ARCT 240: Semester Project

PRACTICAL ANALYSIS AND DESIGN OF STEEL ROOF TRUSSES TO EUROCODE 3

TASK 1:

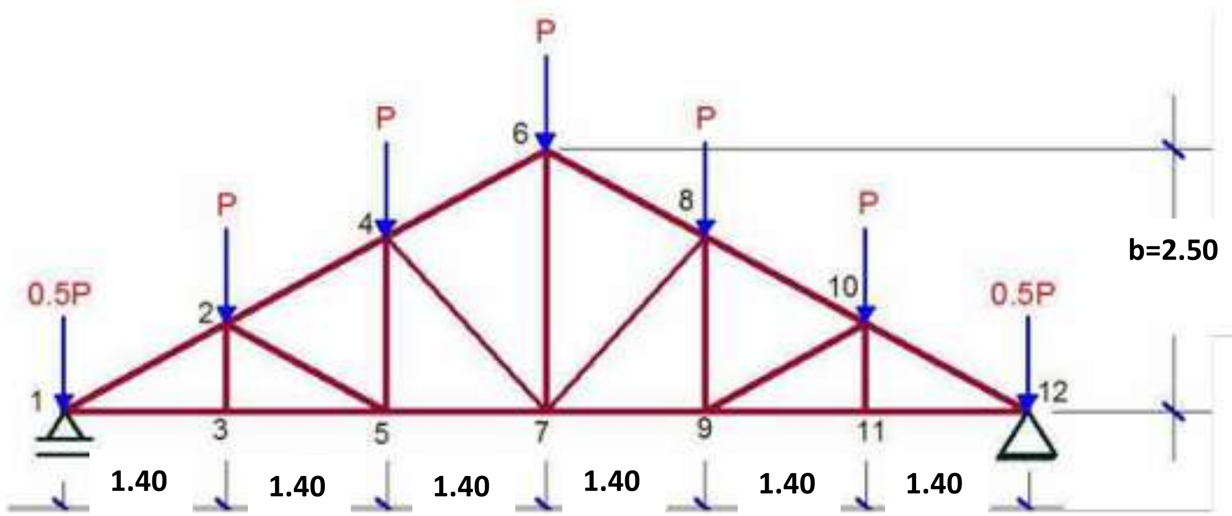
Find the forces in the members due to the dead load Once you have completed task 1, you are required to make a hand calculation check: first draw the FBD of your truss; second, calculate the reactions; third, calculate the member forces for only 2 nodes (node 1 and node 2 in Figure 2). Compare the forces calculated manually and by the online tool and check they are the same/very similar.

TASK 2:

Find the forces in the members due to the live load.

TASK 3:

Find the forces in the members due to the wind load



Weight of long span aluminium roofing sheet	0.019 KN/m^2
Weight of ceiling (adopt 10mm insulation fibre board)	0.077 KN/m^2
Weight of services	0.1 KN/m^2
Weight of purlin (assume CH 150 x 75 x 18 kg/m)	0.147 KN/m^2
Self-weight of trusses	0.2 KN/m^2
Yield strength of steel (F_y)	275 N/mm^2
Ultimate tensile strength of steel (F_u)	430 N/mm^2

Live Load (LL)

Category of roof = Category H – Roof not accessible except for normal maintenance and repairs (Table 6.9 EN 1991-1-1:2001)

Imposed load on roof (Qk) = 0.75 KN/m^2

Wind Load (WL)

Wind velocity pressure (dynamic) is assumed as = qp(z) = 1.5 kN/m^2 When the wind is blowing from right to left, the resultant pressure coefficient on the windward and leeward slopes with positive internal pressure (cpe) is taken as −0.9.

Therefore, the external wind pressure normal to the roof is;

qe = $q_p c_{pe} = -1.5 \times 0.9 = 1.35$ kN/m^2

Vertical component = qe cos θ (acting upwards ↑)

$$\tan\theta = \frac{b}{3c} = \frac{2.50}{4.20} = 0.595 \;\Rightarrow\; \theta \approx 30.7^\circ$$

Task 1 - Forces in members due to dead load (DL)

