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

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A B S T R A C T

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 600x1144x18x22 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 : μ = 0,3
- Expansion coefficient : á = 12 x 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^{0} , 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 (fy = 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

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1,2D + 1,6 (Lr or S or R) + (L or 0.5W)	44,496	127,21	L		32,4	93,97
1,2D + 1,0W + L + 0,5 (Lr or S or R)	52,096	33,24	L		32,4	93,97
1,2D + 1,0E + L + 0,2S	44,496	127,21	L		32,4	93,97
0,9D + 1,0W	49,072	24,93				
0,9D + 1,0E	9,072	24,93				

Source : Analysis Results (2021)

Trekstang Planning

Loading

Dead Load (D)

Curtain weight	=	8.32 kg/m x 2 x 10	= 166.4 kg
Roof covering	=	12 kg/m^2 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 sin30º = 790,28 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 Trestang dimension[11].

Trekstang Design

quality of steel used A36/BJ-37

Minimum yield stress, fy = 240 Mpa

Minimum breaking stress, fu = 370 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:

$T_n = Ø A_g f_y$

$7902{,}8 = 0{,}9\, A_g\,(240)$

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A_g = 36,58 mm^2

Then the required diameter of the tension rod is:

A_g = 0,25 π d^2

d = 6,82 mm

- In the fracture condition, the nominal resistance of the tension member is:

$T_n = \text{Ø}\, A_g f_u$

7902,8 = 0,75 A_g (370)

A_g = 28,47 mm^2

Then the required diameter of the tension rod is:

A_g = 0,25 π d^2

d = 6,02 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 x 4 x 1.54) + (8,32 + 10% x 8.32) x 4) = 110.528 kg

Live Load (P_L) = 100 kg

Load in the direction of the axis of the rod

$P_D = \frac{110.528}{\sin(56.65)} \times 2 = 265.69$ kg

$P_L = \frac{100}{\sin(56.65)} \times 2 = 239.52$ kg

Ultimate Total Load

$P_u = 1.2\, P_D + 1.6\, P_L$

$= 7020.6$ N

Wind Tensile Prisoner

The wind bond is assumed to be 10 mm so that the tensile resistance of the bar at yielding is:

$T_n = A_g\, f_y$

$= ¼ \times \pi \times 10^2 \times 240 = 18840$ N

The tensile resistance of the bar at fracture is:

$T_n = A_g\, f_u$

$= ¼ \times \pi \times 10^2 \times 370 = 29045$ N

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

Diameter of Wind Ties Used

$T_u < \text{Ø}\, T_n$

7020,6 N < 16956 N

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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 total = 2408,975 kg

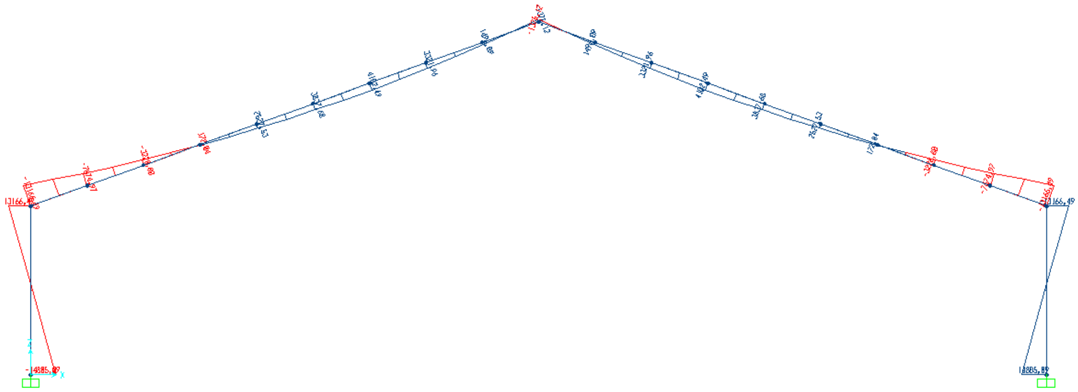
Live Load (L)

