9] :

- Modulus of elasticity : $E = 2100000$ MPa
- Shear modulus : $G = 80.000$ MPa
- Poisson's ratio : $\mu = 0.3$
- Expansion coefficient : $\alpha = 12 \times 10^{-6} / ^{0}$ C

2. Research Method

2.1 Description and Technical Planning Engineering Data

1. The span length of the factory warehouse (distance between columns) is 30 m.
2. The length of the building is 80m, the height is 5m, the angle of inclination is $20^\circ$, the distance between the horses is 4m.
3. Crane load P 10 tons, wind load 40 kg/m2, roof type Galvalume (12 kg/m2)
4. Wind bonding (bracing) of the wall of the stiffening frame, the side walls are exposed.
5. Steel quality A36 ($f_y = 240$ Mpa), bolt connection type A325.
6. Design using an easel.

| Table 1. Load Combination Process |
|----------------------------------|
| Load Combination | Equally | Load Combination | Centered |
|                  | $x$    | $y$    | $x$    | $y$    |
| 1,4D             | 14,112 | 38,78  |         |        |
| 1,2D + 1,6L + 0,5 (Lr or S or R) | 63,936 | 183,59 | 1,6L   | 51,84  | 150,35 |
Trekstang Planning

Loading

Dead Load (D)

| Curtain weight | = 8.32 kg/m x 2 x 10 | = 166.4 kg |
| Roof covering | = 12 kg/m² x 1.54 m x 4 | = 73.92 kg |
| Connection weight | = 10% x 73.92 kg | = 7.39 kg |

**D total = 247.71 kg**

Loading Combination

D = 247.71 kg
L = 100 kg

Choose the biggest combination

Pu = 1,2D + 1,6L + 0,5 (Lr or S or R) = 457,252 kg
Pu = 457,252 kg.

Trekstang

Trekstang is used to reduce the deflection of the x-axis direction (roof slope)[10], so that the force acting is the load in the x-axis direction. Working style:

\[ P_{ux} = P_u \sin 30^\circ = 790,28 \text{ kg} \]

The biggest force is on the top of the handlebar (near the cam) of 790.28 kg. This style will be used in the planning of the Trekstang dimension[11].

Trekstang Design

quality of steel used A36/BJ-37

Minimum yield stress, \( f_y = 240 \text{ Mpa} \)

Minimum breaking stress, \( f_u = 370 \text{ Mpa} \)

Trekstang is used to withstand tensile loads, so the design of the handlebars uses tensile analysis[12][13].

➢ At yielding, the nominal resistance of the tension bar is:

\[ T_n = \varnothing A_{dy} \]

7902.8 = 0.9 \( A_s \) (240)
Then the required diameter of the tension rod is:
\[ A_g = 0.25 \pi d^2 \]
\[ d = 6.82 \text{ mm} \]

In the fracture condition, the nominal resistance of the tension member is:
\[ T_n = \sigma A_g f_u \]
\[ 7902.8 = 0.75 A_g (370) \]
\[ A_g = 28.47 \text{ mm}^2 \]

Then the required diameter of the tension rod is:
\[ A_g = 0.25 \pi d^2 \]
\[ d = 6.02 \text{ mm} \]

So the diameter of the handlebar used is 8 mm.

**Wind Ties Calculation**

The load is planned as a concentrated load at each wind bond joint as follows:

**Dead Load (P_D)**
\[ D = (12 \times 4 \times 1.54) + (8.32 + 10\% \times 8.32) \times 4) = 110.528 \text{ kg} \]

**Live Load (P_L)**
\[ = 100 \text{ kg} \]

**Load in the direction of the axis of the rod**
\[ P_D = \frac{110.528}{\sin(56.65)} \times 2 = 265.69 \text{ kg} \]
\[ P_L = \frac{100}{\sin(56.65)} \times 2 = 239.52 \text{ kg} \]

