Cranes Loads

Middle span Crane loads :

Crane capacity = 32t , span = 38m , wheel spacing = σ = 5.8 m
Max Wheel load = P_{max} = 33 t , Min wheel load = P_{min} = 9.4t

$V_{LL\ \mathbf{max}}$ **=** P_{max} * (1 + (S – σ) / S) = 35 t , $V_{LL\ min}$ = 10 t , V_{DL} = 0.2 t/m\ * 6m / 2 = 0.6 t

BF_{max} = 1/7 * 2 * P_{max} = **10t** , BF_{min} = 2.7 t

H_{max} = 0.1 V_{LL} = **0.4 t** , H_{min} = 1t …… Taken **1.9t**

Vt_{max} = **1.25** V_{LL} + V_{DL} = 45 t …. Taken **40t** , Vt_{min} = 14t …. Taken **19t**

Crane Grider :

Assume that girder resist only one wheel as wheel spacing is 5.8 m

moment due to load = 1.25 x (P_{max} x girder length / 4) = 61.5 t.m -> 1.25 impact factor

moment due to ow = 0.1t/m x 6^2 / 8 = 0.45 t.m

M_x = total moment = 62 t.m

assume web dimensions = 300 x 10 mm , flange thickness = 20 mm

web depth / web thickness = 30 < 190 / $fy^{0.5}$, safe for local buckling

required inertia = 53142 cm^4

flange length = 400 mm , flange length / 2x thickness = 10 < 21 / $fy^{0.5}$…. Safe for local buckling

allowable shear = web area * yield stress = 30 x 1 x 2.1 = 60 tons > P_{max} **…….safe**

M_y = H_{max} x girder length / 4 = 0.4 x 6 / 4 = 0.6 t.m -> resisted by upper plate only

Plate inertia = 40^3 x 2 /12 = 10666 cm^4

$M_{y\ allowable}$ = stress x I_x/y = 2.1 x 10666 / 20 = 1120 t.cm = 11.2 t.m …… safe

Deflection calculations :

Deflection = P L^3 / 3EI = 0.22 cm

Allowable deflection = span / 800 = 0.75 cm …….. safe

Crane bracing :

Design on Breaking force

Design force = 10 / cos(45) = 14.1 tons

L_u = 1.1 m , use bracing of HSS 100X4 ,

Area = 15.5 cm^2 , inertia = 236cm^4 , ix = 3.9 cm , Lx/ix = 28.6

Allowable stress = 1.2 t/cm^2

Applied stress = 0.91 t/cm^2 ………… safe

Use weld of 4 mm

Weld length for box and plate = 10 / (0.2 x 5.2 x 0.4) + 2 x 0.4 = 24cm

Each weld line length for box section = 24cm / 4 = 6 cm

Gusset plate weld length = 20 cm from both sides , with thickness 5 mm

Gusset plate weld -> shear stress = normal stress = 10 / (0.5 x 2 x 20) = 0.5 t/cm^2

Gusset plate weld combined stress = 1 ton/cm^2 < 1.1 x 1.04 t/cm^2 …… safe

Left span crane loads :

Capacity = 10t >>>>> 16 t , crane span = 24 m , wheel spacing = σ = 4.56 m

CTG span = S = 6m

Max wheel load = P_{max} = 8 t

Minimum Wheel load = P_{min} = 2.4t

Max Lateral shock = 0.1 * 8t = 0.8t ….. taken **1.15 t**

Min Lateral shock = 0.31 t **……** taken **0.7t**

V_{DL} = 0.2t/m\ * 6m /2 = 0.6t

$V_{LL} = P_{max} + P_{max} * (S - \sigma) / S$ = **10 t**

$Vt_{max} = 1.25\ V_{LL} + V_{DL}$ = 13t …… taken **11.5t** **,** Vt_{min} = 3.6t ….. taken **7.2t**