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



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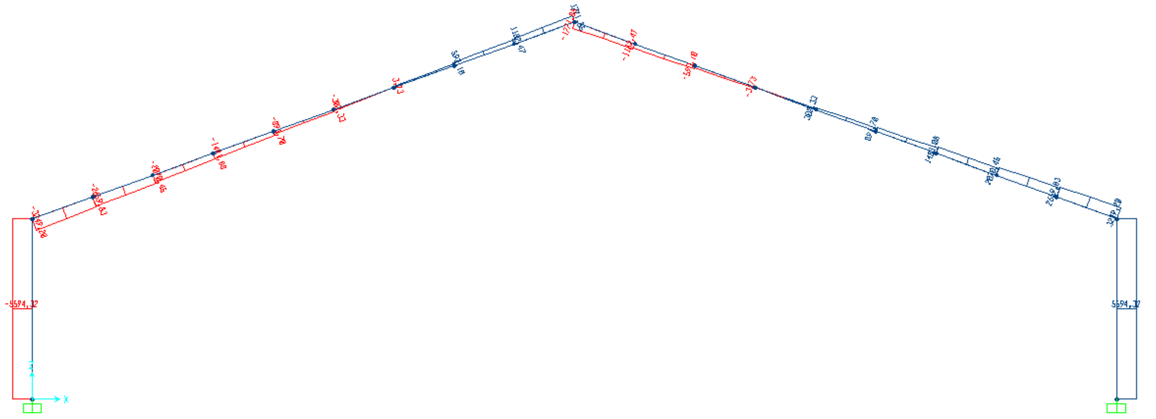
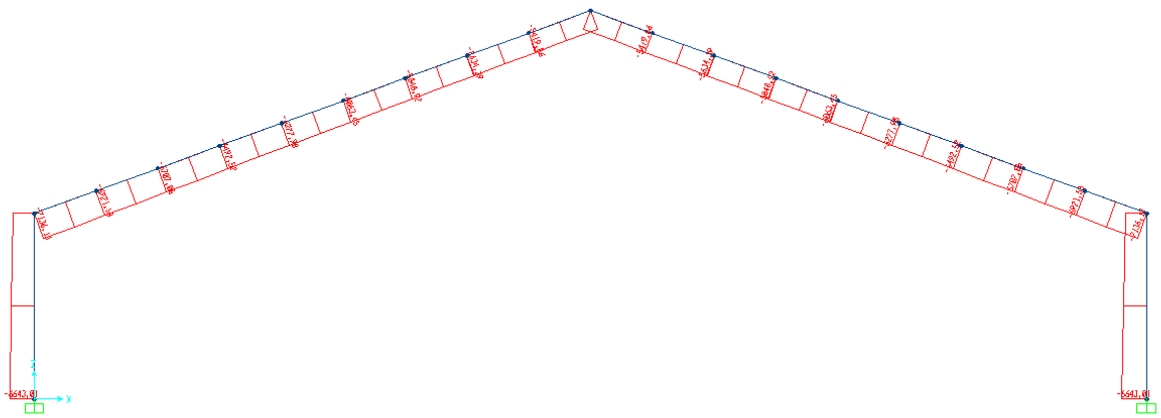
B. Slide



C. Axial



Calculation of the Non-Sway Assumption Rafter

M1 = 13166,49 kgm M2 = 172,04 kgm

V1 = 7136,11 kg V2 = 5419,86 kg

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

Pu = 7136,11 kg

M = 13166,49 kgm

Tried using profile **WF 350x350x19x19** :

Stiffness:

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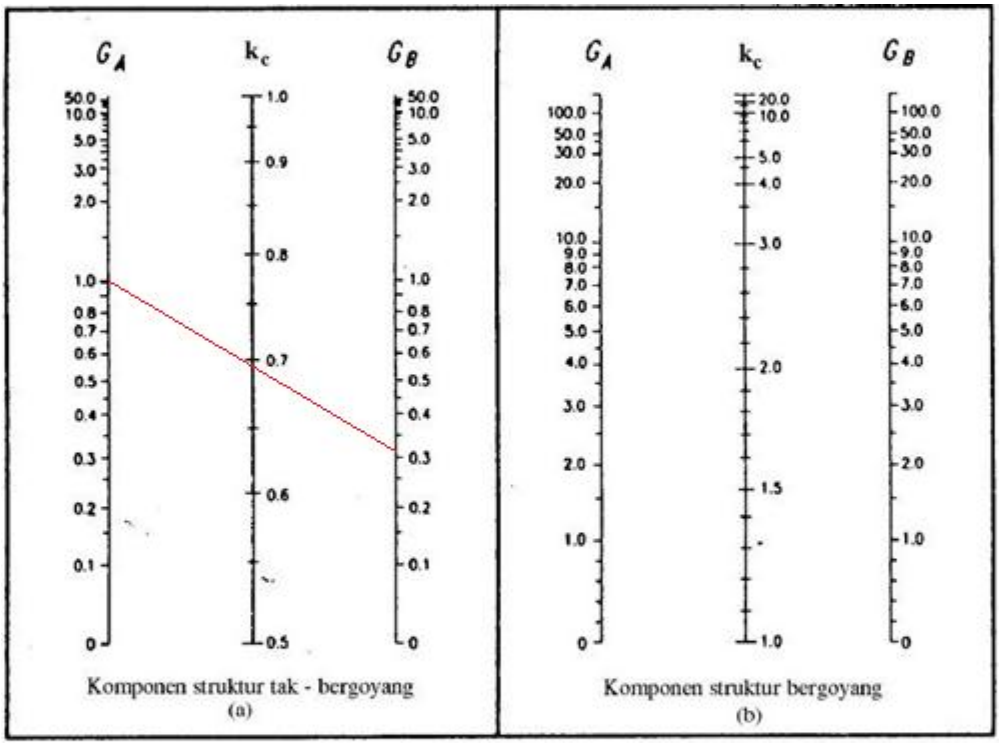
GA $= \dfrac{\sum \frac{I}{LRafter}}{\sum \frac{I}{LColumn}} = 0{,}324$

