ORTA DOĞU TEKNİK ÜNİVERSİTESİ
MIDDLE EAST TECHNICAL UNIVERSITY

CE482 DESIGN OF STEEL STRUCTURE

FINAL PROJECT

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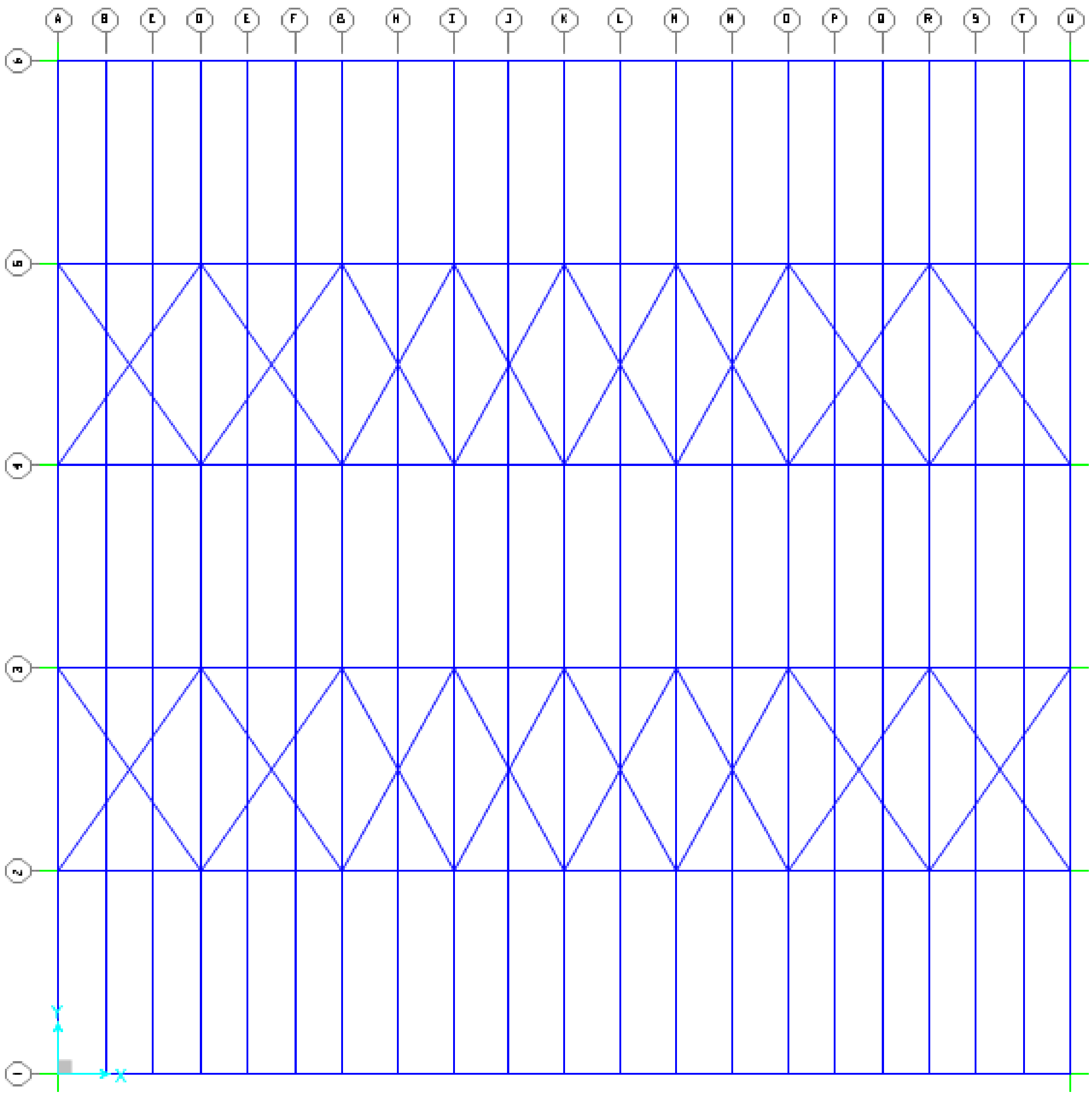
Table of Contents

1. Introduction

The design is about 60 x 60 meter roof having 10% slope. It consists of sandwich panel and designing truss system, purlin design. In other words, the project concept is designing all part of the roof without column design. The best way such a long span length distance design is using steel; therefore, there is steel roof designed in this specific project. Sections are shown like below:



2. Load Cases

Dead Load

- **Assuming Truss Weight:** 0.4 kPa
- **Cover load + service load:** 0.4 kPa
- **HVAC:** 0.2 KPa

Total Dead Load: 1 kPa

Snow Load

$$s = \mu_i \; x \; C_e \; x \; C_t \; x \; s_k$$

$$s = 0.8 \; x \; 1 \; x \; 1 \; x \; 0.75 = 0.6 \; kPa$$

Snow Load: 0.6 kPa

Wind Load

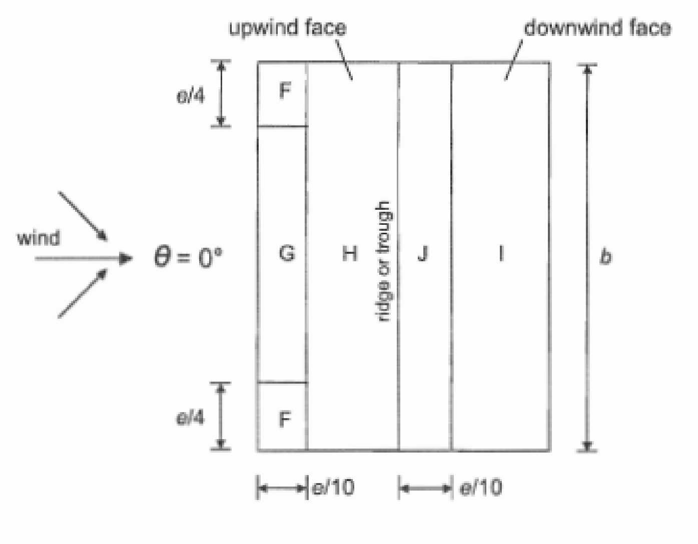
- Terrain Category: III -> $z_0 = 3$ m, $z_{min} = 5$ m
- $V_b = 42$ m/s
- Peak Pressure:
  - $q_p(z) = [1 + 7 \; x \; I_v(z)] \; x \; 0.5 \; x \; p \; x \; v_m(z)^2$
  - $v_m(z) = c_r(z) \; x \; c_0(z) \; x \; v_b \; where \; c_r(z) = k_t \; x \ln(z/z_0)$
  - $k_t = 0.19 \; x \; (z_0/z_{0,II})^{0.07}$
  - $c_r(z) = 0.19 \; x \; (0.3/0.05)^{0.07} \; x \ln(z/0.3) = 0.2154 \ln(z/0.3)$
  - $v_m(z) = 0.2154 \ln(z/0.3) \; x \; 1 \; x \; 42 = 9.046 \; \ln(z/0.3)$
  - $I_v(z) = \frac{k_I}{[c_0(z) \; x \; \ln(z/z_0)]} = \frac{1}{1 \; x \ln(z/z_0)}$

$$q_p(z) = \left[1 + \frac{7}{\ln(z/0.3)}\right] x \; 0.5 \; x \; 1.25 \; x \; [9.046 \; \ln(z/0.3)]^2$$

z = 36 m - > $\mathbf{q_p(z) = 2.89 \; kN/m^2}$

- **Vertical Wind Direction $\theta = 0$**

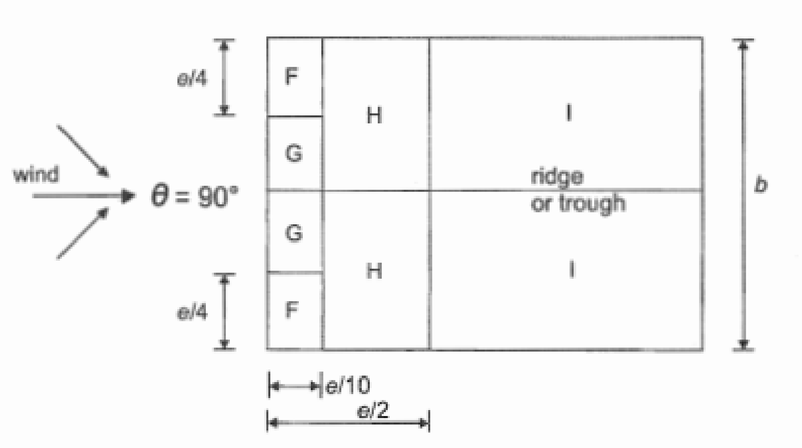
e = 60 meter, $\alpha = 5.71^0$



(b) wind direction θ = 0°

Section	C_{pe}
F	-1.64
G	-1.17
H	-0.58
J	-0.63
I	-0.59

- **Vertical Wind Direction $\theta = 90$**

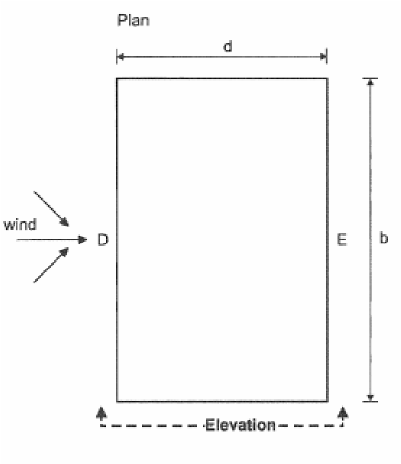


(c) wind direction θ= 90°

Section	C_{pe}
F	-1.58
G	-1.3
H	-0.69
I	-0.59

- **Horizontal Wind Direction**

h/d=



36/60 = 0.6

Section	C_{pe}
D	0.8
E	-0.5

3. Eave Height

The point of determination of eave height is to optimize the forces acting on the truss members. Increasing eave height leads to decreasing forces acting on the member. However, due to the slenderness of the sections, increasing the section length too much is not efficient way. Experiences show that eave height should be approximately one tenth of the span length of the roof.

To determine eave height, 4, 5, 6 and 7 meter eave height trials are used with respect to combination of dead load and snow load without factorized. At the end of the SAP analysis indicates that increasing eave height decreasing bar forces until h = 6 meter. After this height, bar forces become stable. There is no need to increase eave height after then due to the slenderness increasing. Therefore, eave height is chosen 6 meter.