$BF_{max} = 2/7 * P_{max}$ = 2.3t …. Taken **3.3t**

Crane Grider :

Assume that beam resists only one crane wheel as wheels spacing is 4.56 m

Max moment = 0.1t/m * 6^2 / 8 + 8 * 6 /4 = 12.5 tons

web dimensions = 200 x 10 , web depth / thickness < 190/$fy^{0.5}$ …… safe for local buckling

use flange thickness = 20 mm

required inertia = 7143 cm^4

flange length = 12 cm , flange length / thickness < 21/$fy^{0.5}$ … safe

deflection = P L^3 / 3EI = 0.4 cm

allowable deflection = span / 800 = 0.75 cm ….. . safe

required bracket :

moment = P_{max} x (1 + 4.56 / 6) x 0.3 x 1.25 = 5.3 t.m

use web dimension = 150 x 10 mm , flange thickness = 10

required inertia = 2145 cm^4

flange length = 15 cm -> flange length / 2 x thickness = 12.5 < 21 /$fy^{0.5}$ ….. safe for L.B

Crane Bracing Calculations :

Lateral shock = (1.15 + 3.3) / cos(45) = 6.3 t

L_u = 1 m , use bracing of HSS 100X4 ,

Area = 15.5 cm^2 , inertia = 236cm^4 , ix = 3.9 cm , Lx/ix = 25.6

Allowable stress = 1.3 t/cm^2

Applied stress = 0.41 t/cm^2 ………… safe

Use weld of 4 mm

Weld length for box and plate = 6.3 / (0.2 x 5.2 x 0.4) + 2 x 0.4 = 16cm

Weld length for each weld line in box section = 16 / 4 = 4 cm , take it 5 cm

Weld for gusset plate , length = 100 mm , thickness 4 mm , from both sides

Shear stress on gusset plate weld = normal stress = (1.15 + 3.3) / (10 x 0.4 x 2) = 0.56 t/cm^2

Combined stress for gusset plate weld = 1.125 t/cm^2 < 1.1 x 1.04 t/cm^2 …..safe

Corrugated Sheets

roof Corrugated sheets :

Middle span Live load = 60 – 200/3 * tan(α) = 60 – 200/3 * 0.06 = 56 kg / m^2

- use corrugated sheet of allowable live load = 100 kg / m^2 , span **2 m**
  use **continuous** corrugated sheet of thickness **0.55 mm**
- Ow Dead Load = 5.25 kg/m^2
- **Total load Gravity** = 5.25 + 56 = **61.25** kg/m^2
- **Wind Load** (wind side) = q * c * K * a / cos(angle) = 50kg/m^2 * -0.8 * 1.15 / cos(tan^{-1}(1200/20000)) = -**46 kg/m^2**
- **Wind Load** (wind opposite side) = q * c * K * a / cos(angle) = 50kg/m^2 * -0.5 * 1.15 / cos(tan^{-1}(1200/20000)) = -**29 kg/m^2**

side spans live load = 60 – 200/3 * tan(α) = 60 – 200/3 * 0.1 = 53 kg / m^2

- use corrugated sheet of allowable live load = 100 kg / m^2 , span **2.5 m**
- use **continuous** corrugated sheet of thickness **0.7 mm**
- ow = 6.66 kg/m^2
- **Total load =** 6.66 + 53 = **60** kg/m^2
- **Wind Load** = q * c * K * a / cos(angle) = 50kg/m^2 * -0.8 * 1.15 / cos(tan^{-1}(1200/12500)) = -**46 kg/m^2**
- **Wind Load** (wind side) = q * c * K * a / cos(angle) = 50kg/m^2 * -0.5 * 1.15 / cos(tan^{-1}(1200/20000)) = -**29 kg/m^2**

side Corrugated sheets :