GB $= \dfrac{\sum \frac{I}{LRafter}}{\sum \frac{I}{LRafter}} = 1$

Table 4. monogram effective length factor k for frame



Based on the picture, get the value of KK = 0,67

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 cm^2 = 19840 mm^2

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

$P_n = F_{cr} . A_g$

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

$F_{cr} = \left[0{,}658^{\frac{Fy}{Fe}}\right] F_y$ 1.a

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

$F_{cr} = 0{,}877 F_e$ 1.b

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$$Fe = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Equation 1.b is used because

$$\frac{KL}{r} > 4{,}71\sqrt{\frac{E}{Fy}}$$

703,22 > 135,96

$$F_e = \frac{\pi^2\ 200000}{\left(\frac{10338{,}1}{147}\right)^2} = \frac{1971920}{4944{,}90} = 398{,}77$$

So F_{cr} = 0,877 x 398,77

= 349,72 MPa

So as, P_n= F_{cr} x A_g = 349,72 . 19840 = 6938444,8 N

Check cross-sectional compactness

λ = bf/2tf = 357/2x19 = 9,39

λ_p = $0{,}38\sqrt{\frac{E}{Fy}}$ = 10,97

λ_r = $1{,}0\sqrt{\frac{E}{Fy}}$ = 26,86

Check,

$\lambda \leq \lambda_p$

9,39 ≤ 10,97 → **compact**

Mn = Mp = f_y . z_x = 240 . 2450000 = 588000000 Nmm

Pu/ØPn < 0,2

71361,1/5897678,08 < 0,2

0,012 < 0,2 **.......OK**

Check Lateral Bend

P_u = 7136,11 kg = 71361,1 N

P_n = 6938444,8 N = 693844,48 kg

M_{ux} = 13166,49 kgm

M_p = 588000000 Nmm = 58800 kgm

M_{nx} = Z_x . f_y

= 5880000 kgcm = 58800 kgm

M_{ny} = Z_y . f_y

= 1941600 kgcm = 19416 kgm

$$\frac{P_u}{2.\emptyset.P_n} + \left\{\frac{M_{ux}}{\emptyset.\ M_{nx}} + \frac{M_{uy}}{\emptyset.\ M_{ny}}\right\} \leq 1$$

0,254 ≤ 1 **...... OK !!!**

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Calculation of Non Sway Assumption Column

M1 = 14805,09 kgm M2 = 13166,49 kgm

V1 = 6643,01 kg V2 = 5724,84 kg

For the calculation of the column the largest forces are used:

Pu = 6643,01 kg

M = 14805,09 kgm

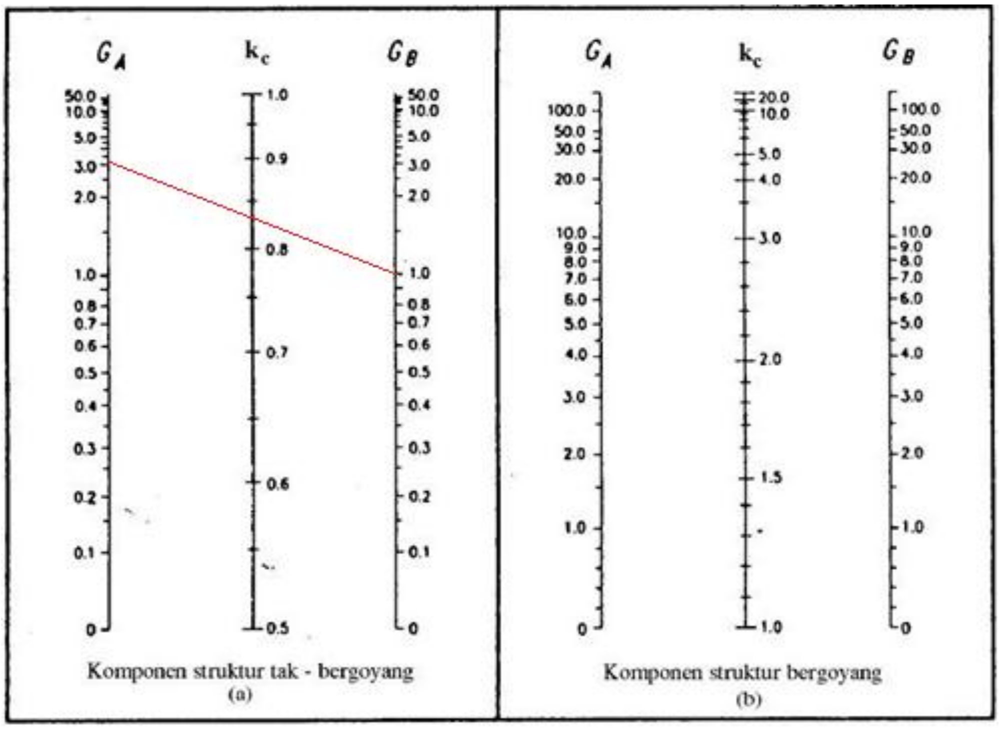
Tried using profile **WF 350x350x19x19** :

Stiffness:

$$GA = \frac{\sum \frac{I}{LRafter}}{\sum \frac{I}{LColumn}} = 3{,}086$$

$$GB = \frac{\sum \frac{I}{LRafter}}{\sum \frac{I}{LRafter}} = 1$$

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 cm = 12800 mm

KL/r_y = 150,06 cm = 1500,6 mm

KL/r_x = 87,07 cm = 870,7 mm

Tried using profile **WF 350x350x19x19** with Ag = 198,4 cm^2 = 19840 mm^2

Equation 1.b is used because

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$\frac{KL}{r} > 4{,}71\sqrt{\frac{E}{Fy}}$

870,7 > 135,96

$F_e = \frac{\pi^2\ 200000}{\left(\frac{12800}{147}\right)^2} = \frac{1971920}{7581{,}18} = 260{,}89$

So F_{cr} = 0,877 x 260,89

= 228,80

So as, $P_n = F_{cr}$ x A_g = 228,80 . 19840 = 4539392 N

Check cross-sectional compactness

λ = bf/2tf = 357/2x19 = 9,39

$\lambda_p = 0{,}38\sqrt{\frac{E}{Fy}}$ = 10,97

$\lambda_r = 1{,}0\sqrt{\frac{E}{Fy}}$ = 26,86

Check,

$\lambda \leq \lambda_p$

9,39 ≤ 10,97 → compact

Mn = Mp = f_y . z_x = 240 . 2450000 = 588000000 Nmm

Pu/ØPn < 0,2

66430,1/385848,32 < 0,2

0,17 < 0,2 **.......OK**

Check Lateral Bend

P_u = 6643,01 kg

P_n = 4539392 N = 453939,2 kg

Mu_x = 14805,09 kgm

M_p = 588000000 Nmm = 58800 kgm

M_{nx} = Z_x . f_y

= 5880000 kgcm = 58800 kgm

M_{ny} = Z_y . f_y

= 1941600 kgcm = 19416 kgm

$$\frac{P_u}{2\,.\emptyset\,.P_n} + \left\{\frac{M_{ux}}{\emptyset\,.\ M_{nx}} + \frac{M_{uy}}{\emptyset\,.\ M_{ny}}\right\} \leq 1$$

0,287 ≤ 1 **...... OK !!!**

Connection Planning

Rafter Bolt and Column Planning For Profile 350x350x19x19

From the calculation results SAP2000v14 obtained :

M_u = 13166,49 kgm = 1316649 kgcm

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Pu = 7136,11 kg

Bolts are used A325 with :