4. Load Combinations

Dead load: (DL = DL0 + DL1 + DL2)

DL0: Self Weight

DL1: HVAC

DL2: Covering

Snow load: SL

Wind load: WX[+], WX[-],WY[+], WY[-]

By equation 6.10:

LVA	DL is unfavorable		LVA	DL is favorable	
SL	**Comb1**	1.35DL	SL	**Comb15**	1.35DL
	Comb2	1.35DL + 1.5SL		**Comb16**	1.35DL + 1.5SL
	Comb3	1.35DL + 1.5SL + 0.9 WX[+]		**Comb17**	1.35DL + 1.5SL + 0.9 WX[+]
	Comb4	1.35DL + 1.5SL + 0.9 WX[-]		**Comb18**	1.35DL + 1.5SL + 0.9 WX[-]
	Comb5	1.35DL + 1.5SL + 0.9 WY[+]		**Comb19**	1.35DL + 1.5SL + 0.9 WY[+]
	Comb6	1.35DL + 1.5SL + 0.9 WY[-]		**Comb20**	1.35DL + 1.5SL + 0.9 WY[-]
WX[+]	**Comb7**	1.35DL + 1.5WX[+]	WX[+]	**Comb21**	1.35DL + 1.5WX[+]
	Comb8	1.35DL + 1.5WX[+] + 0.75SL		**Comb22**	1.35DL + 1.5WX[+] + 0.75SL
WX[-]	**Comb9**	1.35DL + 1.5WX[-]	WX[-]	**Comb23**	1.35DL + 1.5WX[-]
	Comb10	1.35DL + 1.5WX[-] + 0.75SL		**Comb24**	1.35DL + 1.5WX[-] + 0.75SL
WY[+]	**Comb11**	1.35DL + 1.5WY[+]	WY[+]	**Comb25**	1.35DL + 1.5WY[+]
	Comb12	1.35DL + 1.5WY[+] + 0.75SL		**Comb26**	1.35DL + 1.5WY[+] + 0.75SL
WY[-]	**Comb13**	1.35DL + 1.5WY[-]	WY[-]	**Comb27**	1.35DL + 1.5WY[-]
	Comb14	1.35DL + 1.5WY[-] + 0.75SL		**Comb28**	1.35DL + 1.5WY[-] + 0.75SL

* LVA: Leading variable action

By equation 6.10(a):

LVA	DL is unfavorable		LVA	DL is favorable	
SL	**Comb29**	1.35DL	SL	**Comb43**	1.35DL
	Comb30	1.35DL + 0.75SL		**Comb44**	1.35DL + 0.75SL
	Comb31	1.35DL + 0.75SL + 0.9 WX[+]		**Comb45**	1.35DL + 0.75SL + 0.9 WX[+]
	Comb32	1.35DL + 0.75SL + 0.9 WX[-]		**Comb46**	1.35DL + 0.75SL + 0.9 WX[-]
	Comb33	1.35DL + 0.75SL + 0.9 WY[+]		**Comb47**	1.35DL + 0.75SL + 0.9 WY[+]
	Comb34	1.35DL + 0.75SL + 0.9 WY[-]		**Comb48**	1.35DL + 0.75SL + 0.9 WY[-]
WX[+]	**Comb35**	1.35DL + 0.9WX[+]	WX[+]	**Comb49**	1.35DL + 0.9WX[+]
	Comb36	1.35DL + 0.9WX[+] + 0.75SL		**Comb50**	1.35DL + 0.9WX[+] + 0.75SL
WX[-]	**Comb37**	1.35DL + 0.9WX[-]	WX[-]	**Comb51**	1.35DL + 0.9WX[-]
	Comb38	1.35DL + 0.9WX[-] + 0.75SL		**Comb52**	1.35DL + 0.9WX[-] + 0.75SL
WY[+]	**Comb39**	1.35DL + 0.9WY[+]	WY[+]	**Comb53**	1.35DL + 0.9WY[+]
	Comb40	1.35DL + 0.9WY[+] + 0.75SL		**Comb54**	1.35DL + 0.9WY[+] + 0.75SL
WY[-]	**Comb41**	1.35DL + 0.9WY[-]	WY[-]	**Comb55**	1.35DL + 0.9WY[-]
	Comb42	1.35DL + 0.9WY[-] + 0.75SL		**Comb56**	1.35DL + 0.9WY[-] + 0.75SL

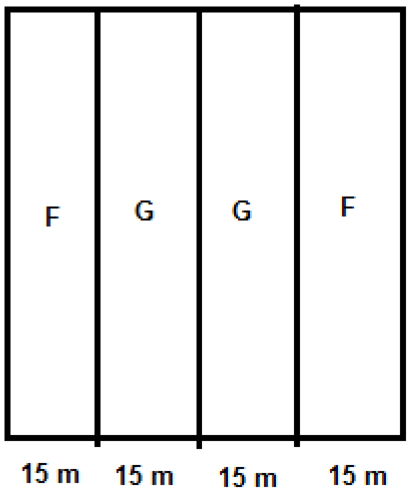
* LVA: Leading variable action

By equation 6.10(b):

LVA	DL is unfavorable		LVA	DL is unfavorable	
SL	**Comb57**	1.15DL	WX[-]	**Comb65**	1.15DL + 0.9WX[-]
	Comb58	1.15DL + 0.75SL		**Comb66**	1.15DL + 0.9WX[-] + 0.75SL
	Comb59	1.15DL + 0.75SL + 0.9 WX[+]	WY[+]	**Comb67**	1.15DL + 0.9WY[+]
	Comb60	1.15DL + 0.75SL + 0.9 WX[-]		**Comb68**	1.15DL + 0.9WY[+] + 0.75SL
	Comb61	1.15DL + 0.75SL + 0.9 WY[+]	WY[-]	**Comb69**	1.15DL + 0.9WY[-]
	Comb62	1.15DL + 0.75SL + 0.9 WY[-]		**Comb70**	1.15DL + 0.9WY[-] + 0.75SL
WX[+]	**Comb63**	1.15DL + 0.9WX[+]			
	Comb64	1.15DL + 0.9WX[+] + 0.75SL			

5. Design of Purlins

Purlins are designed according to load combinations. Active wind load system is shown below:



F: C_{pe} : -1.64 (from 0^0)

G: C_{pe} : -1.3 (from 90^0)

- Wind load:

F -> = -1.64 x 289 = - 473.96 kgf/m^2

G -> = -1.3 x 289 = - 375.7 kgf/m^2

- Covering : 30 kgf/m^2 (DL)
- Snow load : 60 kgf/m^2 (SL)

Turkish Standart is used in order to determine sandwich panel:

F Section	G Section
DL + SL = 90 kgf/m^2	DL + SL = 90 kgf/m^2
DL + WL = - 443.96 kgf/m^2	DL + WL = - 345.7 kgf/m^2
DL + SL + 0.5 WL = -148.98 kgf/m^2	DL + SL + 0.5 WL = -97.85 kgf/m^2
DL + WL + 0.5 SL = -383.96 kgf/m^2	DL + WL + 0.5 SL = -315.7 kgf/m^2
-443.96 kgf/m^2 is most critical for F section	-345.7 kgf/m^2 is most critical for G section

From the ASSAN catalogue, 1000R7 is used and span length is determined as 2.8 meter and 3.3 meter by interpolation.

2.8 meter -> 0 – 16.4 meter & 43.6 – 60 meter

3.3 meter -> 16.4 – 43.6 meter

- **Determination of Ultimate Design Moment**

Dead Load: s = 2.8 m

Covering: 0.3 kPa

HVAC: 0.2 kPa

Assume selfweight of purlin: 0.7 kN/m

d_k = 0.5 x 2.8 + 0.7 = 2.1 kN/m

Snow Load: s = 2.8 m

s_k = 0.6 x 2.8 = 1.68 kN/m

Wind Load: (F Section critical)

$P_e = q_p \times C_{pe}$ =-2.89 x 1.64 = 4.74 kPa

P_{ev} = -4.716 kPa

w_k = -4.716 x 2.8 = -13.2 kN/m

Charachteristic Moments: (L = 12 m)

Due to the dead load: $M_d = (w\, L^2)/8 = 37.8\, kN.m$

Due to the snow load: $M_s = (w\, L^2)/8 = 30.39\, kN.m$

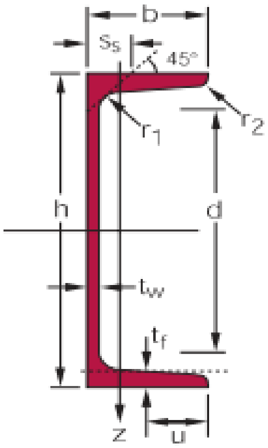
Due to the wind load: $M_w = (w\, L^2)/8 = -237.7\, kN.m$

Most critical case is 1.00DL + 1.5WL

Ultimate Design Moment: $M_u = (1.00)(37.8) + (1.5)(-237.7) = -318.75\, kN.m$

Ultimate Shear: $V_u = (1.00)(12.6) + (1.5)(-79.23) = -106.2\, kN$

Try UPN400 Section



b: 110 mm **h:** 400 mm **t_w :** 14 mm **t_f :** 18 mm **r:** 18 mm

A: 9200 mm^2 **W_{pl} :** 1240 cm^3

A_{nv} = 9200 – 2 x 110 x18+ 2 x (14 + 2 x 18) x 18 = 6140 mm^2

Classification: **Web:** (400 – 18x2) = 26 < 66.24 -> Class 1

Flange: (110-14)/18 = 5.3 < 8.28 -> Class 1

Moment Capacity: $M_{c,Rd}$ = 1240 x 1000 x 275 x 10^{-6} =341 kN.m

Shear Capacity: $V_{pl,\ Rd}$ = 6140 x 275 / $\sqrt{3}$ x 10^{-3} = 974.8 kN

Buckling Resistant Moment:

$$\mathrm{M_{cr,0}} = \sqrt{\frac{\pi^2 \mathrm{x}210\mathrm{x}846\mathrm{x}10^4}{(12000)^2}\mathrm{x}\left(81\mathrm{x}82\mathrm{x}10^4 + \frac{\pi^2\mathrm{x}210\mathrm{x}221\mathrm{x}10^9}{(12000)^2}\right)} = 92\ \mathrm{kN.m}$$

$$C_b = 1\ (L = 12\ m)\ \rightarrow\ \lambda_{LT} = \sqrt{\frac{1240\ x\ 10^3 x\ 275}{92\ x\ 10^6}} = 1.92$$

$$\emptyset_{LT} = 0.5\ x\ [\ 1 + 0.76\ (1.92 - 0.4) + 0.75\ x\ 1.92^2\ ] = 2.47$$

$$X_{LT} = 1/(2.47 + \sqrt{2.47^2 - 0.75x1.92^2} = 0.23$$

$$M_{b,Rd} = 0.23\ x\ 1240\ x\ 275\ x\ 10^{-3} = 79.4\ kN.m < \ M_u$$

Since design moment is bigger than buckling resistance moment, UPN400 is not suitable.

An excel sheet can be done in order to find appropriate purlin section.

		L_{cr}	$M_{cr,0}$	C_b	M_{cr}	landa LT	fi LT	X_{LT}	$M_{b,Rd}$	M_{Ed}
UPN 400		6000	196.3366	1.75	478.0781	0.844555	0.936408	0.657404	224.1747	318.7
		3000	478.0781	1	196.3366	1.317882	1.5001	0.40427	137.8561	
IPN 500		12000	351.3161	1	351.3161	1.592538	1.743238	0.355945	317.1471	306.4
		6000	761.5803	1.75	1332.766	0.81764	0.853022	0.752629	670.5927	
IPN 450		12000	238.0893	1	238.0893	1.664953	1.849439	0.332489	219.4429	311
		6000	510.6513	1.75	893.6398	0.859391	0.889508	0.7264	479.4237	
IPN 400		12000	154.9187	1	154.9187	1.744295	1.970314	0.30909	145.6893	315
		6000	328.8021	1.75	575.4037	0.905077	0.93093	0.697747	328.8832	
		4000	537.2782	1	537.2782	0.936639	0.960461	0.678071	319.6086	

IPN400 is choosed with 4 meter lateral supports

Moment aspect: $M_{b,Rd}$ = 319.6 kN.m > M_{Ed} = 315

Shear aspect: A_{nv} **=** 11800 – 2 x 155 x21.6+ 2 x (14.4 + 2 x 14.4) x 21.6 = 6970 mm^2