$a = 3.75 \text{ m} = 3750 \text{ mm}$

$b = 2.50 \text{ m} = 2500 \text{ mm}$

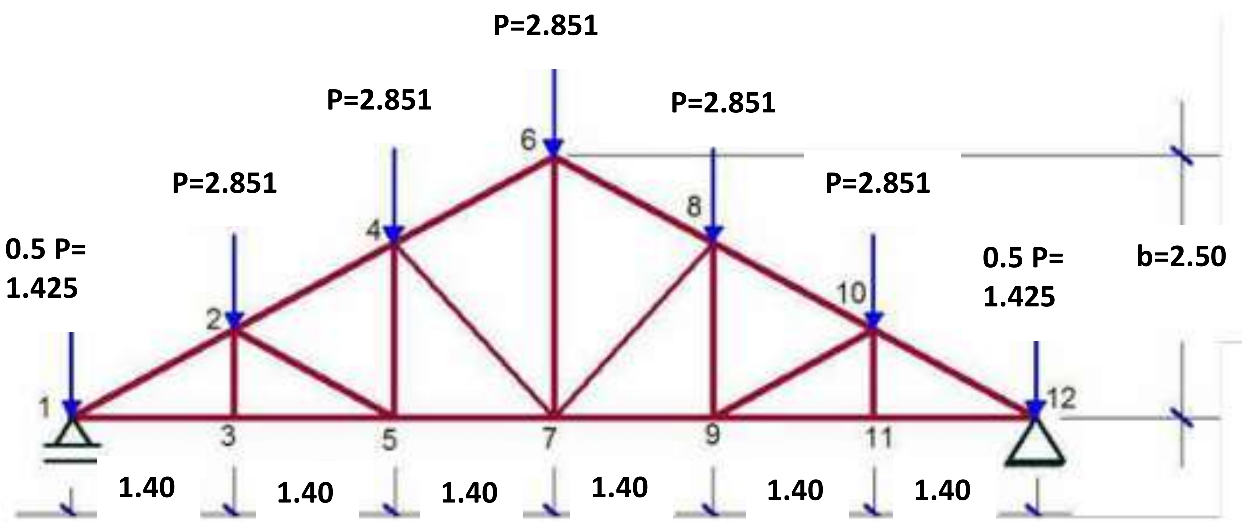
$c = 1.40 \text{ m} = 1400 \text{ mm}$

Dead load calculation

$0.019 \text{ KN}/m^2 + 0.077 \text{ KN}/m^2 + 0.10 \text{ KN}/m^2 + 0.147 \text{ KN}/m^2 + 0.20 \text{ KN}/m^2 = 0.543 \text{ KN}/m^2$ =**$0.000543 \text{ N}/mm^2$.**

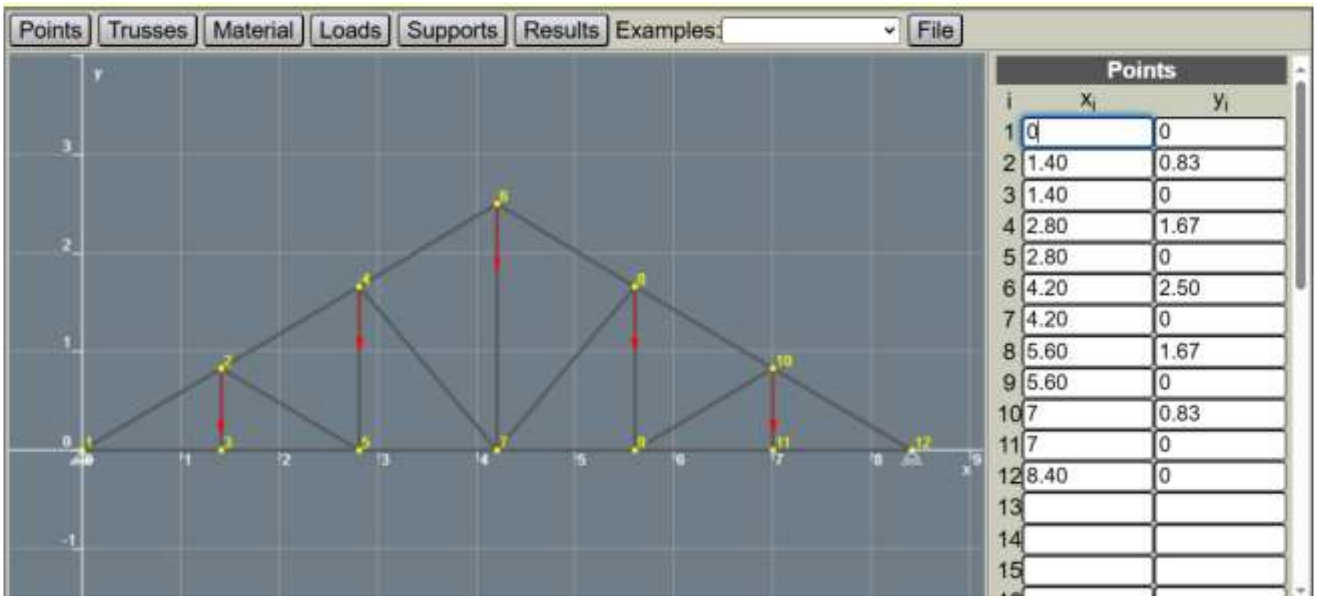
$\text{P} = \text{DL} * \text{a} * \text{c} =$ 0.000543*3750*1400=2850.8 N=**2.851 KN.**