Threaded bolt Ø 7/8" = 22 mm = 2,2 cm

$A_b = \pi r^2 = \boldsymbol{\pi}\left(\frac{2{,}2}{2}\right)^2 = 3{,}454\ cm^2$

Plate thickness (tp) = 12 mm

High quality bolt f_{ub} 825 Mpa = 8250 kg/cm^2

f_{yb} 585 Mpa = 5850 kg/cm^2

Rafter Bolt Plan

Calculation of shear strength of one bolt

$ØRnv = 0{,}75 \times r_1 \times f_{ub} \times A_b$

$= 10685{,}81$ kg

Calculation of the strength to support one for

$ØRn = 0{,}75 \times 2{,}4 \times d \times tp \times f_{ub}$

$= 39204$ kg

Calculation of tensile strength of one bolt

ØRnt = 0,75 x 0,75 x fu x Ab

= 16028,72 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

Ruv = Pu/n = 7136,11 / 8 = 829,01 kg

$$\left(\frac{R_{uv}}{ØR_{nv}}\right)^2 + \left(\frac{R_{ut}}{R_{nt}}\right)^2 \leq 1$$

$$\frac{829{,}01}{10685{,}81} + \frac{R_{ut}}{16028{,}72} \leq 1$$

$R_{ut} = T = 14785{,}2$ kg

- Connection moment

$$a = \left(\frac{\sum T}{fy\ x\ B}\right) = \frac{14785{,}2\ x\ 6}{5850\ x\ 35} = 0{,}43\ cm$$

d1 = (100/10) – 0,43 = 9,57 cm

d2 = d1 + 100/10 = 19,57 cm

d3 = d2 + 100/10 = 29,57 cm

Σd = 58,71 cm

$$ØMn = \frac{0{,}9\ x\ fy\ x\ a^2\ xB}{2} + \Sigma dT$$

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$= \frac{0{,}9\ x\ 5850\ x\ 0{,}43^2\ x\ 35}{2} + 58{,}71 \text{ x } 14785{,}2 \text{ x } 2$

= 17036,22 + 1736078,184

= 1753114,404 kgcm > 1316649 kgcm **...... OK !!!**

From the calculation results SAP2000v14 obtained :

M_u = 14805,09 kgm = 1480509 kgcm

Pu = 6643,01 kg

Used A325 bolts with:

Threaded bolt Ø 7/8'' = 22 mm = 2,2 cm

A_b $= \pi r^2 = \boldsymbol{\pi} \left(\frac{2{,}2}{2}\right)^2 = 3{,}454 \text{ cm}^2$

Plate thickness (tp) = 12 mm

High quality bolt f_{ub} 825 Mpa = 8250 kg/cm^2

f_{yb} 585 Mpa = 5850 kg/cm^2

Rafter Column Bolt Plan

Calculation of shear strength of one bolt

ØRnv = 0,75 x r_1 x f_{ub} x A_b

= 10685,81 kg

Calculation of the bearing strength of one bolt

ØRn = 0,75 x 2,4 x d x tp x f_{ub}

= 39204 kg

Calculation of tensile strength of one bolt

ØRnt = 0,75 x 0,75 x fu x Ab

= 16028,72 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 mm

S = 100 mm

R_{uv} = Pu/n = 6643,01 / 8 = 830,38 kg

$$\left(\frac{R_{uv}}{ØR_{nv}}\right)^2 + \left(\frac{R_{ut}}{R_{nt}}\right)^2 \leq 1$$

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

R_{ut} = T = 14783,15 kg

- Connection moment

$$a = \left(\frac{\sum T}{fy\ x\ B}\right) = \frac{14783{,}15\ x\ 6}{5850\ x\ 35} = 0{,}43 \text{ cm}$$

d1 = (100/10) – 0,43 = 9,57 cm

d2 = d1 + 100/10 = 19,57 cm

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d3 = d2 + 100/10 = 29,57 cm

Σd = 58,71 cm

$$ØMn = \frac{0{,}9\ x\ fy\ x\ a^2\ xB}{2} + \Sigma dT$$

$$= \frac{0{,}9\ x\ 5850\ x\ 0{,}43^2\ x\ 35}{2} + 58{,}71 \text{ x } 14783{,}15 \text{ x } 2$$