$V_{pl,\,Rd}$ = 6970 x 275 / $\sqrt{3}$ x 10^{-3} = 1106.67 kN > 106.2 kN

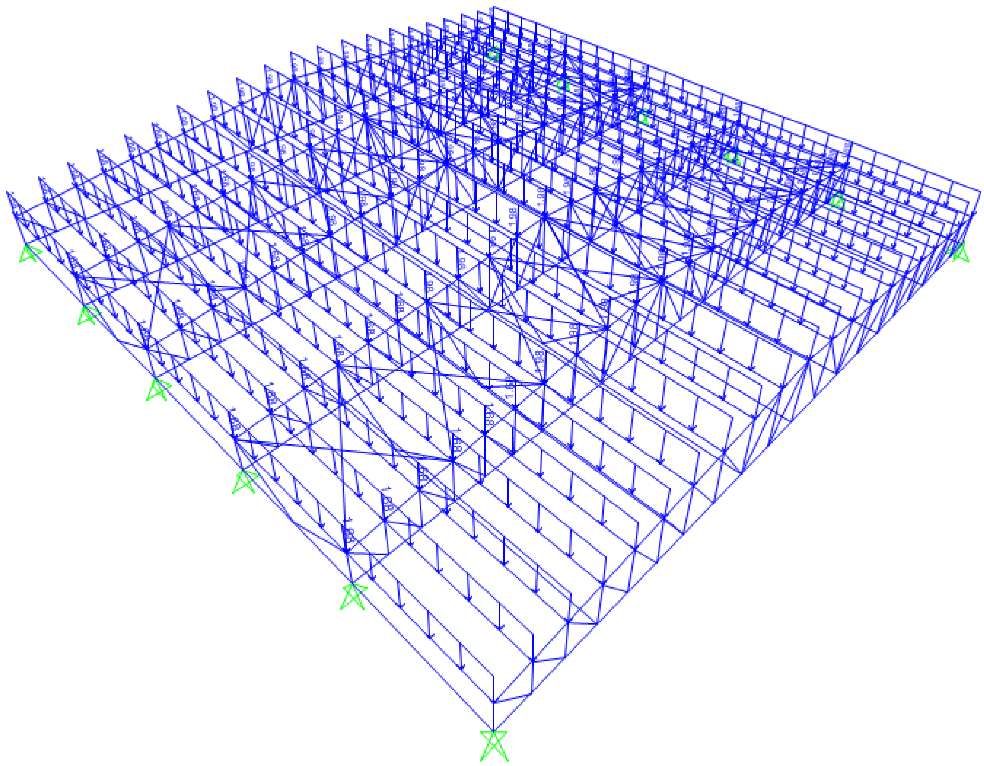
IPN 400 is sutiable with respect to moment and shear design

6. Design of Roof

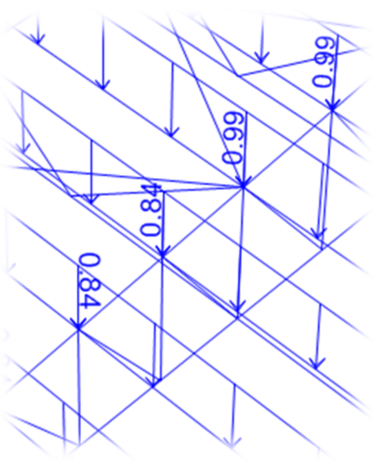
In order to determine sections, all forces are applied to the purlins at the SAP model and analyzed with respect to load combinations. All load combinations can be gathered into the envelope. Therefore, the worst cases of all load combinations can be determined due to load combinations in the envelope.

The most critical truss section is chosen and section axial forces is taken from this critical case and the roof is designed with respect to these critical cases.

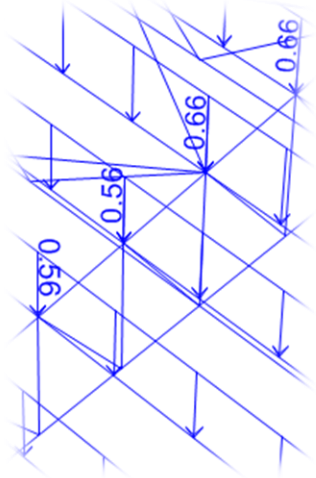
Dead load and snow load acting on the roof



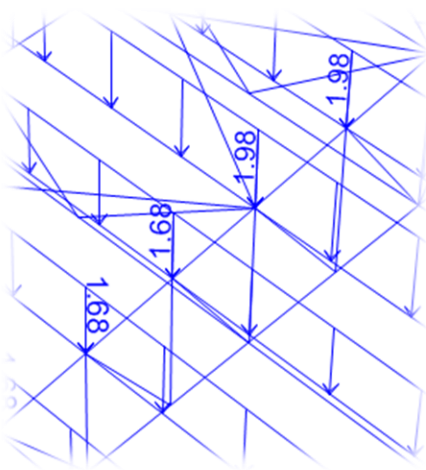
There are two different distributed load acting on the purlin which are for 2.8 meter and 3.3 meter span lengths. (for dead load and snow load)



DL1 (Covering)



DL2 (HVAC)



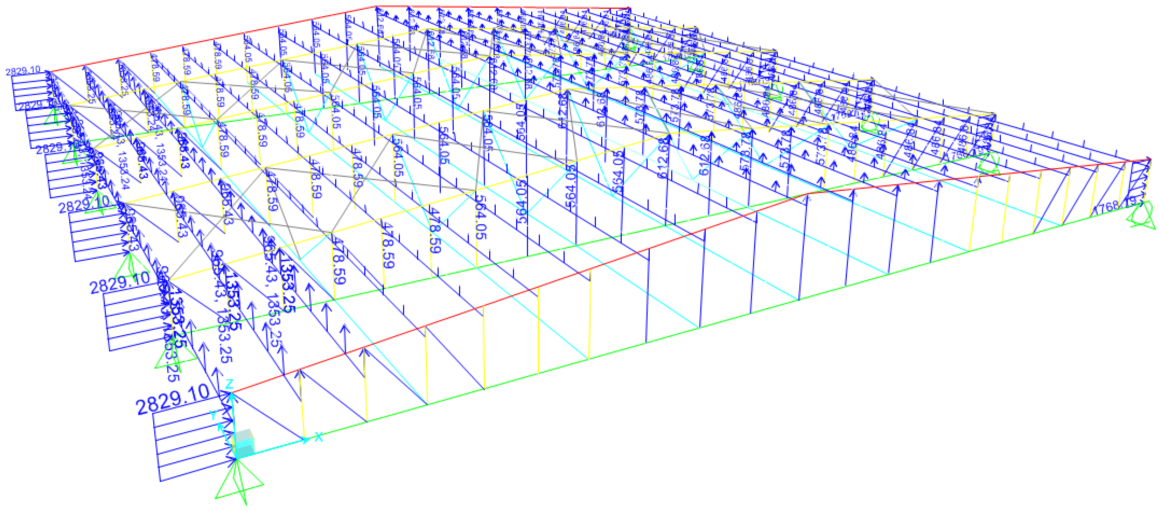
SL (Snow load)

= 0.3 x 2.8 = 0.84 kN/m   = 0.2 x 2.8 = 0.56 kN/m   = 0.6 x 2.8 = 1.68 kN/m

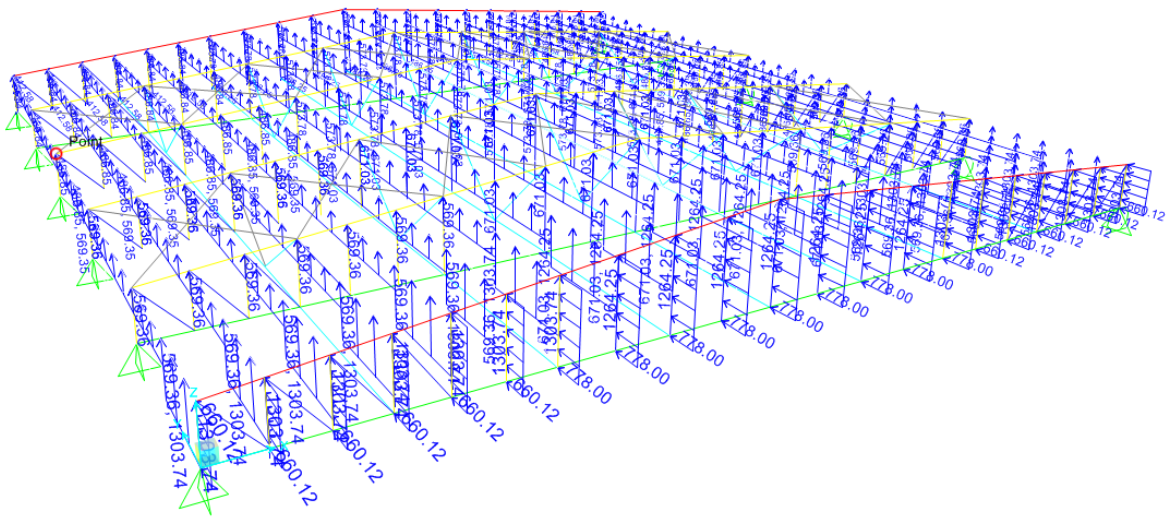
= 0.3 x 3.3 = 0.99 kN/m   = 0.2 x 3.3 = 0.66 kN/m   = 0.6 x 3.3 = 1.98 kN/m

Wind load acting on the roof

There are four different direction for vertical wind load acting on the roof with respect to +x,-x,+y,-y direction. There is also horizontal wind direction acting on the roof in +x,-x,+y,-y direction. The critical point is arranging distributed load according to F,G,H,J,I sections.



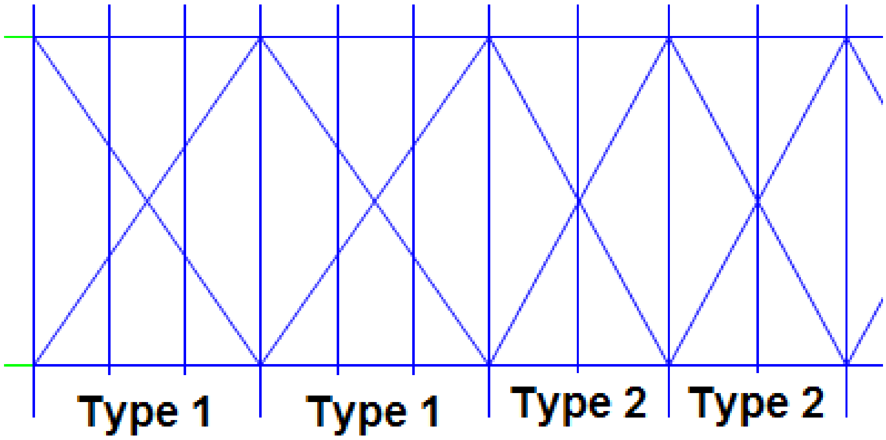
Wx+ direction wind load



Wy+ direction wind load

6.1. Design of Brace System

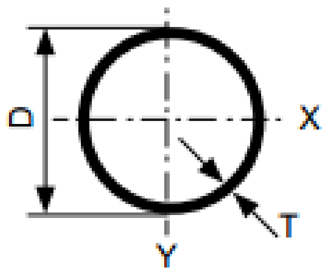
Two different type of steel section is designed for brace system. First one is 14.7 meter for 2.8x3 = 8.4 meter span length. Second one is 13.7 meter for 3.3 x 2 = 6.6 meter span length.