**Ultimate Total Load**
\[ P_u = 1.2 P_D + 1.6 P_L \]
\[ = 7020.6 \text{ N} \]

**Wind Tensile Prisoner**
The wind bond is assumed to be 10 mm so that the tensile resistance of the bar at yielding is:
\[ T_n = A_g f_y \]
\[ = \frac{1}{4} \times \pi \times 10^2 \times 240 = 18840 \text{ N} \]

The tensile resistance of the bar at fracture is:
\[ T_n = A_g f_u \]
\[ = \frac{1}{4} \times \pi \times 10^2 \times 370 = 29045 \text{ N} \]

Tensile resistance at yielding condition is less than tensile resistance under fracture condition, so \( T_n \) at yielding condition is more decisive [14][15].

**Diameter of Wind Ties Used**
\[ T_u < \sigma T_n \]
\[ 7020.6 \text{ N} < 16956 \text{ N} \]
Gable Frame Planning (Non Sway Assumption)

Dead Load (D)
- Roof covering weight 1,54 x 4 x 12 = 73,92 kg
- Block's own weight 156 x 15,43 = 1450,42 kg
- Curtain's own weight 8,32 x 80 = 665,6 kg
- Wind bond weight 0,0005 x 7850 = 0,045 kg
- Connection weight 10% x (2189,985) = 218,99 kg

\[ D \text{ total} = 2408,975 \text{ kg} \]

Live Load (L)

| Table 3. Loading Combination |
|-----------------------------|
| 1,4D                        | 3372,57 |
| 1,2D + 1,6L + 0,5 (Lr or S or R) | 3050,77 |
| 1,2D + 1,6 (Lr or S or R) + (L or 0,5W) | 2992,37 |
| 1,2D + 1,0W + L + 0,5 (Lr or S or R) | 2990,77 |
| 1,2D + 1,0E + L + 0,2S | 2990,77 |
| 0,9D + 1,0W | 2168,08 |
| 0,9D + 1,0E | 2168,08 |

A. Non Sway Field Image

Momen
B. Slide

C. Axial

**Calculation of the Non-Sway Assumption Rafter**

\[
\begin{align*}
M_1 &= 13166.49 \text{ kgm} \\
V_1 &= 7136.11 \text{ kg} \\
M_2 &= 172.04 \text{ kgm} \\
V_2 &= 5419.86 \text{ kg}
\end{align*}
\]

For the calculation of the rafter, the largest forces are used:

\[
\begin{align*}
Pu &= 7136.11 \text{ kg} \\
M &= 13166.49 \text{ kgm}
\end{align*}
\]

Tried using profile **WF 350x350x19x19**:

Stiffness:
GA = \frac{\sum l_{rafter}}{\sum l_{column}} = 0,324

GB = \frac{\sum l_{rafter}}{\sum l_{rafter}} = 1

**Table 4.** monogram effective length factor k for frame

Based on the picture, get the value of KK

KL = 10,34 = 1033,81 cm = 10338,1 mm

KL/r_y = 121,19 cm = 1211,9 mm

KL/r_x = 70,32 cm = 703,2 mm

Tried using profile **WF 350x350x19x19** with Ag = 198,4 cm² = 19840 mm²

Score P_n, according to **SNI 1729-2015**, chapter E.3 is:

\[ P_n = F_{cr} \cdot A_g \]

a. If \( \frac{KL}{r} \leq 4,71 \sqrt{\frac{E}{F_y}} \) or \( \frac{F_y}{F_e} \leq 2,25 \)

\[ F_{cr} = \left[ 0,658 \frac{F_y}{F_e} \right] F_y \]

1.a

b. If \( \frac{KL}{r} > 4,71 \sqrt{\frac{E}{F_y}} \) or \( \frac{F_y}{F_e} > 2,25 \)

\[ F_{cr} = 0,877 F_y \]