= 17036,22 + 1735837,473

= 1752873,693 kgcm > 1316649 kgcm **...... OK !!!**

Crane Structure Planning

Structural Geometry

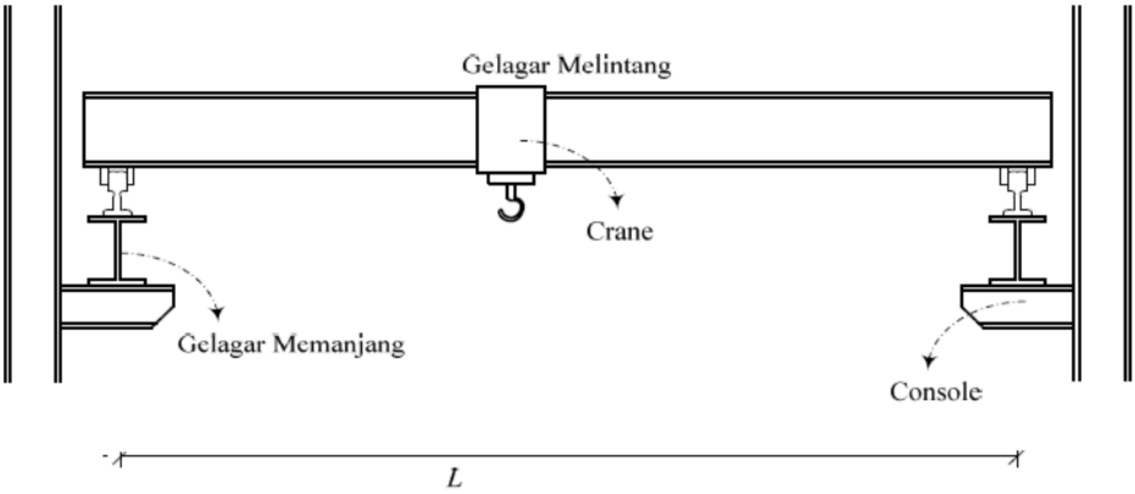

Figure 1. Bridge Beam Design

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 MPa
- Melting strength : E = 200000 MPa
- Shear Modulus : G = 80000 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 (γsteel) of 7850 kg/m3 as listed in Table 3-1 SNI 03-1727- 1989-F and Live Load which includes crane loads (Pcrane) of 10 tons = 10000 kg. [16][17][18]

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Hoist Crane Beam Planning of 30 meters

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

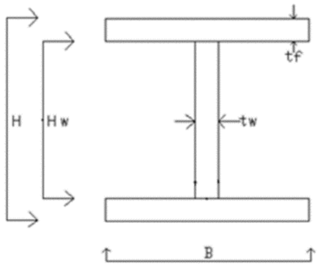

Figure 2. Hoist Crane

- The cross-sectional property data are as follows:

A = $(2.t_f.B) + (t_w.H_w)$ = 46200 mm^2

q = $A.\ \gamma_{baja}$ = 362,67 kg/m

$$I_x = 2\left\{\left[\frac{1}{12}.B.\ {t_f}^3\right] + \left[t_f.B.\left(\frac{H-t_f}{2}\right)^2\right]\right\} + \left\{\frac{1}{12}.t_w.\ {H_w}^3\right\} = 6{,}15\ .\ 10^9\ \text{mm}^4$$

$$I_y = \frac{1}{12}\left[\left(2.t_f.B^3\right) + \left(H.\ {t_W}^3\right)\right] = 7{,}93\ .\ 10^8\ \text{mm}^4$$

$$S_x = \frac{I_X}{0{,}5\ .\ H} = 1{,}07\ .\ 10^7\ \text{mm}^3$$

$$S_Y = \frac{I_Y}{0{,}5\ .\ H} = 2{,}64\ .\ 10^6\ \text{mm}^3$$

$$Z_x = \left[B.t_f.(H - t_f)\right] + \left(\frac{1}{4}.t_w.\ {H_w}^2\right) = 2{,}02\ .\ 10^7\ \text{mm}^3$$

$$Z_y = 1{,}5\ .\ S_y = 3{,}96\ .\ 10^6\ \text{mm}^3$$

$$J = \left({t_w}^3.H_w\right) + \left(2\ .\ {t_f}^3.B\right) = 1{,}19\ .\ 10^7\ \text{mm}^4$$

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

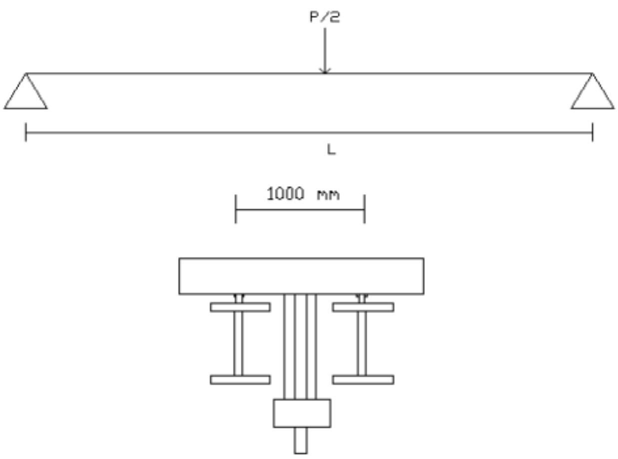

Figure 3. Hoist Crane

Hoist-crane span length : L = 30 m = 30000 mm

Crane load on 1 beam : P = 5 Ton = 5000 kg

Shock factor (impact factor) = 1,25

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Crane load design : P = 5 Ton . 1,25 = 6,25 Ton