level > 10 m , Span = 2 m

- wind load (wind direction) **=** C_e * K * q = 0.8 * 1.15 * 50 = 50 kg / m^2 ,
- Wind load (opposite wind direction) = **=** C_e * K * q = -0.5 * 1.15 * 50 = -**32 kg / m^2** ,
- Ow = 4.75 kg/m^2 (vertical load)
- Use **continuous** corrugated sheets for all side of thickness **0.5 mm**

level < 10 m , Span = 2.5 m

- Use corregated sheet of allowable load = 50 kg / m^2
- **wind load** (wind direction) **=** $C_e * K * q = 0.8 * 1.0 * 50 = 40 kg/m^2$
- **wind load** (wind opposite dir.) **=** $C_e * K * q = -0.5 * 1.0 * 50 = -25 kg/m^2$
- Ow = 4.75 kg/m^2 (vertical load)
- Use **continuous** corrugated sheets for all side of thickness **0.5 mm**

Corrugated Sheets Summery :

- Use continuous in **middle span** roof corrugated sheets of **0.7 mm**
- Use continuous in **side spans** roof corrugated sheets of **0.55 mm**
- Use continuous in **side corrugated** sheets of **0.50 mm**

Mezanin

Flooring = 200 kg/m^2

Storage Floor Live Load = 500 Kg/m^2

Management Floors Live Load = 400 Kg/m^2

Walls distributed load = 200 kg/m^2

Deck span = 2.50 m

Use Metal Deck thickness = 1.2 mm

For **Storage Floor** Use concrete thickness = 8 cm

- concrete load = 2500 kg/m^3 * 0.08 = 200 kg/m^2

- total dead load = 600 kg/m^2
- total live load = 500 kg/m^2
- total working load = 1100 kg/m^2
- Total ultimate load for storage floor = 1.4 * (200 + 200 + 200) + 1.6 * 500 = 1640 kg/m^2
- Allowable load for storage floor = **1758 kg/m^2**

For **management Floor** Use concrete thickness = 7 cm

- concrete load = 2500 kg/m^3 * 0.08 = 200 kg/m^2
- total dead load = 575 kg / m^2
- total live load = 400 kg/m^2
- total working load = 975 kg/m^2
- Total ultimate load for mang. floor = 1.4 * (200 + 175 + 200) + 1.6 * 400 = 1445 kg/m^2
- Allowable load for management floor = **1542 kg/m^2**

<u>Mezanin Summery :</u>

Use Metal Deck of **1.2 mm**

For Management Floors , Concrete Thickness = **7 cm**

For storage Floor , Concrete Thickness = **8 cm**

Thickness	0.8 mm.				1.0 mm.				1.2 mm.				1.6 mm.			
Span \ Concrete thickness	5 cm.	6 cm.	7 cm.	8 cm.	5 cm.	6 cm.	7 cm.	8 cm.	5 cm.	6 cm.	7 cm.	8 cm.	5 cm.	6 cm.	7 cm.	8 cm.
2.0 m	1500	1760	2070	2360	1630	1900	2210	2520	1850	2130	2440	2770	1940	2230	2540	2890
2.4	1000	1180	1400	1630	1100	1280	1500	1730	1290	1480	1700	1930	1350	1550	1770	2000
2.8	610	750	880	1050	720	860	990	1160	920	1060	1210	1380	990	1140	1300	1470

Mezanin <u>storage</u> floor main beams (in plan) (using metal deck allowable load) :

- Dead load from secondary beam = 1.6t/m * 6m = 9.6 t , secondary beams from one side , @2.5m spacing (6m is instead of 2* span /2 = 2 * 6 / 2 = 6m)
- Live load from secondary beam = 1.25t/m * 6m = 7.5 t , secondary beams from one side , @2.5m spacing
- Total loads = 9.6 + 7.5 = 17 tons @ middle of span

Mezanin <u>management</u> floor main beams (using metal deck allowable load) :

- Dead load = 1.55t/m * 6m = 9.3 t, secondary beams from one side , @2.5m spacing
- Live load = 1t/m * 6m = 6 t , secondary beams from one side , @2.5m spacing
- Total load = 16 tons