0.5P=$\frac{2.851}{2}$=**1.425 KN.**

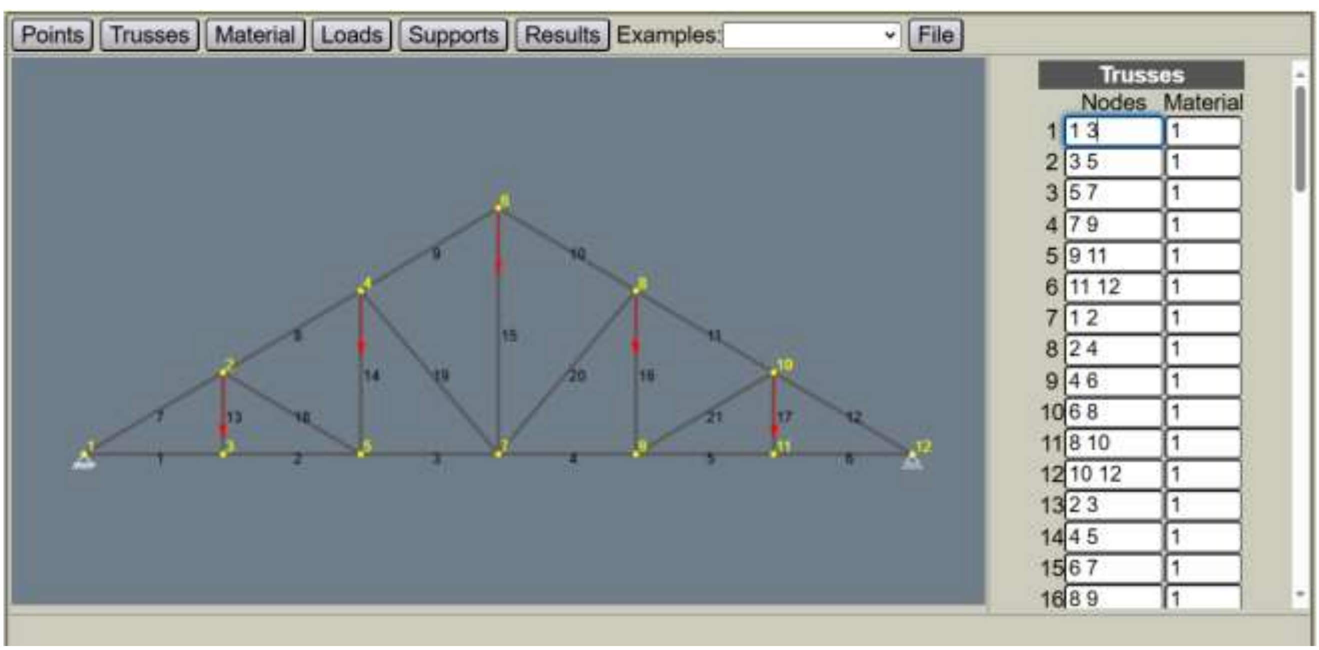


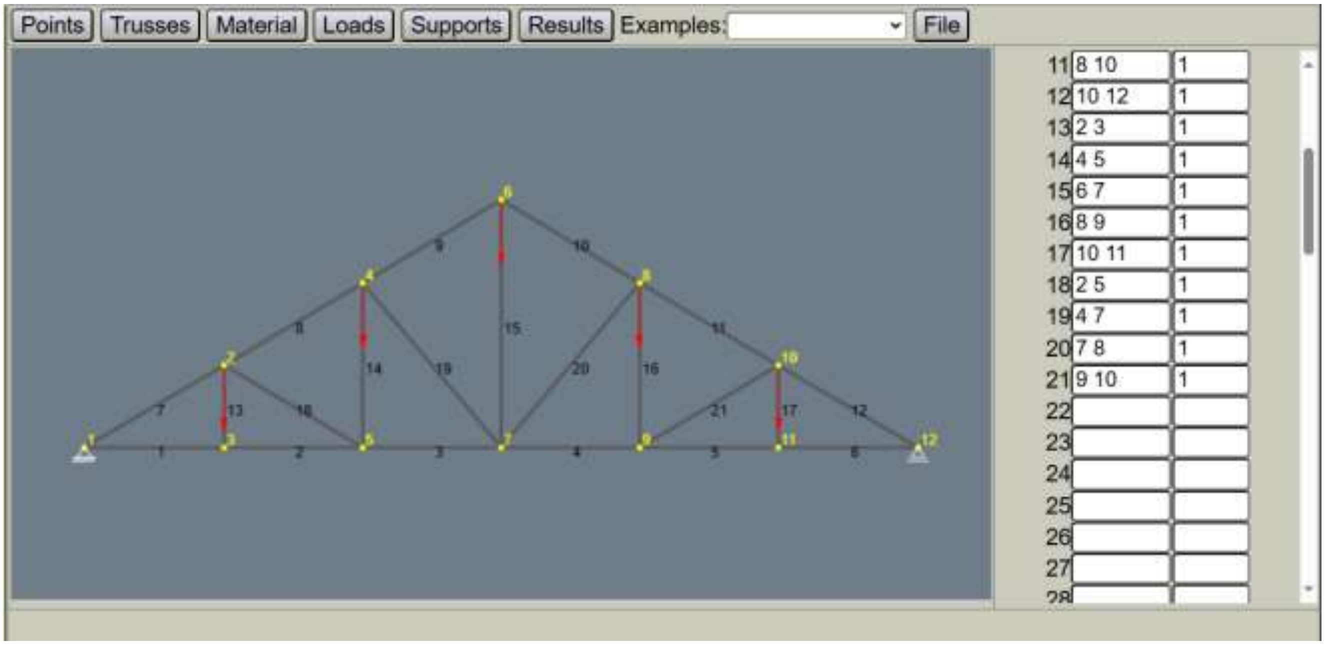