Ultimate moment:

$M_{DL} = 1{,}2 \cdot \frac{1}{8} \cdot q \cdot L^2$

$= 48960{,}45$ kgm

$M_{LL} = 1{,}6 \cdot \frac{1}{4} \cdot P \cdot L$

$= 75000$ kgm

$M_u = M_{DL} + M_{LL}$

$= 123960{,}45$ kgm $= 1239{,}6045$ kNm

Maximum shear force due to crane load:

$V_{DL} = 1{,}2\ (q_u \cdot L)$

$= 13056{,}12$ kg

$V_{LL} = 1{,}6 \cdot \frac{P}{2}$

$= 10000$ kg

$V_u = V_{DL} + V_{LL}$

$= 23056{,}12$ kg

- Check the Slimness of the Cross

1. Wing cross section

Check cross section of the wing

$\lambda = bf/2tf = 27{,}2727$

$\lambda_p = 0{,}38 \sqrt{\frac{E}{Fy}} = 10{,}97$

$\lambda_r = 1{,}0 \sqrt{\frac{E}{Fy}} = 26{,}86$

Check,

$\lambda \leq \lambda_p$

$27{,}2727 \leq 10{,}97$ → **Not compact**

2. Body Cross

$\lambda = bf/2tf = 61{,}11$

$\lambda_p = 3{,}76 \sqrt{\frac{E}{Fy}} = 108{,}54$

$\lambda_r = 5{,}70 \sqrt{\frac{E}{Fy}} = 164{,}54$

Check,

$\lambda \leq \lambda_p$

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61,11 $\leq$ 108,54 → **Compact**

- Lateral Bending Parameter

Finding Value L_p

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{793000000}{46200}} = 130{,}975 \text{ mm}$$

$$L_p = 1{,}76 \,.\, r_y \,.\, \sqrt{\frac{E}{Fy}}$$

$$= 1{,}76 \,.\, 130{,}975 \,.\, \sqrt{\frac{200000}{240}}$$

$$= 6654{,}419 \text{ mm}$$

Finding Value L_r

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

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

$$= 2{,}49 \,.\, 10^{14} \text{ mm}^6$$

$$r_{ts} = \sqrt{\frac{\sqrt{Iy \,.\, Cw}}{S_x}} = \sqrt{\frac{\sqrt{7{,}93 \,.10^8 \; x \; 2{,}49 \,.\, 10^{14}}}{1{,}07 \,.\, 10^7}} = 203{,}322$$

C for double symmetrical I profile = 1

$$L_r = 1{,}95 \, r_{ts} \frac{E}{0{,}7 \,.\, f_y} \sqrt{\frac{J \, x \, C}{S_x \, h_o} + \sqrt{\frac{J \, x \, C}{S_x \, h_o} + 6{,}76 \left(\frac{0{,}7 \, f_y}{E}\right)}} = 30923{,}114 \text{ mm}$$

- Finding the nominal moment value

1. Melting Condition

   Mn = Mp = Zx . fy = 4861,296 kNm

2. Nominal strength of members against bending moment

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

$$C_b = \frac{12.5\, M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C} = 0{,}865595$$

$$M_n = C_b \,.\left[M_p - 0{,}7 \,.\, fy \,.\, Sx).\left(\frac{Lb - Lp}{Lr - Lp}\right)\right] = 1511{,}292 \text{ kNm}$$

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:

$$M_n = C_b \,.\left[M_p - 0{,}7 \,.\, fy \,.\, Sx).\left(\frac{\lambda b - \lambda p}{\lambda r - \lambda p}\right)\right] = 1645{,}847 \text{ kNm}$$

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**

ØMn > Mu

1360,163 kNm > 1239,6045 kNm

The value of ØMn is greater than Mu then it **fulfills the condition**

- **Sliding control**

$V_n = 0,6 \cdot f_y \cdot A_w \cdot C_v = 285120$ kg

ØVn > Vu

256608 kg > 23056,12 kg

The value of ØVn is greater than Vu then it **fulfills the condition**

- **Control against deflection**

Δ permission > Δ happen

$$\frac{L}{500} > \frac{5.q.L^4}{384.E.I_X} + \frac{P.L^3}{48.E.I_X}$$

60 mm > 59,66 mm

The value of Δ 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 kg = 66430,1 N