Mezanin Storage Floor Secondary Beams (out of plan beams) :

- Dead load = 0.6t/m^2 * 2.5m = 1.5 t/m
- Total Live load = 0.5t/m^2 * 2.5m = 1.25 t/m
- ow = 100 kg / m
- Total Dead load = 0.1t/m + 1.5t/m = 1.6 t/m
- Total reaction = 8.5 tons
- Total Load = 2.85 t/m
- Moment = 12.8 t.m
- Required inertia = 12495 cm^4
- Web depth = 380 mm
- Web thickness = 10 mm
- Flanges thickness = 10 mm
- Flanges width = 100mm

Mezanin Management Floor Secondary Beams (out of plan) :

- Dead load = 0.58t/m^2 * 2.5m = 1.45 t/m
- Ow = 0.1 t/m
- Total Dead load = 1.55 t/m
- Live load = 0.4t/m^2 * 2.5m = 1 t/m
- Total Load = 2.55 t/m
- Total reaction = 8 tons
- Moment = 11.5 t.m
- Web dim. = 340 x 10 mm
- Flange thickness = 10 mm
- Required inertia = 10130 cm^4

Flange length = 120 mm

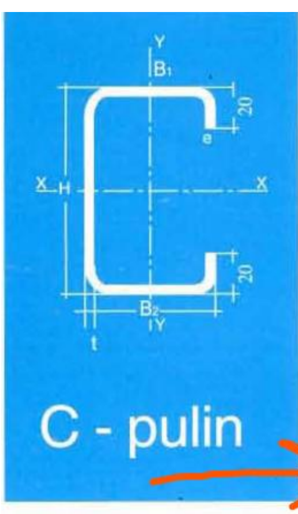
Roof Purlins Design

Purlins Middle span :

- **Live** load = 56 kg/m^2 * 2m = **112 kg/m**
- Ow = 25 kg/m
- **Dead** load = 5.25 kg/m^2 * 2m + 25kg/m = **30.5** kg/m
- Dead Load Reactions = 30.5kg/m * 6m / 2 = 91.5 kg = 0.092t
- Live load Reactions = 112kg/m * 6m / 2 = 336 kg = 0.34t
- Total load Reactions = 427.5 kg = 0.43 t
- Total load = 30.5kg/m + 112kg/m **= 0.15 t/m\ .... critical purlins design load**
- Wind Load (wind direction) = -46kg/m^2 * 2m = -92 kg/m\ = -0.1 t/m\
- Wind Load (opposite direction) = -29kg/m^2 * 2 = -0.06 t/m\
- Wind Load (wind direction) Reaction = -46kg/m^2 * 2m * 6m /2 = -0.28t
- Wind load (opposite wind dir) Reaction = -29kg/m^2 * 2m * 6m /2 = -0.18t
- Frame span = 6m -> Max Moment = 0.15 * 6^2 / 8 = 0.68 t.m = 68 t.cm

Purlins Side Spans :

- **Live** Load = 53 kg/m^2 * 2.5m = **132.5 kg/m**
- Ow = 25 kg/m
- **Dead** Load = 25kg/m + 6.66kg/m^2 * 2.5m = **41.65 kg/m**
- **Dead** load **Reactions** = 41.65kg/m * 6m / 2 = 125 kg = 0.125 t @ 2.5m span
- **Live** load **Reactions** = 132.5kg/m * 6m / 2 = 397.5 kg = 0.4t @ 2.5m span
- Total load Reaction = 0.525t
- Total Load = 41.65 kg/m + 132.5 kg/m **= 0.18 t/m\ ……** **Critical purlins design load**
- Wind Load (wind direction) = -46kg/m^2 * 2m = -92 kg/m\ = -0.1 t/m\
- Wind Load (opposite direction) = -29kg/m^2 * 2 = -0.06 t/m\
- Wind Load (wind direction) Reaction = -46kg/m^2 * 2.5m * 6m /2 = -0.35t
- Wind load (opposite wind dir) Reaction = -29kg/m^2 * 2.5m * 6m /2 = -0.22t
- Frame span = 6m -> Max Moment = 0.18 * 6^2 / 8 = 0.81 t.m = 81 t.cm ,
- Zx = 57cm^3