For type 1:

$Most\ dritical\ N_{ed} = 206\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 302\ kN\ for\ tension$



$\boldsymbol{For\ \ CHCF\ 244.5\ x\ 10\ section}$

Classification:

$$\frac{d}{t} = \frac{244.5}{10} = 24.45 < 50\varepsilon^2 = 50x\ 0.92^2 = 42.32 \rightarrow class\ 1$$

$$\lambda_1 = 93.9\ x\ \sqrt{\frac{235}{275}} = 86.4\ \rightarrow\ \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{14647}{83} x \frac{1}{86.4} = 2.03$$

$\phi = 0.5x(1 + 0.21x(2.03 - 0.2) + 2.03^2) = 2.76$ where hot rolled section a = 0.2

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{2.76 + \sqrt{2.76^2 - 2.03^2}} = 0.216$$

$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.216\ x\ 7367\ x\ 275}{1} = 438\ kN \rightarrow\ \boldsymbol{N_{b,Rd} > N_{ed} = 206\ kN}$$

$$N_{pl,Rd} = \frac{A\,x\,f_y}{\gamma_{m0}} = \frac{7367x275}{1}x10^{-3} = \mathbf{2025\ kN} > \boldsymbol{N_{ed}} = \mathbf{302}$$

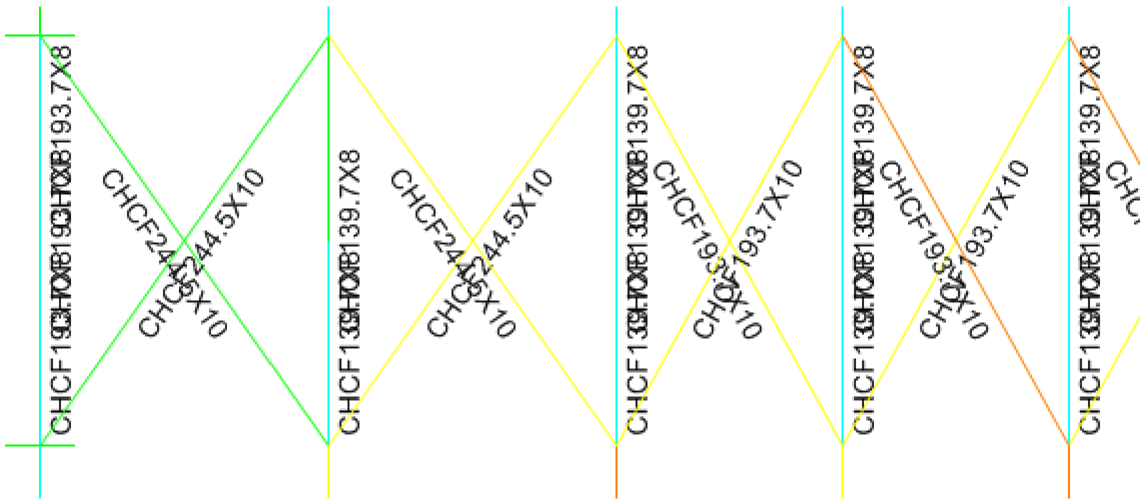
For second section, the same process is applied and the section is determined as CHCF193.7x10.

$$Most\ critical\ N_{ed} = 140\ kN\ for\ compression$$
$$Most\ critical\ N_{ed} = 175\ kN\ for\ compression$$

The sections are tabulated below:

Type	D	T	Area	D/T	Classification	i	λ	ϕ	X	$N_{b,Rd}$
CHCF 244.5 x 10	244.5	10	7367	24.45	Class 1	83	2.03312	2.759267	0.216229	438.0628
CHCF 193.7 x 10	193.7	10	5771	19.37	Class 1	65	2.596138	4.121562	0.136561	216.7265

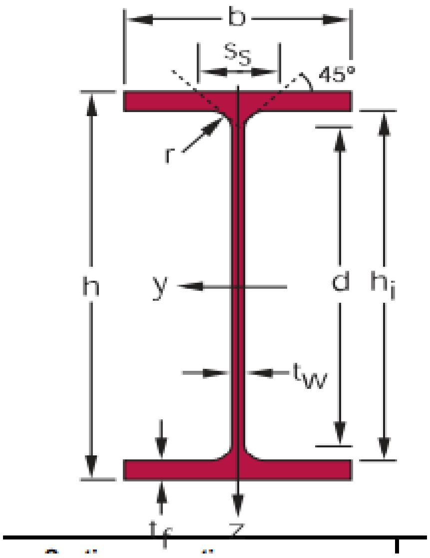
After the new sections are applied into the model and after the results of analyzing, sections are suitable under the most critical load combination. (Envelope)



Horizontal wind bracing system is also designed as CHCF193.7x8 and CHCF139.7x8 respectively and again there is no problematic situations faced at the SAP model.

6.2. Design of Top Chords

HE320A section is designed for top chord:



b = 300 mm

h = 310 mm

t_w = 9 mm

t_f = 15.5 mm

r = 27 mm

d = 225 m

i_z = 74.9 mm

A = 12440 mm²

Classification:

$\boldsymbol{Web}$: d/t = 225/9 = 25 < 43ε = 66.24 → Class 1

$\boldsymbol{Flange}$: $\frac{(300-2x27-9)/2}{15.5} = 7.64 < 8.28 \rightarrow$ Class 1

h/b = 310/300 = 1.03 <1.2

t_f = 15.5 mm < 100 mm

Buckling curve 'b' -> 0.34

$Most\ dritical\ N_{ed} = 2476\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 1703\ kN\ for\ tension$ is obtained from SAP Analysis

Compression check:

More critical section: Length = 3 / cos(5.71^0) = 3.32 meter

$$\lambda_1 = 93.9\ x\ \sqrt{\frac{235}{275}} = 86.4 \rightarrow \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{3320}{74.9} x \frac{1}{86.4} = 0.51$$

$$\phi = 0.5x(1 + 0.34x(0.51 - 0.2) + 0.51^2) = 0.684$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{0.684 + \sqrt{0.684^2 - 0.51^2}} = 0.875$$

$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.875\ x\ 12440\ x\ 275}{1} = 2996\ kN \rightarrow \boldsymbol{N_{b,Rd} > N_{ed}}$$

Tension check:

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{12440x275}{1} x10^{-3} = \boldsymbol{3421\ kN} \rightarrow \boldsymbol{N_{pl,Rd} > N_{ed}}$$

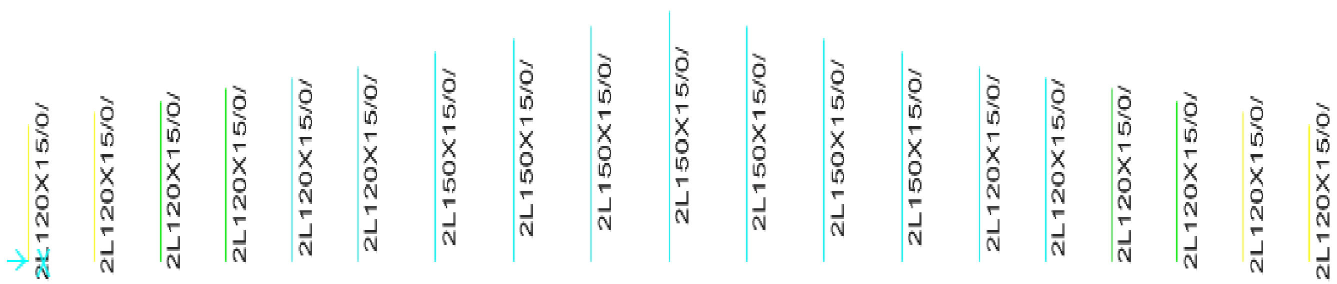
6.3. Design of Vertical Bracing Member

Two different vertical bracing member is designed with respect to considering N_{Ed}. Both first 6 and last 6 vertical bracing member and 7 vertical member at the middle is designed as 2L120x15 and 2L150x15 respectively.

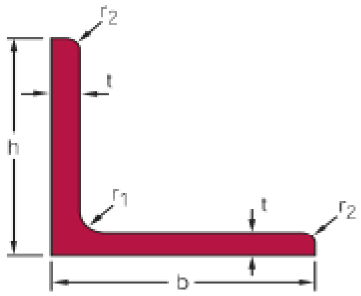
y = 0 and y = 60 meter sections are more critical due to the horizontal wind loads.

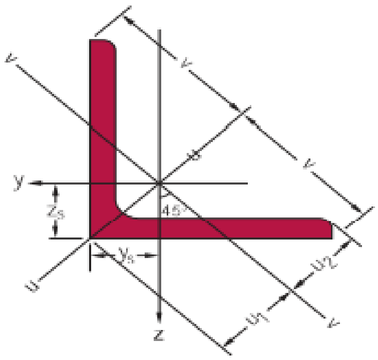
1-1’ & 6-6’ section

2-2’ section



2L 120x120x15 section is designed for first 6 and last 6 member without main columns





b = 120 mm

h = 120 mm

t_w = 15 mm

t_f = 15 mm

$i_{y=z}$= 36.2 mm

A = 3390 mm^2

Classification:

$$\frac{\mathrm{h}}{\mathrm{t}} = \frac{120}{15} < 14\varepsilon = 12.88$$

$$\frac{\mathrm{b+h}}{2(\mathrm{t})} = \frac{120+120}{2\mathrm{x}15} = 8 < 11.5\varepsilon = 10.58$$

Therefore, Class 3

Since equal leg section is used,

buckling curve is 'b'

$Most\ dritical\ N_{ed} = 817\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 659\ kN\ for\ tension$ is obtained from SAP Analysis

<u>Compression check:</u>

Critical section: Maximum compression is occurred at first and last vertical member. (L = 3.28 m)

$$\lambda_1 = 93.9\ x \sqrt{\frac{235}{275}} = 86.4 \ \rightarrow \ \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{3280}{36.2} x \frac{1}{86.4} = 1.05$$

$$\phi = 0.5x(1 + 0.34x(1.05 - 0.2) + 1.05^2) = 1.19$$

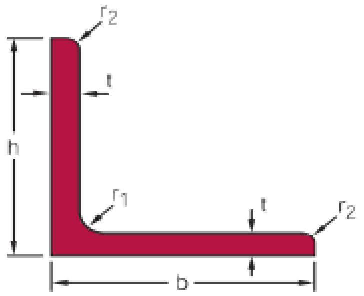
$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{1.19 + \sqrt{1.19^2 - 1.05^2}} = 0.567$$

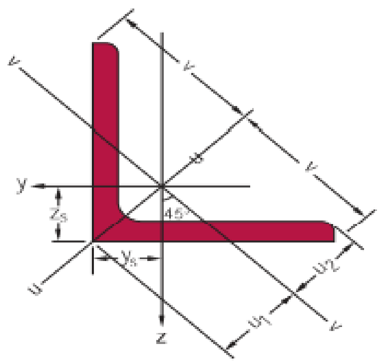
$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.567\ x\ 3390\ x\ 275x2}{1} x10^{-3} = 1058\ kN \rightarrow \boldsymbol{N_{b,Rd} > N_{ed}}$$

<u>Tension check:</u>

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{3390\ x\ 2\ x275}{1} x10^{-3} = 1864.5\ kN \rightarrow \boldsymbol{N_{pl,Rd} > N_{ed}}$$

2L 150x150x15 section is designed for first 6 and last 6 member





b = 150 mm

h = 150 mm

t_w = 15 mm

t_f = 15 mm

$i_{y=z}$= 46 mm

A = 4300 mm^2

Classification:

$$\frac{h}{t} = \frac{150}{15} < 14\varepsilon = 12.88$$

$$\frac{b+h}{2(t)} = \frac{150+150}{2x15} = 10 < 11.5\varepsilon = 10.58$$

Therefore, Class 3

Since equal leg section is used,

buckling curve is 'b'

back to back distance = 0

$Most\ dritical\ N_{ed} = 100\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 117.4\ kN\ for\ tension$ is obtained from SAP Analysis

Compression check:

Critical section: Maximum compression is occurred at vertical bracing member having L = 5 m

$$\lambda_1 = 93.9\ x\ \sqrt{\frac{235}{275}} = 86.4 \rightarrow \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{5000}{46} x \frac{1}{86.4} = 1.26$$

$$\phi = 0.5x(1 + 0.34x(1.26 - 0.2) + 1.26^2) = 1.47$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{1.47 + \sqrt{1.47^2 - 1.26^2}} = 0.448$$

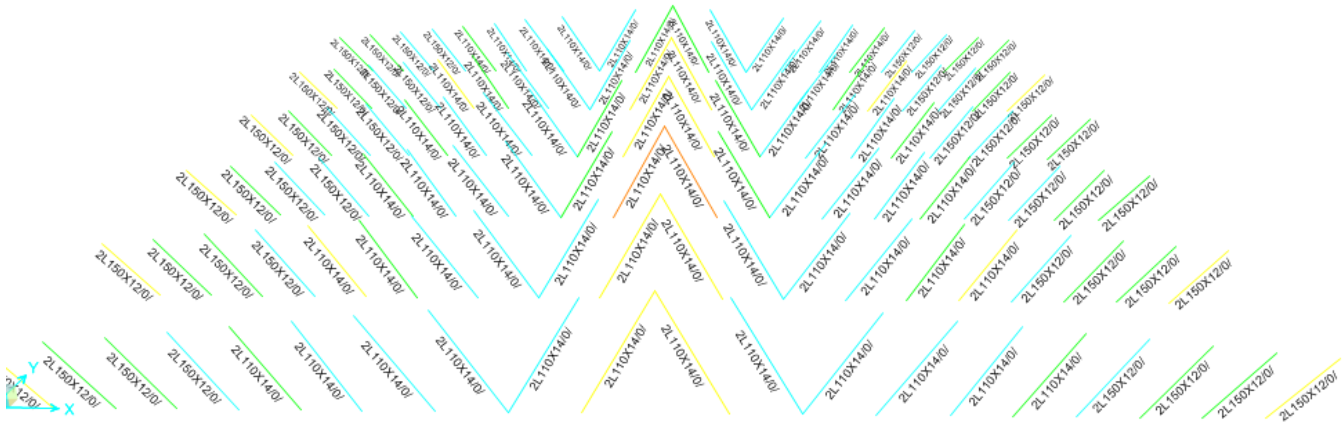
$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.448\ x\ 4300\ x\ 275x2}{1} x10^{-3} = 1060\ kN \rightarrow \boldsymbol{N_{b,Rd} > N_{ed}}$$