1.b

https://doi.org/10.30736/cvl.v2i2
Fe = \frac{\pi^2 E}{(KL)^2}

Equation 1.b is used because
\[ \frac{KL}{r} > 4.71 \frac{E}{F_y} \]
703.22 > 135.96
\[ F_e = \frac{\pi^2 200000}{(10338.1)^2} \sqrt{\frac{4944.90}{703.22}} = 398.77 \]
So \[ F_c = 0.877 \times 398.77 \]
So as, \[ P_n = F_c \times A_g = 349.72 \times 19840 = 6938444.8 \text{ N} \]

Check cross-sectional compactness
\[ \lambda = \frac{bf}{2tf} = 357/2 \times 19 = 9.39 \]
\[ \lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 10.97 \]
\[ \lambda_r = 1.0 \sqrt{\frac{E}{F_y}} = 26.86 \]
\[ \text{Check, } \lambda \leq \lambda_p \]
9.39 \leq 10.97 \rightarrow \text{compact} \]

Mn = Mp = f_y \cdot Z_x = 240 \cdot 2450000 = 588000000 \text{ Nmm} \]
Pu/OPn < 0.2
71361,1/5897678,08 < 0.2
0.012 < 0.2 ......OK 

Check Lateral Bend
\[ P_u = 7136,11 \text{ kg} = 7136,11 \text{ N} \]
\[ P_n = 6938444,8 \text{ N} = 693844,48 \text{ kg} \]
MX = 13166,49 kgm
MP = 588000000 Nmm = 58800 kgm
\[ M_{nx} = Z_x \cdot f_y \]
\[ = 5880000 \text{ kgcm} = 58800 \text{ kgm} \]
\[ M_{ny} = Z_y \cdot f_y \]
\[ = 1941600 \text{ kgcm} = 19416 \text{ kgm} \]
\[ \frac{P_u}{2 \cdot \phi \cdot P_n} + \left( \frac{M_{nx}}{\phi \cdot M_{nx}} + \frac{M_{ny}}{\phi \cdot M_{ny}} \right) \leq 1 \]
0,254 \leq 1 ...... OK !!
Calculation of Non Sway Assumption Column

\[ M_1 = 14805.09 \text{ kgm} \quad M_2 = 13166.49 \text{ kgm} \]
\[ V_1 = 6643.01 \text{ kg} \quad V_2 = 5724.84 \text{ kg} \]

For the calculation of the column the largest forces are used:
\[ P_u = 6643.01 \text{ kg} \]
\[ M = 14805.09 \text{ kgm} \]

Tried using profile WF 350x350x19x19:

Stiffness:
\[ G_A = \frac{\sum_i \frac{1}{l_{r_{i,eff}}}}{\sum_i \frac{1}{l_{c,i}}}, \quad G_B = 1 \]

Table 5. monogram effective length factor \( k \) for frame

Based on the picture, get the value of \( K \)
\[ K = 0.83 \]
\[ KL = 12.80 = 1280 \text{ cm} = 12800 \text{ mm} \]
\[ KL/r_y = 150.06 \text{ cm} = 1500.6 \text{ mm} \]
\[ KL/r_x = 87.07 \text{ cm} = 870.7 \text{ mm} \]

Tried using profile WF 350x350x19x19 with \( Ag = 198.4 \text{ cm}^2 = 19840 \text{ mm}^2 \)

Equation 1.b is used because

https://doi.org/10.30736/cvl.v2i2
\[
\frac{KL}{r} > 4.71 \sqrt{\frac{E}{F_y}}
\]

870.7 > 135.96

\[
F_e = \frac{\pi^2 W_{net}}{L^2} = \frac{1971920}{7581.18} = 260.89
\]

So \( F_e = 0.877 \times 260.89 = 228.80 \)

So as, \( P_n = F_e \times A_g = 228.80 \times 19840 = 4539392 \) N

Check cross-sectional compactness

\[\lambda = \frac{bf}{2tf} = 357/2 	imes 19 = 9.39\]

\[\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 10.97\]

\[\lambda_r = 1.0 \sqrt{\frac{E}{F_y}} = 26.86\]