DIMS				G. PROPERTIES		X - X		Y - Y	
H	B_1	B_2	t	AREA	Kg Wt/m	IX	ZX	IY	ZY
mm			mm	cm^2	ks / m	cm^4	cm^3	cm^4	cm^3
100	50	50	2	2.64	3.712	73.92	14.78	17.595	5.64
150	50	50	2	5.8	4.55	199.8	26.65	26.06	7.24
200	60	60	2	7.04	5.632	417.95	41.8	34.005	7.87
250	70	70	2	8.44	6.63	772.39	61.97	52.47	10.11

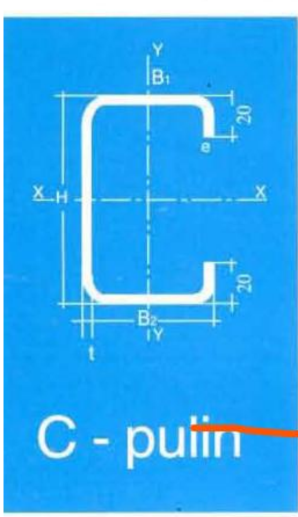
Side Purlins

Side Purlins Beams < 10.00 m level:

- Wind Lateral Load (wind direction) = 40kg/m^2 * 2.5m = 100kg/m **= 0.1 t/m** **.....critical purlins design load**
- Wind Lateral Load (opposite wind dir) = -25kg/m^2 * 2.5m = 63kg/m = 0.063 t/m
- Vertical load = corrugated sheet ow + purlin ow = 4.75 kg/m^2 * 2.5m + 25kg/m = 36.9kg/m **= 0.04 t/m**\
- Lateral Reaction (wind direction) = 0.1t/m * 6m / 2 = 0.3t
- Later reaction (opposite wind dir) = 0.063t/m * 6m /2 = 0.19 t
- Vertical Reaction = 36.9kg/m * 6m / 2 = 0.11t
- Max moment = 0.45 t.m = 45 t.cm ,

Side Purlins Beams > 10.00 m level: (design like purlins at level below 10.00m)

- Wind Lateral Load (wind direction) = 46kg/m^2 * 2 m = 92kg/m = 0.092 t/m
- Wind Lateral Load (opposite wind dir) = -32kg/m^2 * 2 m = 64kg/m = 0.064 t/m
- Vertical load = corrugated sheet ow + purlin ow = 4.75 kg/m^2 * 2 m + 25kg/m = 35kg/m **= 0.035 t/m**\
- Lateral Reaction (wind dir) = 0.092t/m * 6m / 2 = 0.28t
- Lateral Reaction (opposite wind dir) = 0.064t/m * 6m / 2 = 0.19t
- Vertical Reaction = 35kg/m * 6m / 2 = 0.11t
- Moment = 0.092 t/m * 6^2 / 8 = 0.414 t.m = 41 t.cm
- Zx = 41 t.cm / 0.58*fy = 41/ 1.4 = 29.3 cm^3



DIMS				G. PROPERTIES		X - X		Y - Y	
H	B_1	B_2	t	AREA	Kg Wt/m	IX	ZX	IY	ZY
mm			mm	cm2	ks / m	cm4	cm3	cm4	cm3
100	50	50	2	2.64	3.712	73.92	14.78	17.595	5.64
150	50	50	2	5.8	4.55	199.8	26.65	26.06	7.24
200	60	60	2	7.04	5.632	417.95	41.8	34.005	7.87
250	70	70	2	8.44	6.63	772.39	61.97	52.47	10.11