Tension check:

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{4300\ x\ 2\ x275}{1} x10^{-3} = 2365\ kN \rightarrow \boldsymbol{N_{pl,Rd} > N_{ed}}$$

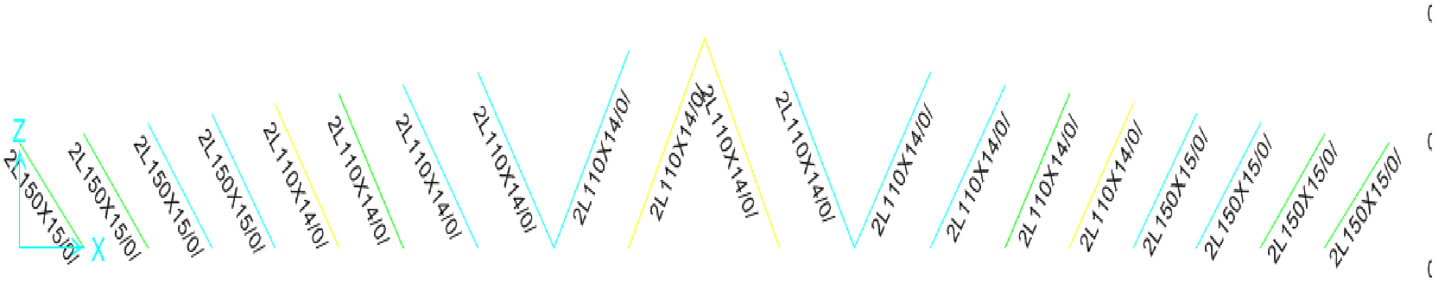
6.4. Design of Diagonal Members of Truss

There are two type of diagonal members designed.

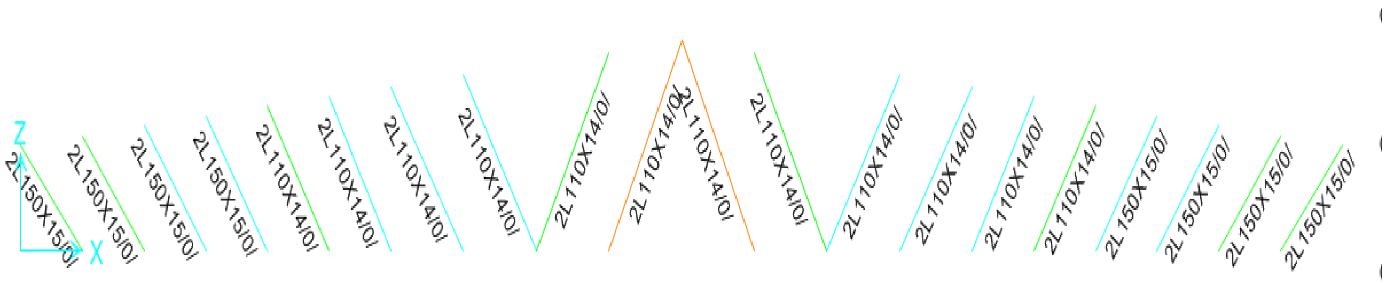


Both first 4 and last 4 diagonal member of truss and rest of the diagonals are designed as 2L 150x15 and 2L 110x14 having no back to back distance.

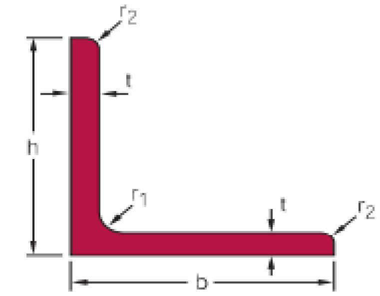
1-1' section

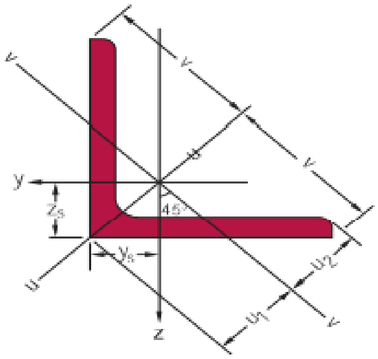


2-2' section



2L 150x150x15 section is designed for first 6 and last 6 member without main columns





b = 150 mm

h = 150 mm

t_w = 15 mm

t_f = 15 mm

$i_{y=z}$= 46 mm

A = 4300 mm^2

Classification:

$$\frac{\mathrm{h}}{\mathrm{t}} = \frac{150}{12} = 12.5 < 14\varepsilon = 12.88$$

$$\frac{\mathrm{b}+\mathrm{h}}{2(\mathrm{t})} = \frac{150+150}{2\mathrm{x}15} = 10 < 11.5\varepsilon = 10.58$$

Therefore, Class 3

Since equal leg section is used,

buckling curve is 'b'

$Most\ dritical\ N_{ed} = 889\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 1103\ kN\ for\ tension$ is obtained from SAP Analysis

Compression check:

Critical section: Maximum compression is occurred at first and last diagonal member. (L = 4.1 m)

$$\lambda_1 = 93.9\ x\ \sqrt{\frac{235}{275}} = 86.4\ \rightarrow\ \bar{\lambda} = \frac{L_{cr}}{i}\,x\,\frac{1}{\lambda_1} = \frac{4100}{46}\,x\,\frac{1}{86.4} = 1.03$$

$$\phi = 0.5x(1 + 0.34x(1.03 - 0.2) + 1.03^2) = 1.17$$

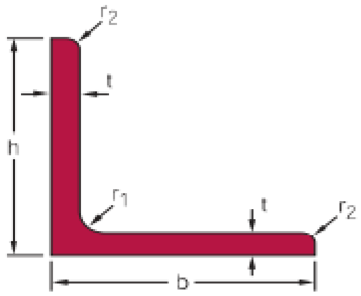
$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{1.17 + \sqrt{1.17^2 - 1.03^2}} = 0.576$$

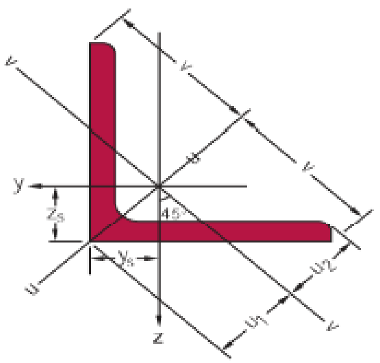
$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.576\ x\ 4300\ x\ 275x2}{1}x10^{-3} = 1362\ kN \rightarrow\ \boldsymbol{N_{b,Rd} > N_{ed}}$$

Tension check:

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{4300\ x\ 2\ x275}{1}x10^{-3} = 2365\ kN \rightarrow\ \boldsymbol{N_{pl,Rd} > N_{ed}}$$

2L 110x110x14 section is designed for middle 7 section





b = 110 mm

h = 110 mm

t_w = 14 mm

t_f = 14 mm

$i_{y=z}$= 33.1 mm

A = 2884 mm^2

Classification:

$$\frac{h}{t} = \frac{110}{14} < 14\varepsilon = 12.88$$

$$\frac{b+h}{2(t)} = \frac{110+110}{2x14} = 7.85 < 11.5\varepsilon = 10.58$$

Therefore, Class 3

Since equal leg section is used,

buckling curve is 'b'

back to back distance = 0

$Most\ dritical\ N_{ed} = 193\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 203\ kN\ for\ tension$ is obtained from SAP Analysis

Compression check:

Critical section: Maximum compression is occurred at diagonal bracing member having L =6.85 m

$$\lambda_1 = 93.9\ x\ \sqrt{\frac{235}{275}} = 86.4 \rightarrow \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{6850}{33.1} x \frac{1}{86.4} = 2.4$$

$$\phi = 0.5x(1 + 0.34x(2.4 - 0.2) + 2.4^2) = 3.6$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{3.6 + \sqrt{3.6^2 - 2.4^2}} = 0.16$$

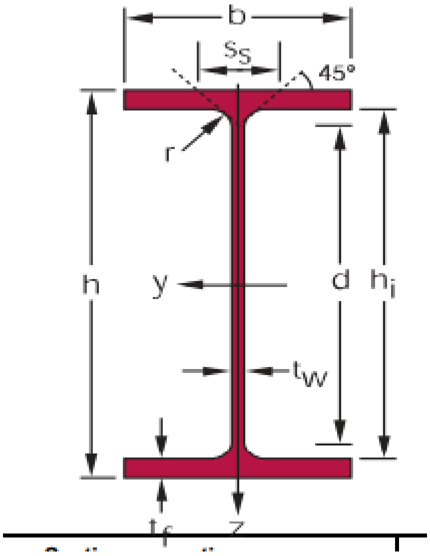
$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.16\ x\ 2884\ x\ 275x2}{1} x10^{-3} = 252\ kN \rightarrow \boldsymbol{N_{b,Rd} > N_{ed}}$$

Tension check:

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{2884\ x\ 2\ x275}{1} x10^{-3} = 1586.2\ kN \rightarrow \boldsymbol{N_{pl,Rd} > N_{ed}}$$

6.5. Design of Bottom Chord

HE300A section is designed for top chord:



b = 300 mm

h = 290 mm

t_w = 8.5 mm

t_f = 14 mm

r = 27 mm

d = 208 mm

i_z = 74.9 mm

A = 11250 mm^2

Classification:

$\boldsymbol{Web}$: $d/t = 208/8.5 = 24.5 < 43\varepsilon = 66.24 \rightarrow$ Class 1

$\boldsymbol{Flange}$: $\frac{(300-2x27-8.5)/2}{14} = 8.48 < 9.2 \rightarrow$ Class 2

h/b = 290/300 = 0.97 <1.2

t_f = 14 mm < 100 mm

Buckling curve 'b' -> 0.34

$Most\ dritical\ N_{ed} = 2014\ kN\ for\ compression$

$Most\ critical\ N_{ed} = 2621\ kN\ for\ tension$ is obtained from SAP Analysis

Compression check:

More critical section: Length = 3.3 meter

$$\lambda_1 = 93.9\ x \sqrt{\frac{235}{275}} = 86.4 \ \rightarrow \ \bar{\lambda} = \frac{L_{cr}}{i} x \frac{1}{\lambda_1} = \frac{3300}{74.9} x \frac{1}{86.4} = 0.51$$