Results from 2 D truss calculator

Truss points:

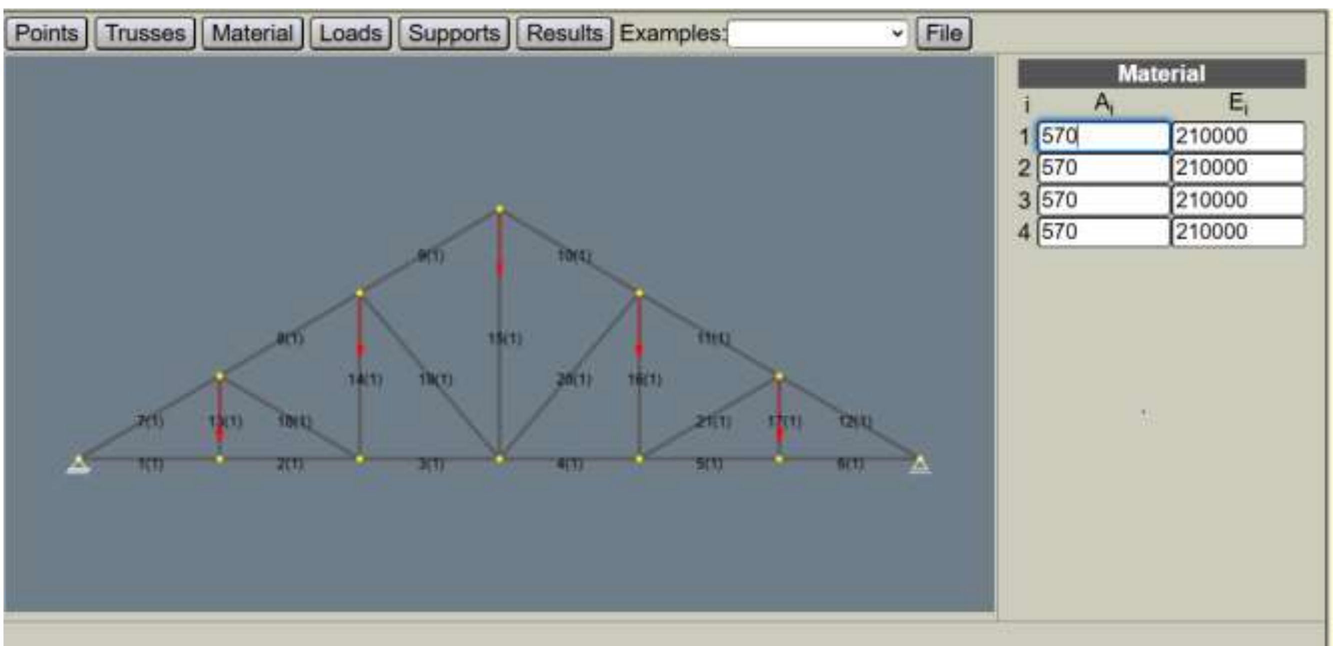


Truss members:

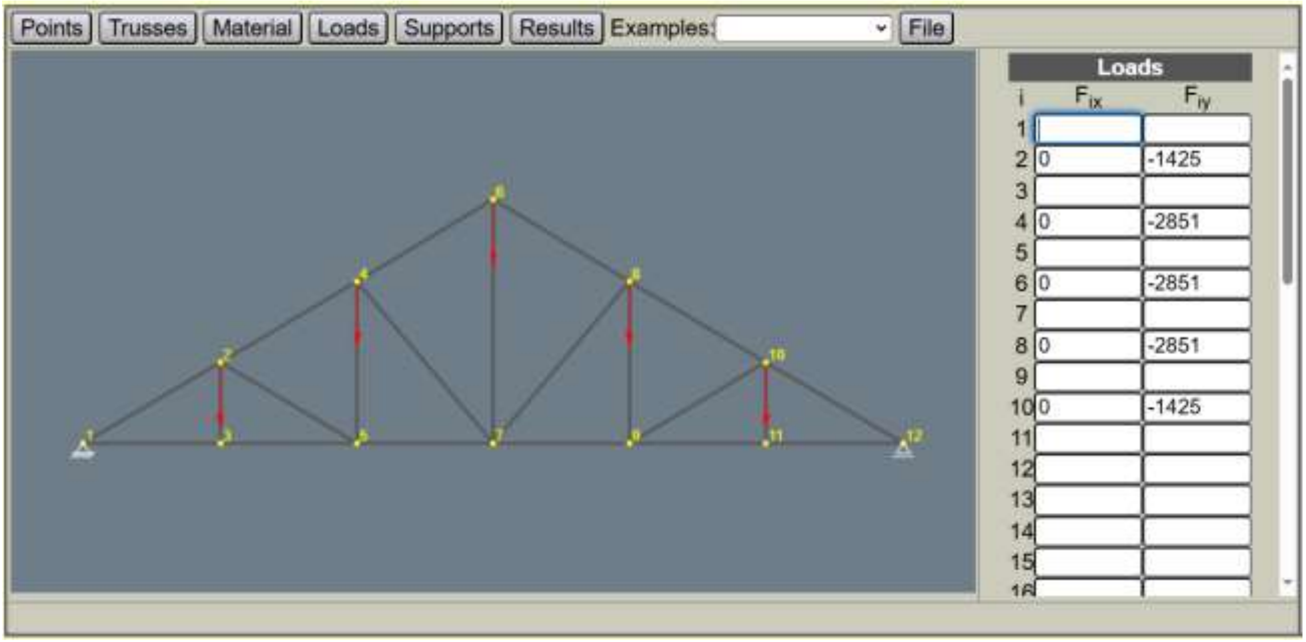




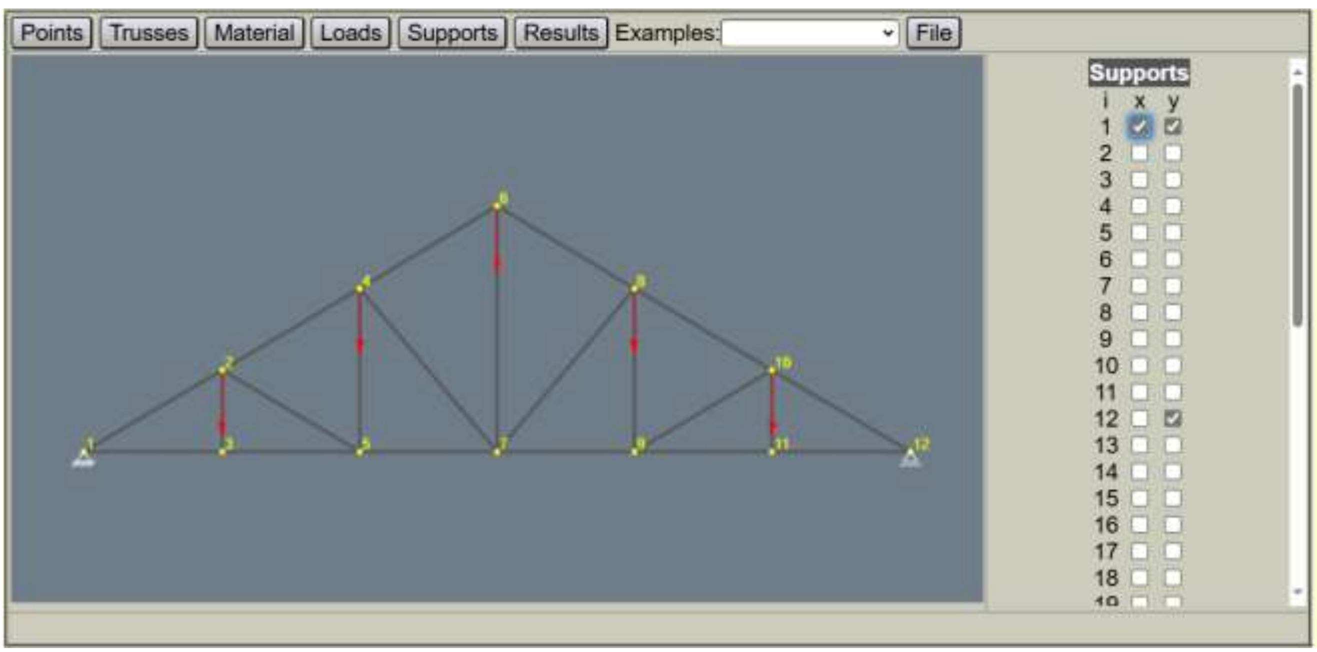
Truss Materials:



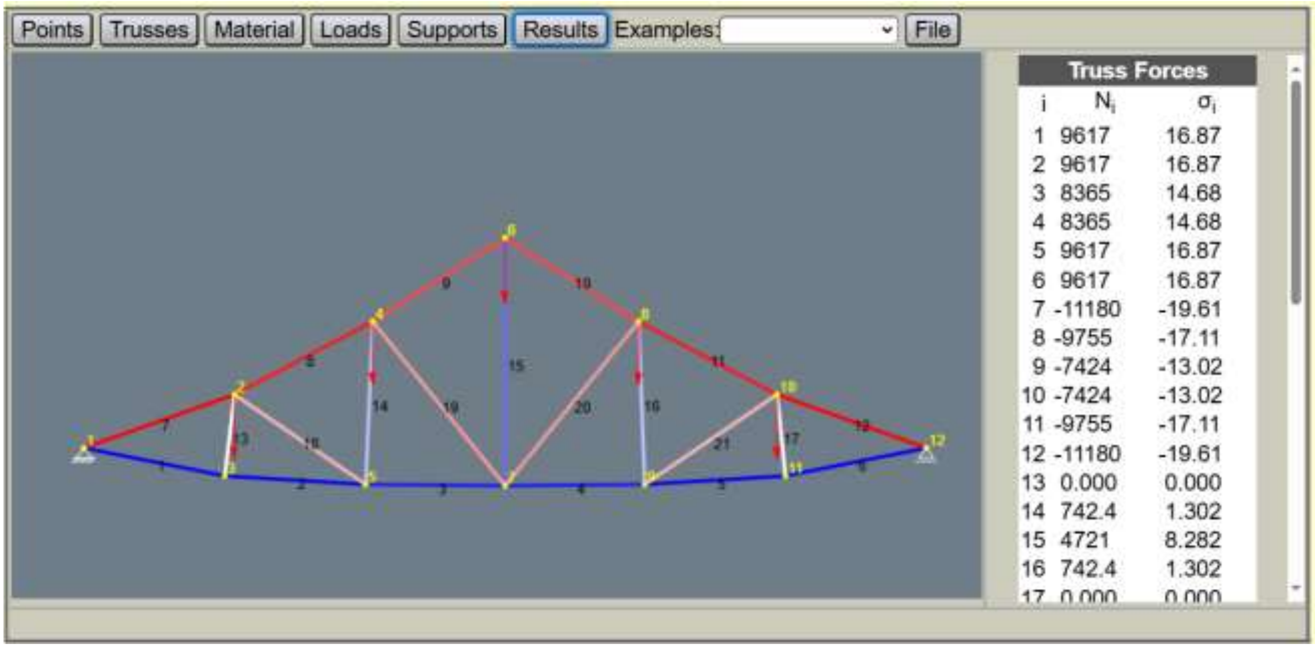
Loads:

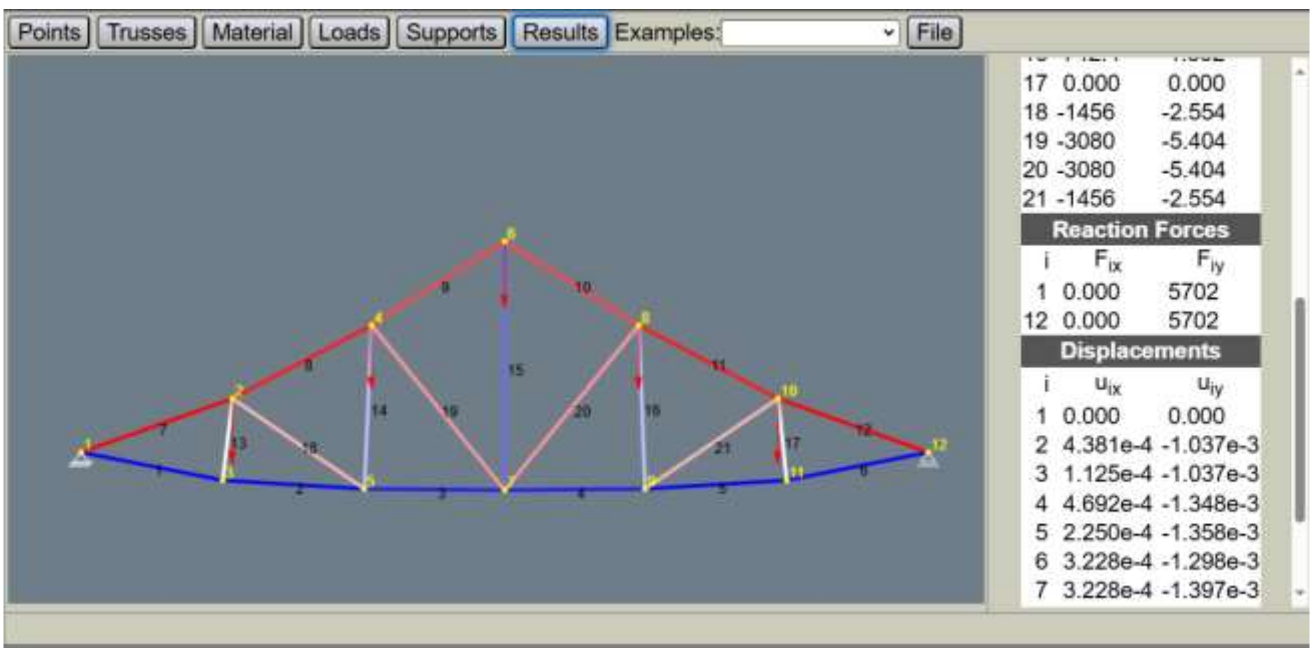


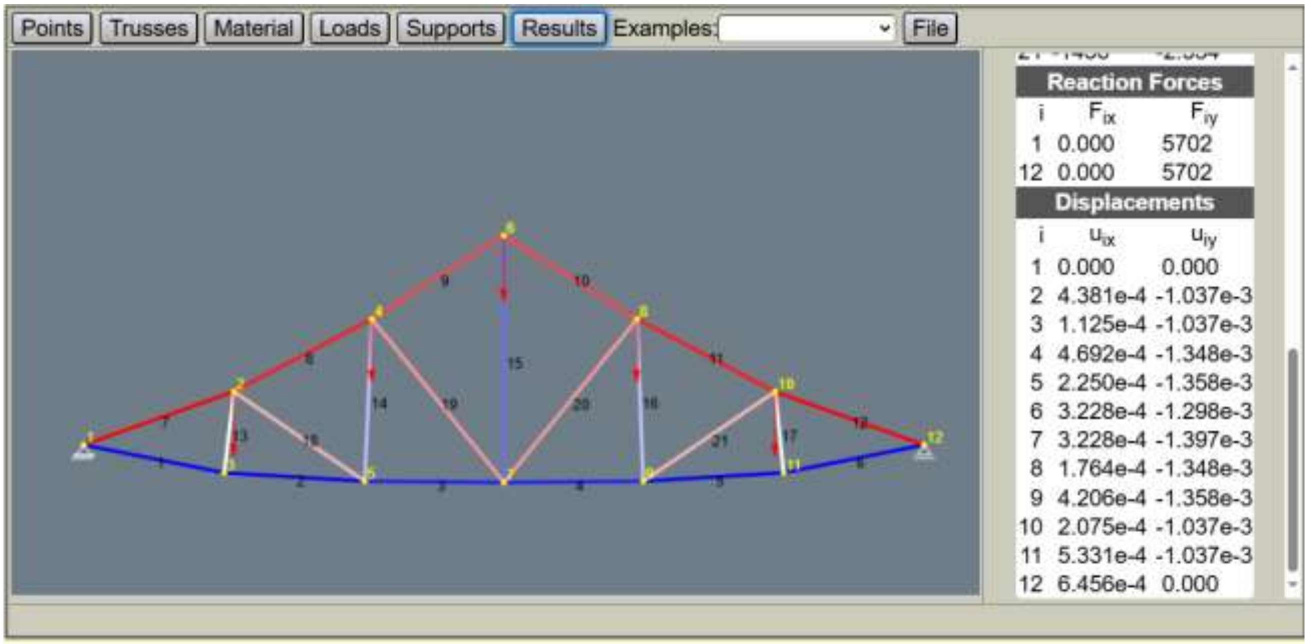
Supports:



Result:


Points
Trusses
Material
Loads
Supports
Results
Examples:
File
Truss Forces
i N_i σ_i
1 9617 16.87
2 9617 16.87
3 8365 14.68
4 8365 14.68
5 9617 16.87
6 9617 16.87
7 -11180 -19.61
8 -9755 -17.11
9 -7424 -13.02
10 -7424 -13.02
11 -9755 -17.11
12 -11180 -19.61
13 0.000 0.000
14 742.4 1.302
15 4721 8.282
16 742.4 1.302
17 0.000 0.000