Frame girder load

- Purlins concentrated **Dead** loads from each sides= 2 * 0.125t = **0.25t** ,9purlins@2.5m
- Purlins concentrated **Live** loads from each sides = 2 * 0.4t = **0.8t** ,9purlins@2.5m
- Utilities **Dead** loads = **0.1 t /m**

Truss loads

- Purlins concentrated **Dead** loads at nodes from each sides = 2* 0.092t = **0.185t** ,purlins@2.5m at each node
- Purlins concentrated **Live** loads at nodes from each sides = 2* 0.34t = **0.68t** ,purlins@2.5m at each node
- Utilities **Dead** Load = 100 kg / node = **0.1t** /node

Truss Tension Splice :

Connected box HSS 70X5

Tension force = 24 tons

Use 4 bearing bolts M10/10.9

Force on one bolt = 6 tons

Allowable force on bolt = 3.14 x 0.25 x 10.9 = 8.5 tons ……… safe

Use plate 130 X 120 X 20

Moment on plate = 12 tons X 1.5cm = 18 ton.cm

Plate inertia = 8 cm^4

Stress on plate = 2.25 ton/cm^2 < 2.4 ton/cm^2 …………. Safe

Truss Compression Splice and diagonal members :

The connected box HSS 90X5

Compression force = 24 tons

Use plate 150 x 120 x 20

Stress on plate = 24 / 144 = 0.17 ton/cm^2

Moment on plate = 0.24 x ((15 – 9) / 2)) ^{2} = 2.16 ton.cm/cm\

Stress on plate = 2.16 X 0.5 X 1.5 = 1.62 ton/cm^2 < 2.4 ton/cm^2 ……….safe

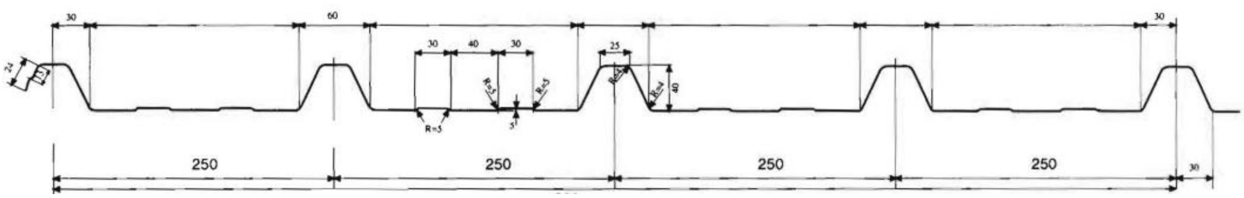
Use minimum bearing bolts 2 M10/10.9

End Gable Purlins

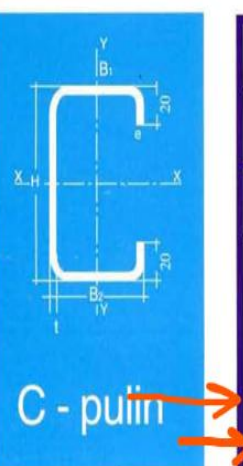
Wind load = 46kg/m^2 -> span = 2m

Wind load = 40 kg/m^2 -> span = 2.5m

Use corrugated sheets of **thickness = 0.5 mm ,** weight = 4.8 kg/m^2



Used purlins :