Mu = 14805,09 kgm

Tried using profile **WF 350x350x19x19** :

Tried B = 500 mm, dan N = 500 mm

A_1 = 250000 mm^2

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

$$f_{1.2} = \frac{P}{B \, x \, N} \pm \frac{M \, x \, c}{I} = = \frac{6643,01}{500 \, x \, 500} \pm \frac{14805090 \, x \, 250}{5208333333} = -0,2657 \pm 0,71064$$

f_1 = -0,4449

f_2 = 0,9764

Try column with size 600 x 600 mm → A_2 = 360000 mm^2

$$\frac{A2}{A1} = \frac{360000}{250000} = 1,44 \text{ mm}$$

$$f_p = 0,85 \text{ x } \varphi_c \text{ x f'c} \sqrt{\frac{A2}{A1}}$$

= 0,85 x 0,6 x 20 x 1,2

= 12,24 MPa

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f_2 = 0,9764 < f_p **............OK!!!**

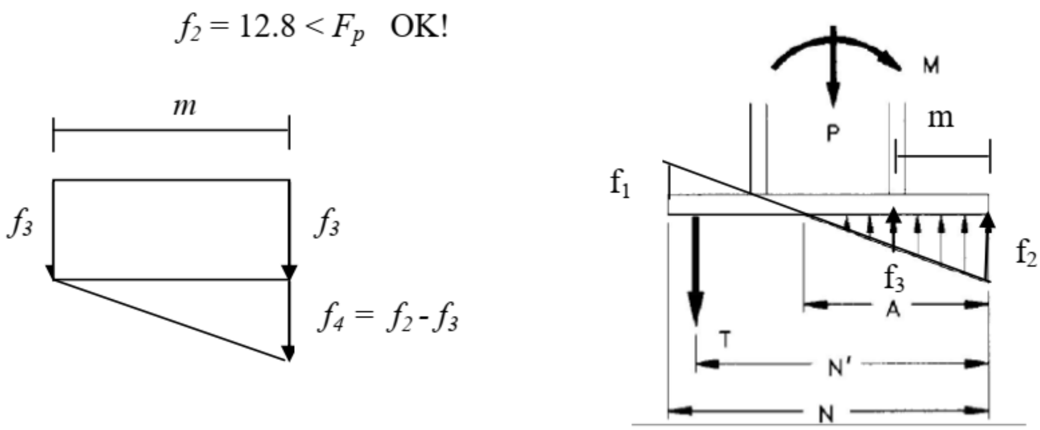

Figure 4. Column With Load

Determining the Plate Thickness

m $= \frac{N-0{,}95d}{2} = \frac{350-(0{,}95\,x\,350)}{2}$ = 83,75 mm

n $= \frac{B-0{,}8bf}{2} = \frac{350-(0{,}8\,x\,357)}{2}$ = 107,2 mm

Neutral Line

A $= \frac{f2}{f2+f1}$ x N $= \frac{0{,}97636472}{0{,}97636472+0{,}44}$ x 500 = 343,70

f_3 $= \frac{A-m}{A}$ x f_2 $= \frac{343{,}70-83{,}75}{343{,}70}$ x 0,97636472 = 0,738454

f_4 = f_2 - f_3

= 0,97636472 - 0,738454

= 0,237911

M_{plu} $= (\frac{1}{2}$ x $f_3 + \frac{1}{3}$ x $f_4)$ m^2 x B

$= (\frac{1}{2}$ x 0,738454 $+ \frac{1}{3}$ x 0,237911) $83{,}75^2$ x 500

= 1503480 mm

Plate moment capacity

Mn $= \varphi_y$ Z $f_y = \varphi_y$ $(\frac{1}{4}$ B $t_p^2)$ x f_y = 0,9 $(\frac{1}{4}$ B $t_p^2)$ x $f_y = M_{plu}$

t_p $= \sqrt{\frac{4\,M_{plu}}{0{,}9\,B\,fy}}$ $= \sqrt{\frac{4\,x\,1503480}{0{,}9\,x\,500\,x\,240}}$ = 7,46 → 8mm

Conclusion

Size Plate 500 x 500 x 8 mm

Column Size 600 x 600 mm

4. Conclusion and Suggestion

4.1 Conclusion

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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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