Check,

\[\lambda \leq \lambda_p\]

9.39 \leq 10.97 \rightarrow \text{compact}

\[
M_n = M_p = f_y \cdot z_x = 220 \times 2450000 = 588000000 \text{ Nmm}
\]

Pu/OPn < 0.2

66430.1/385848.32 < 0.2

0.17 < 0.2 \ldots \ldots \text{OK}

Check Lateral Bend

\[P_u = 6643.01 \text{ kg}\]

\[P_n = 4539392 \text{ N} = 453939.2 \text{ kg}\]

\[M_{ux} = 14805.09 \text{ kglm}\]

\[M_p = 588000000 \text{ Nmm} = 58800 \text{ kglm}\]

\[M_{ud} = Z_x \cdot f_y = 5880000 \text{ kgcmm} = 58800 \text{ kglm}\]

\[M_{uy} = Z_y \cdot f_y = 1941600 \text{ kgcmm} = 19416 \text{ kglm}\]

\[
\frac{P_u}{2 \cdot M_p} + \left\{ \frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right\} \leq 1
\]

0.287 \leq 1 \ldots \ldots \text{OK} !!!

Connection Planning

Rafter Bolt and Column Planning For Profile 350x350x19x19

From the calculation results SAP2000v14 obtained:

\[M_a = 13166.49 \text{ kglm} = 1316649 \text{ kgcmm}\]
Bolts are used A325 with:
Threaded bolt Ø 7/8” = 22 mm = 2.2 cm

\[ A_b = \pi r^2 = \pi \left(\frac{22}{2}\right)^2 = 3,454 \text{ cm}^2 \]

Plate thickness (tp) = 12 mm

High quality bolt

\[ f_{ub} = 825 \text{ Mpa} = 8250 \text{ kg/cm}^2 \]
\[ f_{yb} = 585 \text{ Mpa} = 5850 \text{ kg/cm}^2 \]

**Rafter Bolt Plan**

Calculation of shear strength of one bolt

\[ \varnothing R_{nv} = 0.75 \times r_1 \times f_{ub} \times A_b \]
\[ = 10685.81 \text{ kg} \]

Calculation of the strength to support one for

\[ \varnothing R_n = 0.75 \times 2.4 \times d \times tp \times f_{ub} \]
\[ = 39204 \text{ kg} \]

Calculation of tensile strength of one bolt

\[ \varnothing R_{nt} = 0.75 \times 0.75 \times f_{ub} \times A_b \]
\[ = 16028.72 \text{ kg} \]

Assumed number of bolts

It is assumed that there are 8 bolts

Edge distance (S1) = 1.25 x db = 1.25 x 22 = 27.5 mm

Distance between bolts (S) = 3 x db = 3 x 22 = 66 mm

S1 = 100 mm

S = 100 mm

\[ R_{uv} = \frac{P_u}{n} = \frac{7136.11}{8} = 829.01 \text{ kg} \]

\[ \frac{829.01}{10685.81} + \frac{R_{ut}}{16028.72} \leq 1 \]

\[ R_{ut} = T = 14785.2 \text{ kg} \]

Connection moment

\[ a = \left(\frac{\sum T}{f_{y} \times b}\right) = \frac{14785.2}{5850 \times 35} = 0.43 \text{ cm} \]

\[ d_1 = (100/10) - 0.43 = 9.57 \text{ cm} \]
\[ d_2 = d_1 + 100/10 = 19.57 \text{ cm} \]
\[ d_3 = d_2 + 100/10 = 29.57 \text{ cm} \]
\[ \Sigma d = 58.71 \text{ cm} \]

\[ \varnothing M_n = \frac{0.9 \times f_{y} \times a \times b}{2} + \Sigma d \]
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From the calculation results SAP2000v14 obtained:

\[ M_u = 14805,09 \text{ kgm} = 1480509 \text{ kgcm} \]

\[ P_u = 6643,01 \text{ kg} \]