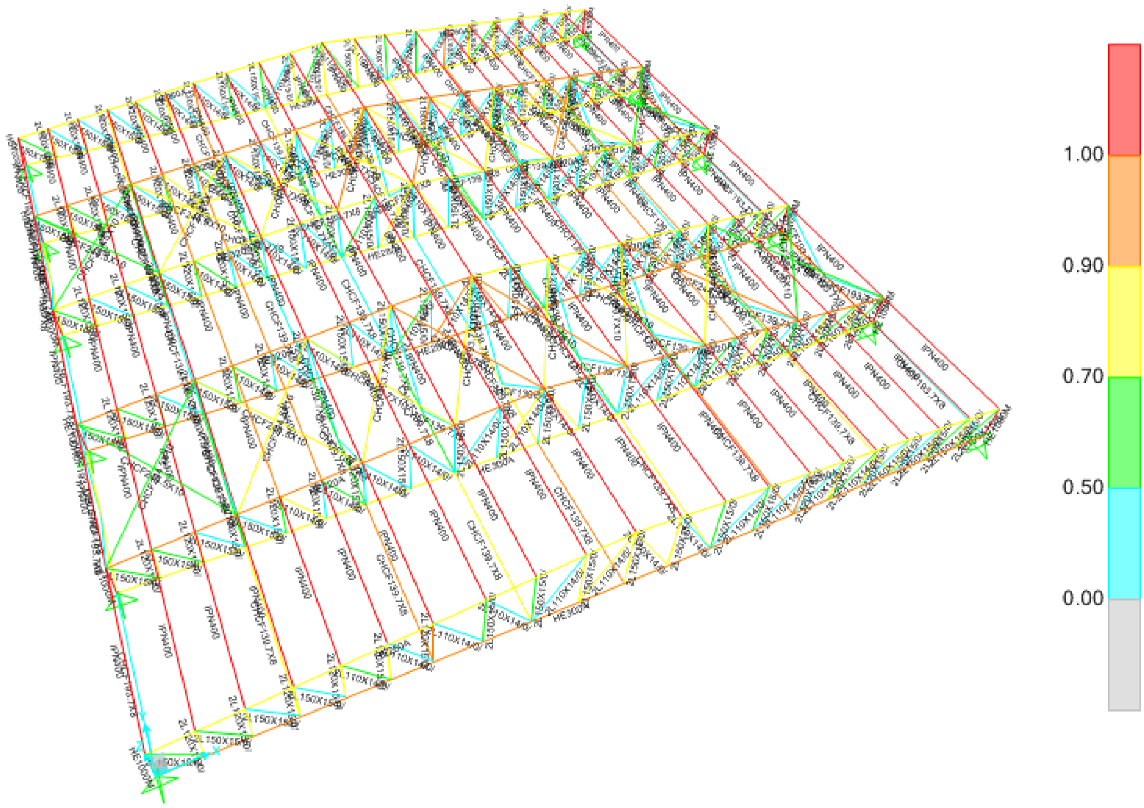
$$\phi = 0.5x(1 + 0.34x(0.51 - 0.2) + 0.51^2) = 0.684$$

$$X = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}} = \frac{1}{0.684 + \sqrt{0.684^2 - 0.51^2}} = 0.875$$

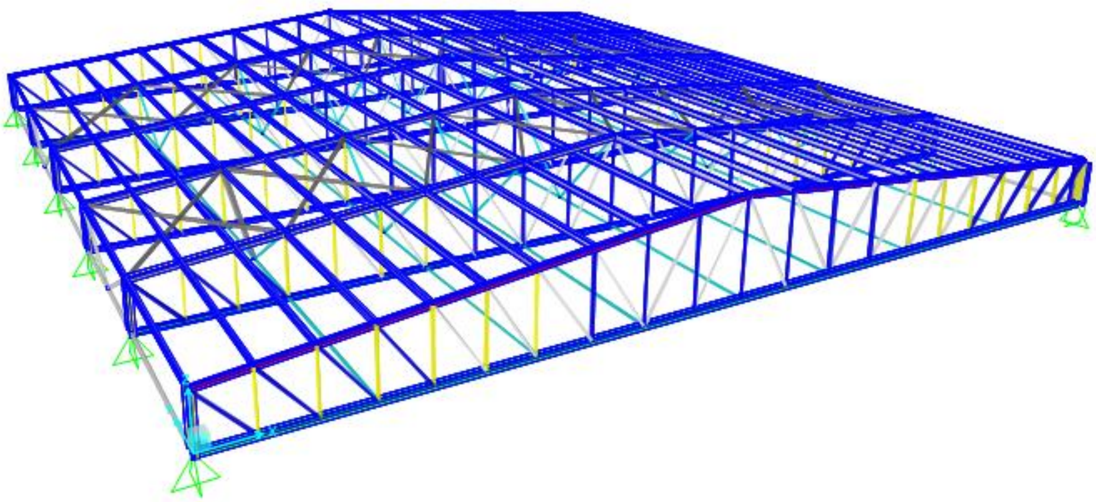
$$N_{b,Rd} = \frac{X\ x\ A\ x\ f_y}{\gamma_{m1}} = \frac{0.875\ x\ 11250\ x\ 275}{1} = 2707\ kN \rightarrow \boldsymbol{N_{b,Rd} > N_{ed}}$$

Tension check:

$$N_{pl,Rd} = \frac{A\ x\ f_y}{\gamma_{m0}} = \frac{11250x275}{1} x10^{-3} = \boldsymbol{3093\ kN} \rightarrow \boldsymbol{N_{pl,Rd} > N_{ed}}$$

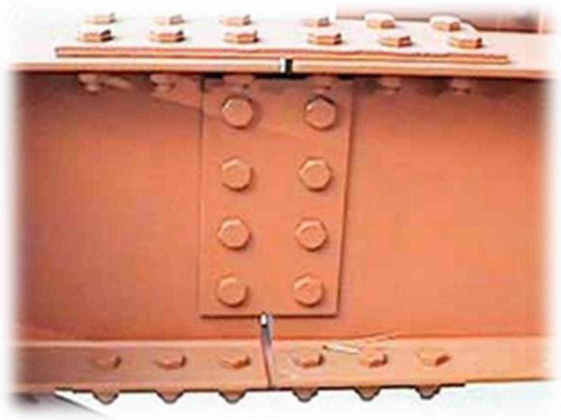


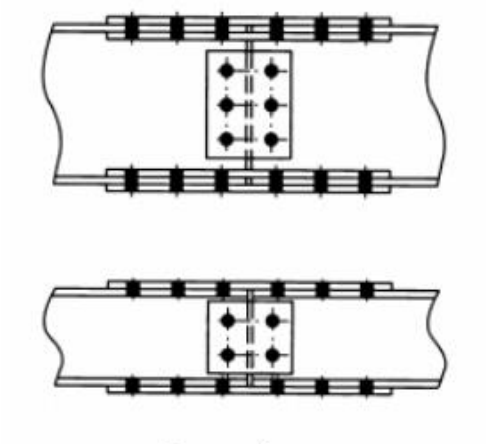
Purlins are not seem to be suitable since there is not any bridging leading to lateral support at purlin section at the SAP model. In addition, although when main column of truss which is HE1000M is not problematic by hand calculation (also overdesign), SAP indicates these columns as failure. This situation is ignored and hand calculation is used as HE320A for main columns.



6.6. Design of Connections

- **Splice Plate Connection**





Beam splices

Material data:

- Cover-plates grade S275 to EN 10025-2.
- Steel grade S275
- Yield strength f_y = 275 N/mm2
- Ultimate tensile strength f_u = 430 N/mm2

Bolted connections data

- Category of bolted connections Category C
- Bolt Class 10.9
- Yield strength f_{yb} = 900 N/mm2
- Ultimate tensile strength f_{ub} = 1000 N/mm2

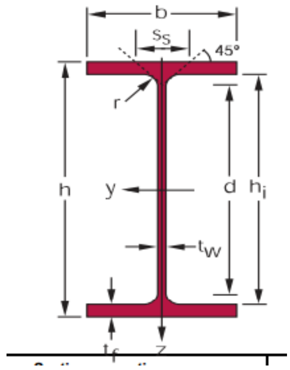
For flanges cover plates

- Nominal bolt diameter d_f = 30 mm
- Hole diameter $d_{0,f}$ = 33 mm

For web cover plates

- Nominal bolt diameter d_w = 18 mm
- Hole diameter $d_{0,w}$ = 20 mm

I Section Properties (HEA300)



b = 300 mm,

h = 290 mm

t_w = 8.5 mm

t_f = 14 mm

r = 27 mm

d = 208 mm

i_z = 74.9 mm

A = 11250 mm^2 , I_y =18260 cm^4 , W_y = 1260 cm^3

The bending moment and shear can be ignored due to the very low results. Only axial force is considered during design.

$N_{ed} = 2621\ kN\ \ V_{Ed} = 11\ kN$

Design of Web Connection

Category C connection -> the design tension resistance is:

$$N_{t,Rd} = N_{net,Rd} = \frac{A_{net}\ x\ f_y}{\gamma_{M0}}$$

For the cross section, we consider 8 fastener holes for fasteners (4 by flange, 2 for the web)

$$A_{net} = 11250 - 4x33x14 - 2x20x8.5 = 9062\ mm^2$$

$$N_{t,Rd} = N_{net,Rd} = \frac{9062\ x\ 275}{1} x10^{-3} = 2492\ kN$$

The axial force distribuition between web and flange is mainly based on the ratio of the web and flange cross section.

$$A_w = (h - 2t_f)t_w = (290 - 2x14)x8.5 = 2227\ mm^2 \rightarrow\ N_{N,w} = \frac{N_{Ed}A_w}{A} = \frac{2621x2227}{11250} = 518.8\ kN$$

$$A_f = \frac{(A - A_w)}{2} = (11250 - 2227)/2 = 4511.5\ mm^2 \rightarrow N_{N,f} = \frac{N_{Ed} - N_{N,w}}{2} = \frac{(2621 - 518.8)}{2} = 1051\ kN$$

Design Shear Force, $F_{v,Ed}$

For the component web

$F_{V,Ed,w}$ = $N_w/8 = 518.8/8=64.85$ kN

For each component plate

$F_{V,Ed,p}$ = ($N_w/2)/8 = 32.4$ kN

Design Slip Resistance $F_{s,Rd}$

Assumptions:

- Bolts in normal holes -> $k_s = 1.0$
- Class friction surfaces = Class A -> $\mu = 0.5$

$A_{s,w} = 192\ mm^2 \rightarrow\ F_{p,c} = 0.7\ x\ f_{ub}\ x\ A_{s,w} = 0.7x192x1000 = 134.4\ kN$

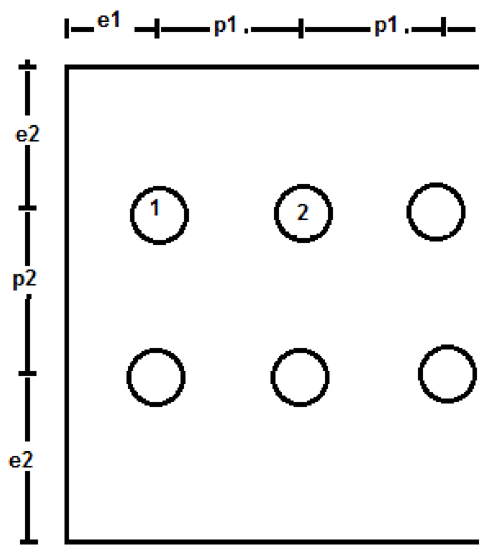
$n = number\ of\ the\ friction\ surface \rightarrow\ n_w = 2\ for\ web,\ n_p = 1\ for\ plate\ component$

$$F_{s,Rd,w} = \frac{k_s n_w \mu}{\gamma_{M3}} F_{p,c} = \frac{1x2x0.5}{1.25} x134.4 = 107.52\ kN$$

$$F_{s,Rd,p} = \frac{k_s n_p \mu}{\gamma_{M3}} F_{p,c} = \frac{1x2x0.5}{1.25} x134.4 = 53.76\ kN$$

Design Bearing Resistance

Plate Component (Web)



e_1	90 mm
p_1	60 mm
e_2	90 mm
p_2	60 mm

Max p_1=max p_2 = 14 x t =59.5=60 mm

For bolts

$$\propto_{b\ edge} = \min\left(\frac{90}{60}\ ; 1\right) = 1$$

$$k_{1\ edge} = \min\left(1.4\ x \frac{60}{20} - 1.7;\ \ 2.8\ x\ \frac{90}{20} - 1.7; 2.5\right)$$

$$k_{1\ edge} = 2.5$$

$$\propto_{b\ inner} = \min\left(\frac{60}{60} - 0.25\ ; 1\right) = 0.75$$

$$k_{1\ inner} = \min\left(1.4\ x \frac{60}{20} - 1.7;\ 2.5\right) = 2.5$$

For end bolts $\alpha_{b,end} = \min\left\{\frac{e_1}{3d_0}; \frac{f_{ub}}{f_u}; 1,0\right\}$

$$k_{1,end} = \min\left\{1,4\frac{p_2}{d_0} - 1,7; 2,8\frac{e_2}{d_0} - 1,7; 2,5\right\}$$