DIMS				G. PROPERTIES		X - X		Y - Y	
H	B₁	B₂	t	AREA	Kg Wt/m	IX	ZX	IY	ZY
mm			mm	cm2	ks / m	cm4	cm3	cm4	cm3
100	50	50	2	2.64	3.712	73.92	14.78	17.595	5.64
150	50	50	2	5.8	4.55	199.8	26.65	26.06	7.24
200	60	60	2	7.04	5.632	417.95	41.8	34.005	7.87
250	70	70	2	8.44	6.63	772.39	61.97	52.47	10.11

n	length m	load t/m	moment tm	stress = Mx / Zx	zx
1	4	0.1	0.2	0.686577412	29.13
2	4	0.1	0.2	0.686577412	29.13
3	4	0.05	0.1	0.343288706	29.13
5	9	0.1	1.0125	1.599526066	63.3
6	9	0.1	1.0125	1.599526066	63.3
7	4.5	0.1	0.253125	0.868949537	29.13
8	4.5	0.1	0.253125	0.868949537	29.13
9	8	0.1	0.8	1.263823065	63.3
10	8	0.1	0.8	1.263823065	63.3
11	4	0.08	0.16	0.549261929	29.13
12	4	0.06	0.12	0.411946447	29.13

Stairs

Live Load = 500 kg/m^2

Dead load on Carriage = 100 $kg/m^{\backslash}$

Steps Calculations :

Stair width = 2.5 m

stair Tread = 267 mm

stair riser = 167 mm

distributed load on Tread = 0.267m * 0.5t/m^2 = 0.134 $t/m^{\backslash}$

Moment on Tread = 0.134$t/m^{\backslash}$ * 2.5^2 / 8 = 0.105 t m

Shear on Tread = 0.17 tons

Use chakkar plates of steel grade 37

Required stiffness = 7.5 cm^4

Chakkar plate stiffness of thickness 1 mm = 16^3 x 0.1 /12 = 34 cm^4 -> safe

Using two Bolts to fixing Tread with Angle of Diameters = 10 mm

Stress due to moment = Mx * t_p / (2 I_x) = 6 Mx / (t_p^2 * width) = 0.105 * 100 / (1.2^2 * (26.7 - 2)) = 1.77 t/cm^2 < (0.58 * fy = 2.1 t/cm^2) ……… ok

Stress due to shear = Q / (t_p * width) = 0.17 / (1.2 * (26.7 - 2)) = almost zero

Step Fixation Angle Calculations :

Bolts construction conditions = 3 * bolts diameter * number of bolts = 60 mm

Welding length = Q / (allowable weld stress * S_w) = 0.17 / (0.2 * 5.2 * 0.4) = 0.35 cm = 3.5 mm

*Use angle 50*5*5 of length 10 cm*

Base connection design

use base plate of 220 x 200 x 10

base load = 2.2 tons

stress on concrete = 2.2 / (22 x 20) = 0.005 ton / cm^2 -> safe on concrete bearing applied moment on plate = $0.005 \times 6.25^2 / 2 = 0.1$ ton.cm/cm\

applied stress on plate = $0.1 \times 0.5 / (1 / 12) = 0.6$ tons/cm^2 < 3.6 ton/cm^2 -> safe

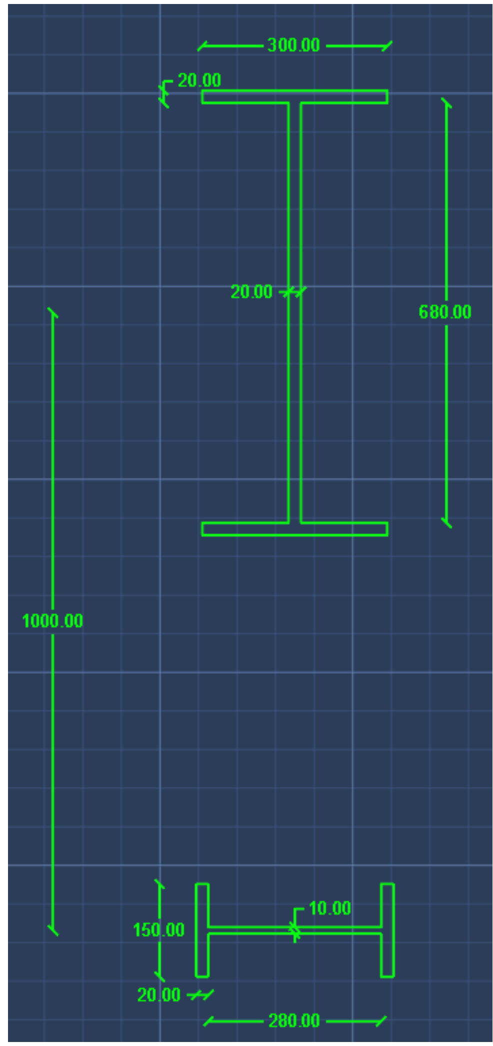
base welding :