Used A325 bolts with:

Threaded bolt \( \phi 7/8'' = 22 \text{ mm} = 2,2 \text{ cm} \)

\[ A_b = \pi r^2 = \pi \left(\frac{22}{2}\right)^2 = 3,454 \text{ cm}^2 \]

Plate thickness (tp) = 12 mm

High quality bolt \( f_{ub} = 825 \text{ Mpa} = 8250 \text{ kg/cm}^2 \)
\( f_{yb} = 585 \text{ Mpa} = 5850 \text{ kg/cm}^2 \)

**Rafter Column Bolt Plan**

Calculation of shear strength of one bolt

\[ \Omega R_n = 0,75 \times r_1 \times f_{ub} \times A_b \]
\[ = 10685,81 \text{ kg} \]

Calculation of the bearing strength of one bolt

\[ \Omega R_n = 0,75 \times 2,4 \times d \times tp \times f_{ub} \]
\[ = 39204 \text{ kg} \]

Calculation of tensile strength of one bolt

\[ \Omega R_{nt} = 0,75 \times 0,75 \times f_{u} \times A_b \]
\[ = 16028,72 \text{ kg} \]

It is assumed that there are 8 bolts

Edge distance (S1) = 1,25 x db = 1,25 x 22 = 27,5 mm

Distance between bolts (S) = 3 x db = 3 x 22 = 66 mm

\[ S1 = 100 \text{ mm} \]
\[ S = 100 \text{ mm} \]

\[ R_{uv} = P_u/n = 6643,01 / 8 = 830,38 \text{ kg} \]

\[ \left( \frac{R_{uv}}{\Omega R_{nv}} \right)^2 + \left( \frac{R_{ut}}{\Omega R_{nt}} \right)^2 \leq 1 \]

\[ \frac{830,38}{10685,81} + \frac{R_{ut}}{16028,72} \leq 1 \]

\[ R_{ut} = T = 14783,15 \text{ kg} \]

\[ a = \left( \frac{\Sigma T}{f_y B} \right) = \frac{14783,15 \times 6}{5850 \times 35} = 0,43 \text{ cm} \]

\[ d_1 = (100/10) - 0,43 = 9,57 \text{ cm} \]

\[ d_2 = d_1 + 100/10 = 19,57 \text{ cm} \]
\[ \Sigma d = 58,71 \text{ cm} \]

\[ \Phi M_n = \frac{0.9 \times f_y \times a^2 \times b}{2} + \Sigma d \Gamma \]
\[ = \frac{0.9 \times 5850 \times 0.43^2 \times 35}{2} + 58,71 \times 14783,15 \times 2 \]
\[ = 17036,22 + 1735837,473 \]
\[ = 1752873,693 \text{ kgcm} > 1316649 \text{ kgcm} \ldots \text{ OK !!!} \]

**Crane Structure Planning**

**Structural Geometry**

![Figure 1. Bridge Beam Design](image)

The length of the bridge beam that will be reviewed is 30 meters and 80 meters.

**Material Data**

The steel material used in the design of this steel structure is Hot-Rolled steel (WF, C, T profiles and steel plates) with the following data:

- Melting strength : \( F_y = 240 \text{ MPa} \)
- Melting strength : \( E = 200000 \text{ MPa} \)
- Shear Modulus : \( G = 80000 \text{ MPa} \)

**Loading**

The loads that are reviewed as design loads in the calculation of this factory warehouse structure are:

Dead Load which includes the self-weight of the steel profile with a steel density \( (\gamma_{\text{steel}}) \) of 7850 kg/m³ as listed in Table 3-1 SNI 03-1727-1989-F and Live Load which includes crane loads \( (P_{\text{crane}}) \) of 10 tons = 10000 kg. [16][17][18]
Hoist Crane Beam Planning