For inner bolts $\alpha_{b,inner} = \min\left\{\frac{p_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1,0\right\}$

$$k_{1,inner} = \min\left\{1,4\frac{p_2}{d_0} - 1,7; 2,5\right\}$$

$k_{1\,edge} = k_{1\,inner} = 2.5$

$\propto_{b\,inner} = 1$

$\propto_{b\,edge} = 1$

$$for\ bolt\ 1 \rightarrow F_{b,Rd,1} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = 2x\frac{2.5x1x430x20x8}{1.25} = 275.2\ kN$$

$$for\ bolt\ 2 \rightarrow F_{b,Rd,2} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = 2x\frac{2.5x0.75x430x20x8}{1.25} = 206.4\ kN$$

$F_{b,Rd,p} = 206.4\ kN$

Web Component

e_1	90 mm
p_1	60 mm
e_2	115 mm
p_2	60 mm

For bolts:

$$\propto_{b\,edge} = \min\left(\frac{115}{60}\,;1\right) = 1$$

$$k_{1\,edge} = \min\left(1.4\,x\frac{60}{20} - 1.7;\ \ 2.8\,x\,\frac{115}{20} - 1.7; 2.5\right)$$

$$k_{1\,edge} = 2.5$$

$$\propto_{b\,inner} = \min\left(\frac{60}{60} - 0.25\,;1\right) = 0.75$$

$$k_{1\,inner} = \min\left(1.4\,x\frac{120}{20} - 1.7;\ 2.5\right) = 2.5$$

$$for\ bolt\ 1 \rightarrow F_{b,Rd,1} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = \frac{2.5x1x430x20x8.5}{1.25} = 146.2\ kN$$

$$for\ bolt\ 2 \rightarrow F_{b,Rd,2} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = \frac{2.5x0.75x430x20x8.5}{1.25} = 110\ kN$$

$F_{b,Rd,w} = 110\ kN$

Checking bolts:

For web

- Design bearing resistance: $F_{v,Ed,w} = 86.47\ kN < F_{b,Rd,w} = 110\ kN$
- Design slip resistance: $F_{v,Ed,w} = 86.47\ kN < F_{s,Rd,w} = 107.52\ kN$

Group of fasteners (considering shear plane does not pass through the threaded portion)

$$F_{v,Rd} = \frac{\propto_v\ x f_{ub} x A}{\gamma_{M2}} = \frac{0.6x1000x254.47}{1.25} = 122.15\ kN$$

$F_{v,Rd} < F_{b,Rd,w}$ -> Shear governs.

$\boldsymbol{F_{gr,b,Rd,w} = 8\ x\ 122.15 = 977.2\ kN > \ N_{N,w} = 518.8\ kN}$

For plate

- Design bearing resistance: $F_{v,Ed,p} = 32.4\ kN < F_{b,Rd,p} = 206.4\ kN$
- Design slip resistance: $F_{v,Ed,p} = 32.4\ kN < F_{s,Rd,p} = 53.76\ kN$

Design of net cross section

Web component

- Net cross section: $A_{w,net} = A_w - 3d_{0,w}t_w = 2227 - 3x20x8.5 = 1717\ mm^2$
- Design resistance: $N_{w,net,Rd} = \frac{A_{w,net}\ x\ f_y}{\gamma_{M0}} = \frac{1717\ x\ 275}{1} = 472.17\ kN$
- $N_{w,net,Rd} = 472.17\ kN > \sum F_{v,Ed,w} = 2\ x\ 64.85 = 129.7\ kN$

Plate Component

- Net cross section: $A_{p,net} = A_p - 3d_{0,w}t_p = 1920 - 3x20x8 = 1440\ mm^2$
- Design resistance: $N_{p,net,Rd} = \frac{A_{p,net}\ x\ f_y}{\gamma_{M0}} = \frac{1440\ x\ 275}{1} = 396\ kN$
- $N_{p,net,Rd} = 396\ kN > \sum F_{v,Ed,w} = 2\ x\ 32.4 = 64.85\ kN$

Block Tearing:

- Web component:

$$A_{nt} = (2xp_2 - 2xd_0)xt_w = (2x60 - 2x33)x14 = 680\ mm^2$$

$$A_{nv} = 2x(e_1 + p_1 - 1.5\ d_0)xt_w = 2x(90 + 60 - 1.5x20)x8.5 = 2040\ mm^2$$

$$V_{eff,1,Rd} = \frac{f_u\ x\ A_{nt}}{\gamma_{M2}} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{f_u\ x\ A_{nv}}{\gamma_{M0}}\right) = \frac{430\ x680}{1.25} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{430\ x\ 2040}{1.0}\right) = 740\ kN$$

$$V_{eff,1,Rd} = 740\ kN > N_{N,w} = 518.8$$

- Plate component:

$$A_{nt} = (2xe_2 - 2xd_0)xt_p = (2x90 - 2x20)x8 = 1120\ mm^2$$

$$A_{nv} = 2x(e_1 + p_1 - 1.5\ d_0)xt_p = 2x(90 + 60 - 1.5x20)x8 = 1920\ mm^2$$

$$V_{eff,1,Rd} = \frac{f_u\ x\ A_{nt}}{\gamma_{M2}} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{f_u\ x\ A_{nv}}{\gamma_{M0}}\right) = \frac{430\ x1120}{1.25} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{430\ x1920}{1.0}\right) = 862\ kN$$

$$V_{eff,1,Rd} = 862\ kN > \frac{N_{N,w}}{2} = 259.4kN$$

Design of Flange Connection

$$N_{N,f} = 1051\ kN \rightarrow F_{N,bi,h} = \frac{1051}{8} = 131.38\ kN$$

Design Slip Resistance $F_{s,Rd}$

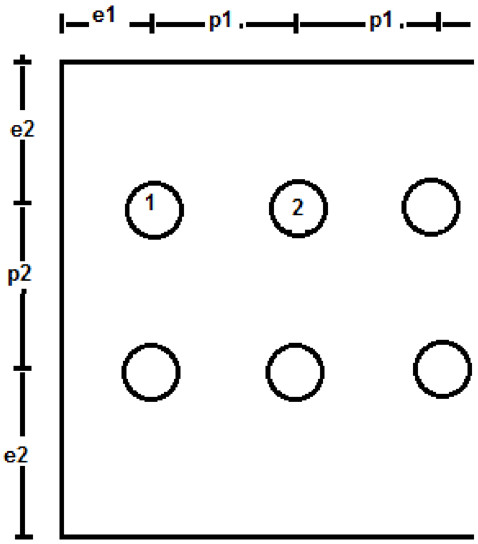
Assumptions:

- Bolts in normal holes -> $k_s = 1.0$
- Class friction surfaces = Class A -> µ = 0.5

$$A_{s,f} = 303\ mm^2 \rightarrow\ F_{p,c} = 0.7\ x\ f_{ub}\ x\ A_{s,f} = 0.7x561x1000 = 392.7\ kN$$
$$n = number\ of\ the\ friction\ surface \rightarrow\ n_w = 1\ for\ flange,\ n_p = 1\ for\ plate\ component$$
$$F_{s,Rd,w} = F_{s,Rd,p} = \frac{k_s n_f \mu}{\gamma_{M3}} F_{p,c} = \frac{1x1x0.5}{1.25} x392.7 = 157.1\ kN$$

Design Bearing Resistance

Plate Component



e_1	90 mm
p_1	100 mm
e_2	90 mm
p_2	100 mm

Plate of the flange is designed as same as the the plate of the web. Therefore;

$$for\ bolt\ 1 \rightarrow F_{b,Rd,1} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = 2x\frac{2.5x1x430x20x8}{1.25} = 275.2\ kN$$

$$for\ bolt\ 2 \rightarrow\ F_{b,Rd,2} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = 2x\frac{2.5x0.75x430x20x8}{1.25} = 206.4\ kN$$

$$F_{b,Rd,p} = 206.4\ kN$$

Flange Component

e_1	90 mm
p_1	100 mm
e_2	100 mm
p_2	100 mm

$$for\ bolt\ 1 \rightarrow F_{b,Rd,1} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = \frac{2.5x1x430x20x14}{1.25} = 240.8\ kN$$

$$for\ bolt\ 2 \rightarrow\ F_{b,Rd,2} = \frac{k_1 x \propto_b x f_u x d x t}{\gamma_{M2}} = \frac{2.5x0.75x430x20x14}{1.25} = 180.6\ kN$$

$$F_{b,Rd,f} = 180.6\ kN$$

Checking bolts:

For flange

- Design bearing resistance: $F_{v,Ed,f} = 131.8\ kN < F_{b,Rd,f} = 180.6\ kN$
- Design slip resistance: $F_{v,Ed,f} = 131.8\ kN < F_{s,Rdf} = 157.1\ kN$

Group of fasteners (considering shear plane does not pass through the threaded portion)

$$F_{v,Rd} = \frac{\propto_v\ x f_{ub} x A}{\gamma_{M2}} = \frac{0.6x1000x855}{1.25} = 410\ kN$$

$F_{v,Rd} > F_{b,Rd,w}$ -> Bearing capacity governs.

$\boldsymbol{F_{gr,b,Rd,w} = 8\ x\ 180.6 = 1444.8\ kN > \ N_{N,w} = 1051\ kN}$

For plate

- Design bearing resistance: $F_{v,Ed,p} = 131.8\ kN < F_{b,Rd,p} = 206.4\ kN$
- Design slip resistance: $F_{v,Ed,p} = 131.8\ kN < F_{s,Rd,p} = 157.1\ kN$

Design of net cross section

Flange component

- Net cross section: $A_{f,net} = A_f - 3d_{0,f}t_f = 4511.5 - 3x33x14 = 3125.5\ mm^2$
- Design resistance: $N_{f,net,Rd} = \frac{A_{f,net}\ x\ f_y}{\gamma_{M0}} = \frac{3125.5\ x\ 275}{1} = 859.5\ kN$
- $N_{w,net,Rd} = 859.5\ kN > \sum F_{v,Ed,f} = 2\ x\ 131.8 = 263.6\ kN$

Plate Component

- Net cross section: $A_{p,net} = A_p - 3d_{0,w}t_p = 1920 - 3x20x8 = 1440\ mm^2$
- Design resistance: $N_{p,net,Rd} = \frac{A_{p,net}\ x\ f_y}{\gamma_{M0}} = \frac{1440\ x\ 275}{1} = 396\ kN$
- $N_{p,net,Rd} = 396\ kN > \sum F_{v,Ed,w} = 2\ x\ 32.4 = 129.6\ kN$

Block Tearing:

- Flange component:

$A_{nt} = (2xp_2 - 2xd_0)xt_w = (2x100 - 2x33)x14 = 1876\ mm^2$

$A_{nv} = 2x(e_1 + p_1 - 1.5\ d_0)xt_w = 2x(90 + 100 - 1.5x33)x14 = 3934\ mm^2$

$$V_{eff,1,Rd} = \frac{f_u\ x\ A_{nt}}{\gamma_{M2}} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{f_u\ x\ A_{nv}}{\gamma_{M0}}\right) = \frac{430\ x1876}{1.25} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{430\ x\ 3934}{1.0}\right) = 1622\ kN$$

$V_{eff,1,Rd} = 1622\ kN > N_{N,w} = 1055\ kN$

- Plate component:

$A_{nt} = (2xe_2 - d_0)xt_p = (2x90 - 2x33)x14 = 1596\ mm^2$

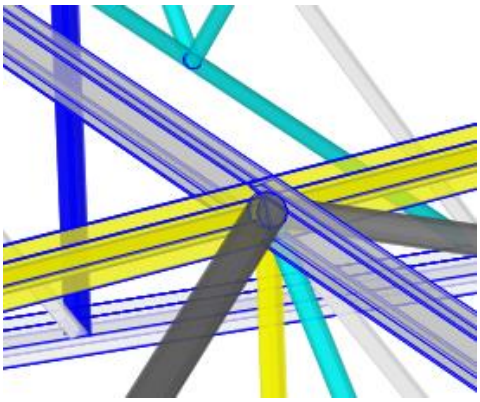
$A_{nv} = 2x(e_1 + p_1 - 1.5\ d_0)xt_p = 2x(90 + 100 - 1.5x33)x14 = 3934\ mm^2$

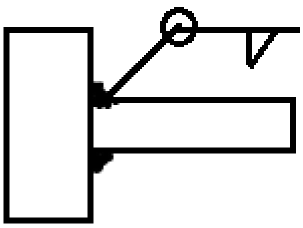
$$V_{eff,1,Rd} = \frac{f_u\ x\ A_{nt}}{\gamma_{M2}} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{f_u\ x\ A_{nv}}{\gamma_{M0}}\right) = \frac{430\ x1596}{1.25} + \left(\frac{1}{\sqrt{3}}\right)\left(\frac{430\ x3934}{1.0}\right) = 862\ kN$$

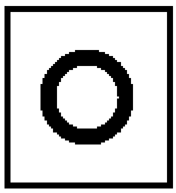
$V_{eff,1,Rd} = 1526\ kN > N_{N,w} = 1055\ kN$

There are 2 rows 8 bolts at the flanges, 2 rows 8 blots at the web due to the splice plate connection.