use minimum weld thickness for base plate = 4 mm

plate length required = $2.2 / (0.2 \times 5.2 \times 0.4) + 2 \times 0.4 = 6$ cm

use minimum weld in both sides of web , 5cm length with 4 mm thickness

Laced Column Calculations



centroid:
Y = 76.32cm
X = 16cm

inertia:
Ix = 998867cm4 = 9.99x10E9 mm4 == 10E10mm4
Iy = 26223 cm4 = 2.62x10E8

Area = 372 cm2

ix = 51.8 cm
iy = 8.395cm

Lx (in plane length) = 10m * 2 = 20m = 2000cm (fixed free)
Ly (out of plane) = 870cm (fixed hinged)

lambda @y = Ly / iy = 103.57cm
lambda @x = 38.6cm

Fcall = 7500 / lambda-squared = 0.7 t/cm2

applied load = 133 + 91 = 224 tons

Fapplied = 0.6 t/cm2 (safe)

End Gable

Base Design (calculation for max column loads @ axis A +8m rigth):

Max shear on end gable base = 1.2 tons

Max Normal force = purlins own weigth x columns span x Purlins for column + column o.w + corrugated sheet ow x area

Max Normal force = 5.6 kg/m x (9 + 4) / 2 x 5 + 0.1t/m x 14m + 5.75 kg/m^2 x (9 + 4) /2 * 10m
Max Normal force = 2 tons

use column base steel plate = 15 x 32 cm x 1 cm

Concrete allowable stress = 75 kg/cm^2

Applied normal stress = 2 / (15 x 32) = 0.0042 ton / cm^2

Allowable normal force = 15 x 32 x 75 / 1000 = 34 tons ………. Safe

Use Minimum tie rod Diameter , use M20 / 10.9

R_{shear} = 0.2 x 10.9 x 3.14 x 2^2 / 4 = 6.8 tons

$R_{bearing}$ = Diameter x plate thickness x F_u x 0.6 = 2 x 1 x 5.2 x 0.6 = 6.24 tons

R_{min} = 6.24 tons ……….. safe

Use minimum weld thickness = 4mm

Use weld in web only -> weld length = (20 – 0.8) x 2 = 38 cm

Normal stress = 0.0521 ton/cm^2 < 1.04 t/cm^2 ………..safe

Shear stress = 0.031 t/cm^2 < 1.04 t/cm^2 ………safe

Combined stresses < 1.1 x 1.04 t/cm^2 ………… safe

Moment applied on plate = ((plate length – col height) / 2) ^{2} x stress on concrete = 0.07 t.m/m\

Stress on plate = moment x y / I_x = 0.07 x 0.5 / (1/12) = 0.4 ton/cm^2 < 2.4 ton/cm^2 -> safe

Gusset plates stiffeners calculations

Buckling limit is L / i_x < 16

All gusset plates are with thickness 10 mm

Without stiffeners $L_{allowable}$ = 16 x i_x = 16 / (b x t / (b x t^3 / 12) $)^{0.5}$ = 4.5 x t mm = 45 mm

With stiffeners , $L_{allowable}$ = n x 4.5 x h mm , where n is stiffeners count , h is stiffener height and use stiffener thickness similar to plate thickness 10 mm , neglect plate actions.

use stiffener directly with minimum weld thickness 4mm and minimum weld length 5 cm in both sides