In this plan, 2 girders are used *bridge beam*

Built-up IWF profile data:

B = 600 mm
H = 1144 mm
H_w = 1100 mm
t_w = 18 mm
t_f = 22 mm

- The cross-sectional property data are as follows:

\[ A = (2.t_f.B) + (t_w.H_w) \]
\[ q = A. \gamma_{baja} \]
\[ I_x = 2 \left[ \left( \frac{1}{12} \cdot B \cdot t_f^3 \right) + \left( t_f \cdot B \cdot \left( \frac{H-t_f}{2} \right)^2 \right) + \left( \frac{1}{12} \cdot t_w \cdot H_w \right)^3 \right] = 6.15 \cdot 10^9 \text{ mm}^4 \]
\[ I_y = \frac{1}{12} \left[ \left( 2 \cdot t_f \cdot B^3 \right) + (H \cdot t_w^3) \right] \]
\[ S_x = \frac{I_y}{0.5 \cdot H} = 1.07 \cdot 10^7 \text{ mm}^3 \]
\[ S_y = \frac{I_y}{0.5 \cdot H} = 2.64 \cdot 10^6 \text{ mm}^3 \]
\[ Z_x = B \cdot t_f \cdot (H - t_f) \] + \left( \frac{1}{4} \cdot t_w \cdot H_w^2 \right) \]
\[ Z_y = 1.5 \cdot S_y = 3.96 \cdot 10^6 \text{ mm}^3 \]
\[ J = (t_w^3 \cdot H_w) + (2 \cdot t_f^3 \cdot B) \]
\[ J = 1.19 \cdot 10^7 \text{ mm}^4 \]

- Calculation of the forces in the ultimate on the hoist-crane beam:

\[ P/L \]
\[ \text{Hoist-crane span length : } L = 30 \text{ m} = 30000 \text{ mm} \]
\[ \text{Crane load on 1 beam : } P = 5 \text{ Ton} = 5000 \text{ kg} \]
\[ \text{Shock factor (impact factor)} = 1.25 \]

Figure 2. Hoist Crane

Figure 3. Hoist Crane
Crane load design: P = 5 Ton \cdot 1.25 = 6.25 Ton

Ultimate moment:

\[ M_{DL} = 1.2 \cdot \frac{1}{8} \cdot q \cdot L^2 = 48960.45 \text{ kNm} \]

\[ M_{LL} = 1.6 \cdot \frac{1}{4} \cdot P \cdot L = 75000 \text{ kNm} \]

\[ M_u = M_{DL} + M_{LL} = 123960.45 \text{ kNm} = 1239.6045 \text{ kNm} \]

Maximum shear force due to crane load:

\[ V_{DL} = 1.2 \left( q_u \cdot L \right) = 13056.12 \text{ kg} \]

\[ V_{LL} = 1.6 \cdot \frac{P}{2} = 10000 \text{ kg} \]

\[ V_u = V_{DL} + V_{LL} = 23056.12 \text{ kg} \]

- Check the Slimness of the Cross

1. Wing cross section

Check cross section of the wing

\[ \lambda = \frac{bf}{2tf} = 27.2727 \]

\[ \lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 10.97 \]

\[ \lambda_r = 1.0 \sqrt{\frac{E}{F_y}} = 26.86 \]

Check,

\[ \lambda \leq \lambda_p \]

27,2727 \leq 10.97 \quad \rightarrow \text{Not compact} \]

2. Body Cross

\[ \lambda = \frac{bf}{2tf} = 61.11 \]

\[ \lambda_p = 3.76 \sqrt{\frac{E}{F_y}} = 108.54 \]

\[ \lambda_r = 5.70 \sqrt{\frac{E}{F_y}} = 164.54 \]