- **Welded Connection 1**







Braced tube section (CHCF244.5x10) connected to the top chord of the truss.

Maximum tension = 367 kN

Section properties: D = 244.5 mm, T = 10 mm

Select a mm weld with a > 3 mm

- $N_{Ed} = 367\ kN \rightarrow F_{w,Rd} = a\ x\ \pi\ x\ 244.5\ x\ \frac{430}{\sqrt{3}}\ x\ \frac{1}{1.25\ x\ 0.85} = 367000 \rightarrow a = 2\ mm$

- $A = \pi\ (244.5^2 - 235.5^2)/4 = 3393\ mm^2$

  $N_{u,Rd} = 0.9\ x\ 430\ x3393/1.25 = 1050\ kN$

  $\rightarrow F_{w,Rd} = a\ x\ \pi\ x\ 244.5\ x\ \frac{430}{\sqrt{3}}\ x\ \frac{1}{1.25\ x\ 0.85} = 367000 \rightarrow a = 5.85\ mm$

a = 4 mm is chosen for welded connection.

- **Column - Base Plate Connection**

Material Strength:

- Steel Grade S275 f_y = 275 Mpa
- Concrete C35/45 f_{ck} = 35 Mpa
- HE320A

Column flange thickness is t_f = 15.5 mm. The thickness of the base plate shoould not be less than the thickness of the column flange. Therefore, use a base-plate thickness t > t_f

t = 20 mm (say) Determine the maximum potential effective bearing width, c, of a plate

$c = t\left(\frac{f_y}{3f_j\gamma_{m0}}\right)^{0.5} where$ (Annex L.1 (3))

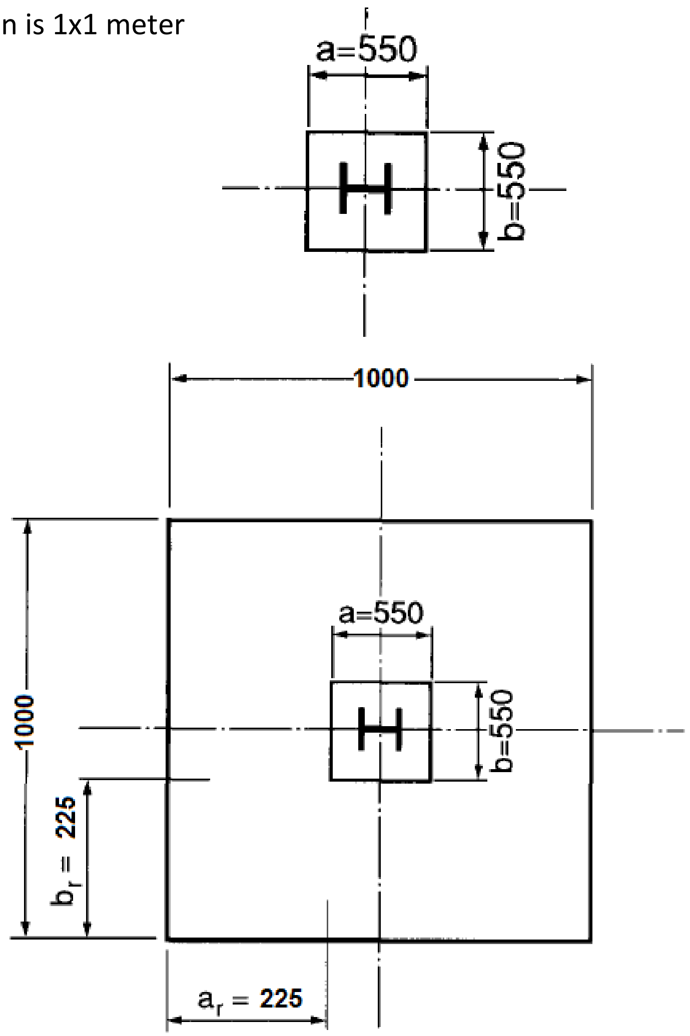
$f_j = \beta_j k_j f_{cd} \rightarrow f_{cd} = \frac{f_{ck}}{\gamma_c} = \frac{35}{1.5} = 23\ N/mm^2$ (Annex L.1 (6))

$\beta_j = 0.67$

$k_j\ is\ the\ concentration\ factor = \left(\frac{a_1 b_1}{ab}\right)^{0.5}$ (Annex L.1 (7))

Let say a = b = 550 mm

Let's assume column is 1x1 meter



a_1 = lesser of;

- $a + 2a_r = 550 + 2 \times 225 = 1000$ mm
- $5a = 5 \times 550 = 2750$ mm
- $a + h = 550 + 30000$ (elevation) $= 30550$ mm

$a_1 = 1000$ mm

due to the square, $b_1 = a_1$

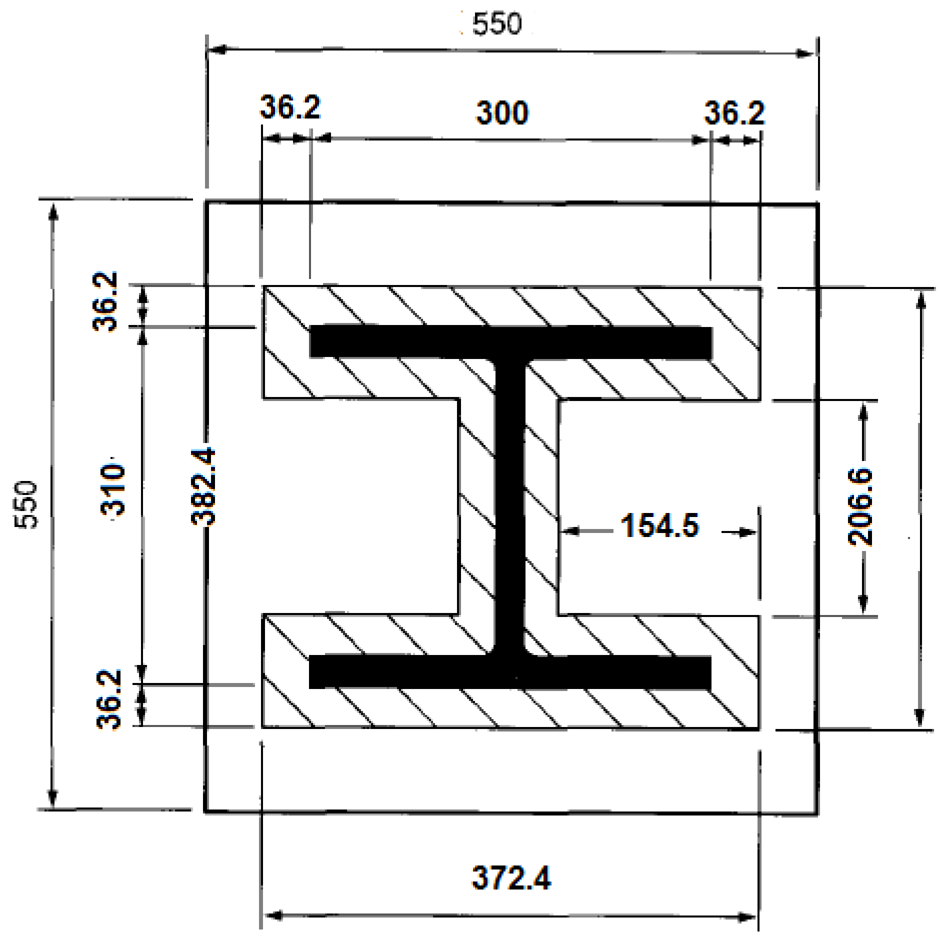
$$k_j = \left(\frac{a_1 b_1}{ab}\right) = \left(\frac{1000\ x\ 1000}{550\ x\ 550}\right)^{0.5} = 1.81$$

$$f_j = 0.67\ x\ 1.81\ x\ 23 = 28\ N/mm^2$$

$$\gamma_{m0} = 1.0\ , f_y = 275\ MPa$$

$$c = t\left(\frac{f_y}{3f_j\gamma_{m0}}\right)^{0.5} \rightarrow c = 20\left(\frac{275}{3x28x1}\right)^{0.5} = 36.17\ mm$$

The figure shows that the effective area of the base plate is based on a cantilever projection of 36.17 mm from the column. The bearing stresses are therefore distributed in an I shaped profile.



Effective area: $A_{eff} = (310 + 2x36.2)(300 + 2x36.2) - 2x206.6x154.5 = 78566.4\ mm^2$

Design bearing pressure = $\frac{N_{Sd}}{Aeff} = \frac{1001000}{78566.4} = 12.7\ N/mm^2$

Bearing strength = $f_j = 28\ N/mm^2 > 12.7\ N/mm^2$ -> **satisfactory**

- Shear Resistance

$$N_{sd} = 1001\ kN\ (from\ SAP\ analysis) \rightarrow \frac{N_{sd}}{5} = 0.2\ x\ 1001 = 200.2\ kN$$
$$V_{sd} = 75\ kN\ (from\ SAP\ analysis) < 721\ kN \rightarrow \mathbf{satisfactory}$$

- Plate dimension

Minimum width of plate required = b + 2c = 300 + 2x36.2 = **372.4 mm**
Minimum depth of plate required = d + 2c = 310 + 2x36.2 = **382.4 mm**

Use 550x550x20 mm thick S275 base-plate

- Welding requirements

$$V_{sd} = 75\ kN\ and\ weld\ shear\ strength\ f_{vw,d} = \frac{f_u}{\beta_w \gamma_{Mw}\ x\ \sqrt{3}}\ \ where$$

$$f_u = 430\ N/mm^2, \beta_w = 0.85, \gamma_{Mw} = 1.35$$
$$f_{vw,d} = \frac{430}{0.85x1.35x\ \sqrt{3}} = 216.3\ N/mm^2$$

$Using\ 6\ mm\ fillet\ weld$: –> Throat thickness, a = 0.7 x 6 = **4.2 mm**

$$Resistance\ of\ weld/mm:\ \ F_{v,Rd} = \ f_{vw,d}\ x\ a = 216.3\ x\ 4.2 = 908.46\ \ N/mm$$

$$length\ of\ weld\ required = \frac{75000}{908.46} = \mathbf{83\ mm}$$

7. Masses and Weights Analysis of the Roof

Groups 3 - Masses and Weights

File View Format-Filter-Sort Select Options

Units: As Noted

Groups 3 - Masses and Weights

GroupName Text	SelfMass Kgf-s2/m	SelfWeight Kgf	TotalMassX Kgf-s2/m	TotalMassY Kgf-s2/m	TotalMassZ Kgf-s2/m
ALL	30806.47	302108.23	30806.47	30806.47	30806.47