Check,

\[ \lambda \leq \lambda_p \]
Lateral Bending Parameter

Finding Value $L_p$

$$r_y = \sqrt{\frac{J}{A}} = \sqrt{\frac{793000000}{46200}} = 130.975 \text{ mm}$$

$$L_p = 1.76 \cdot r_y \cdot \frac{E}{F_y}$$

$$= 1.76 \cdot 130.975 \cdot \sqrt{\frac{200000}{240}}$$

$$= 6654.419 \text{ mm}$$

Finding Value $L_r$

$$H_o = H - t_f = 1144 - 22 = 1122 \text{ mm}$$

$$C_w = \frac{1}{4} \cdot I_y \cdot H_o^2$$

$$= 2.49 \cdot 10^{14} \text{ mm}^6$$

$$r_s = \sqrt{\frac{f_y \cdot C_w}{S_x}} = \sqrt{\frac{7.93 \cdot 10^3 \times 2.49 \cdot 10^{14}}{1.07 \cdot 10^7}} = 203.322$$

C for double symmetrical I profile

$$L_x = 1.95 r_s \sqrt{\frac{E}{f_y}} \left[ \frac{1.4 C}{S_x h_o} + \frac{f_c}{S_x h_o} + 6.76 \left( \frac{0.7 f_y}{E} \right) \right] = 30923.114 \text{ mm}$$

Finding the nominal moment value

1. Melting Condition

$$M_n = M_p = Z_x \cdot f_y = 4861.296 \text{ kNm}$$

2. Nominal strength of members against bending moment

Because the structural components meet the requirements $L_p \leq L \leq L_r$, then the nominal strength of the structural member against the bending moment is as follows:

$$C_b = \frac{12.5 \cdot M_{max}}{2.5 \cdot M_{max} + 3 \cdot M_A + 4 \cdot M_B + 3 \cdot M_C} = 0.865595$$

$$M_n = C_b \cdot M_p - 0.7 \cdot f_y \cdot Z_x \cdot \left( \frac{L_f - L_p}{r_f - L_p} \right) = 1511.292 \text{ kNm}$$

With the provision of: $M_n \leq M_p$

3. Local bending of the wing plate

Due to the flange plate is a Non-compact section then:

$$M_n = C_b \cdot M_p - 0.7 \cdot f_y \cdot Z_x \cdot \left( \frac{L_f - L_p}{r_f - L_p} \right) = 1645.847 \text{ kNm}$$

4. Local bending of the body plate

Since the web plate is of compact cross-section, local buckling in the flange plate does not occur[19][20].
The nominal moment value is taken from the smallest nominal moment value, that is 1511,292 kNm.

- **Control of bending**
  \[ \Omega M_n > Mu \]
  
  \[ 1360,163 \text{ kNm} > 1239,6045 \text{ kNm} \]

  The value of \( \Omega M_n \) is greater than \( Mu \) then it fulfills the condition.

- **Sliding control**
  \[ V_n = 0.6 \cdot f_y \cdot A_w \cdot C_v = 285120 \text{ kg} \]
  
  \[ \Omega V_n > V_u \]
  
  \[ 256608 \text{ kg} > 23056,12 \text{ kg} \]

  The value of \( \Omega V_n \) is greater than \( V_u \) then it fulfills the condition.

- **Control against deflection**
  \[ \Delta \text{ permission} > \Delta \text{ happen} \]
  
  \[ \frac{L}{500} > \frac{5q.L^4}{384.E.I_x} + \frac{P.L^3}{48.E.I_x} \]
  
  \[ 60 \text{ mm} > 59,66 \text{ mm} \]

  The value of \( \Delta \) permission is greater than occurs then it fulfills the conditions.

The weight of the bridge beam in this design is 21760.2 kg.

### Base Plate Planning

#### Technical Data

Case of column with vertical load and moment (large eccentricity)

\[ Pu = 6643,01 \text{ kg} = 66430,1 \text{ N} \]

\[ Mu = 14805,09 \text{ kgm} \]

Tried using profile **WF 350x350x19x19** :

Tried \( B = 500 \text{ mm}, \text{ dan } N = 500 \text{ mm} \)

\[ A_1 = 250000 \text{ mm}^2 \]

\[ e = \frac{Mu}{Pu} = \frac{14805,09}{6643,01} = 2,228672 \text{ m} = 2228,67 \text{ mm} > \frac{N}{2} = \frac{500}{2} = 250 \text{ mm} \]

\[ f_{1,2} = \frac{P}{B 	imes N} \pm \frac{M_x \cdot c}{I} = \frac{6643,01}{500 \times 500} \pm \frac{14805090 \times 250}{5208333333} = -0,2657 \pm 0,71064 \]

\[ f_1 = -0,4449 \]

\[ f_2 = 0,9764 \]

Try column with size 600 x 600 mm → \( A_2 = 360000 \text{ mm}^2 \)

\[ \frac{A_2}{A_1} = \frac{360000}{250000} = 1,44 \text{ mm} \]

\[ f_p = 0,85 \times \varphi_c \times f_c \times \frac{A_2}{\sqrt{A_1}} \]

\[ = 0,85 \times 0,6 \times 20 \times 1,2 \]

\[ = 12,24 \text{ MPa} \]
\[ f_2 = 12.8 < F_p \quad \text{OK!} \]

\[ f_3 = f_2 - f_3 \]

**Figure 4. Column With Load**

**Determining the Plate Thickness**

\[
m = \frac{N - 0.95d}{2} = \frac{350 - (0.95 \times 350)}{2} = 83.75 \text{ mm}
\]

\[
n = \frac{B - 0.8f}{2} = \frac{350 - (0.8 \times 357)}{2} = 107.2 \text{ mm}
\]

**Neutral Line**

\[
A = \frac{f_2}{f_2 + f_1} \times N = \frac{0,97636472}{0,97636472 + 0,44} \times 500 = 343.70
\]

\[
f_3 = \frac{A - m}{A} \times f_2 = \frac{343.70 - 83.75}{343.70} \times 0,97636472 = 0.738454
\]

\[
f_4 = f_2 - f_3 = 0.97636472 - 0.738454 = 0.237911
\]

\[
M_{plu} = \left( \frac{1}{2} \times f_3 + \frac{1}{3} \times f_4 \right) m^2 \times B
\]

\[
= \left( \frac{1}{2} \times 0.738454 + \frac{1}{3} \times 0.237911 \right) 83.75^2 \times 500
\]

\[
= 1503480 \text{ mm}
\]

**Plate moment capacity**

\[
M_n = \varphi_y Z f_y = \varphi_y \left( \frac{1}{4} B t_p^2 \right) \times f_y = 0.9 \left( \frac{1}{4} B t_p^2 \right) \times f_y = M_{plu}
\]

\[
t_p = \sqrt{\frac{4 M_{plu}}{0.9 B f_y}} = \sqrt{\frac{4 \times 1503480}{0.9 \times 500 \times 240}} = 7.46 \rightarrow 8 \text{ mm}
\]

**Conclusion**

Size Plate 500 x 500 x 8 mm

Column Size 600 x 600 mm

### 4. Conclusion and Suggestion

#### 4.1 Conclusion
The overall results of the calculations that have been carried out can be drawn as follows:

From the results of the planning of the WF steel roof structure on the factory warehouse construction project in Pamekasan, it was obtained planning data: Gording using Profile C 125x50x40x4.5. Trekstang uses 8 mm diameter, Wind ties use 10mm diameter steel, Rafter uses WF 350x350x19x19 profile, column uses WF 350x350x19x19 profile, 8 pieces A325 bolts with 22 mm diameter, Hoist Crane Beam uses IWF Bulit-Up beam with 600x1144x18x22 profile, Base Plate uses a size of 500x500x8mm with a column of 600x600.

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