Points
Trusses
Material
Loads
Supports
Results
Examples:
File
17 0.000 0.000
18 -1456 -2.554
19 -3080 -5.404
20 -3080 -5.404
21 -1456 -2.554
Reaction Forces
i F_ix F_iy
1 0.000 5702
12 0.000 5702
Displacements
i u_ix u_iy
1 0.000 0.000
2 4.381e-4 -1.037e-3
3 1.125e-4 -1.037e-3
4 4.692e-4 -1.348e-3
5 2.250e-4 -1.358e-3
6 3.228e-4 -1.298e-3
7 3.228e-4 -1.397e-3



Points
Trusses
Material
Loads
Supports
Results
Examples:
File
Reaction Forces
i F_{ix} F_{iy}
1 0.000 5702
12 0.000 5702
Displacements
i u_{ix} u_{iy}
1 0.000 0.000
2 4.381e-4 -1.037e-3
3 1.125e-4 -1.037e-3
4 4.692e-4 -1.348e-3
5 2.250e-4 -1.358e-3
6 3.228e-4 -1.298e-3
7 3.228e-4 -1.397e-3
8 1.764e-4 -1.348e-3
9 4.206e-4 -1.358e-3
10 2.075e-4 -1.037e-3
11 5.331e-4 -1.037e-3
12 6.456e-4 0.000

Task 1 – Hand Calculation Confirmation

Forces in Y direction

$$(1425.4 \times 2) + (2850.8 \times 3) - R_{1y} - R_{12y} = 0$$
$$11\,403.2 - R_{1y} - R_{12y} = 0$$

Since the truss and loading are symmetrical:

$$R_{1y} = R_{12y}$$
$$R_{1y} = R_{12y} = \frac{11\,403.2}{2} = 5701.6\ N$$

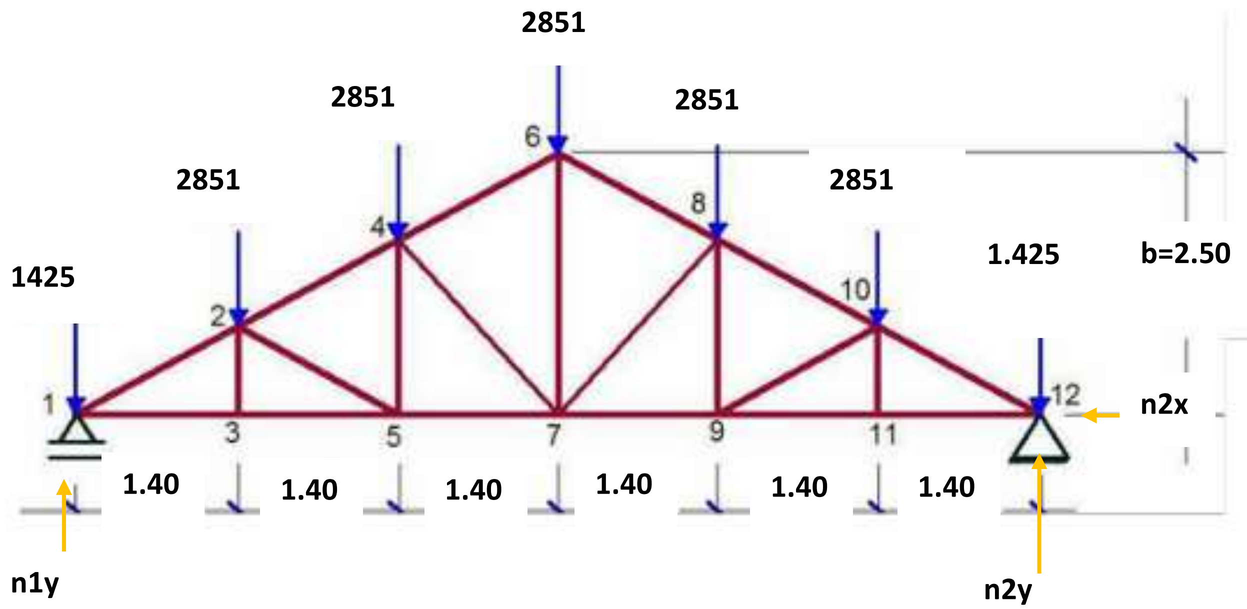
Forces in X direction

$$R_{12x} = 0$$

Moment about left support (Node 1)

Taking moments about node 1 to check R_{12y}:

$$(1425.4 \times 8.40) + (2850.8 \times 7.00) + (2850.8 \times 5.60) + (2850.8 \times 4.20) + (1425.4 \times 2.80) - (8.40 \times R_{12y}) = 0$$
$$44\,765.6 - 8.40R_{12y} = 0$$
$$R_{12y} = 5700.7\ N$$

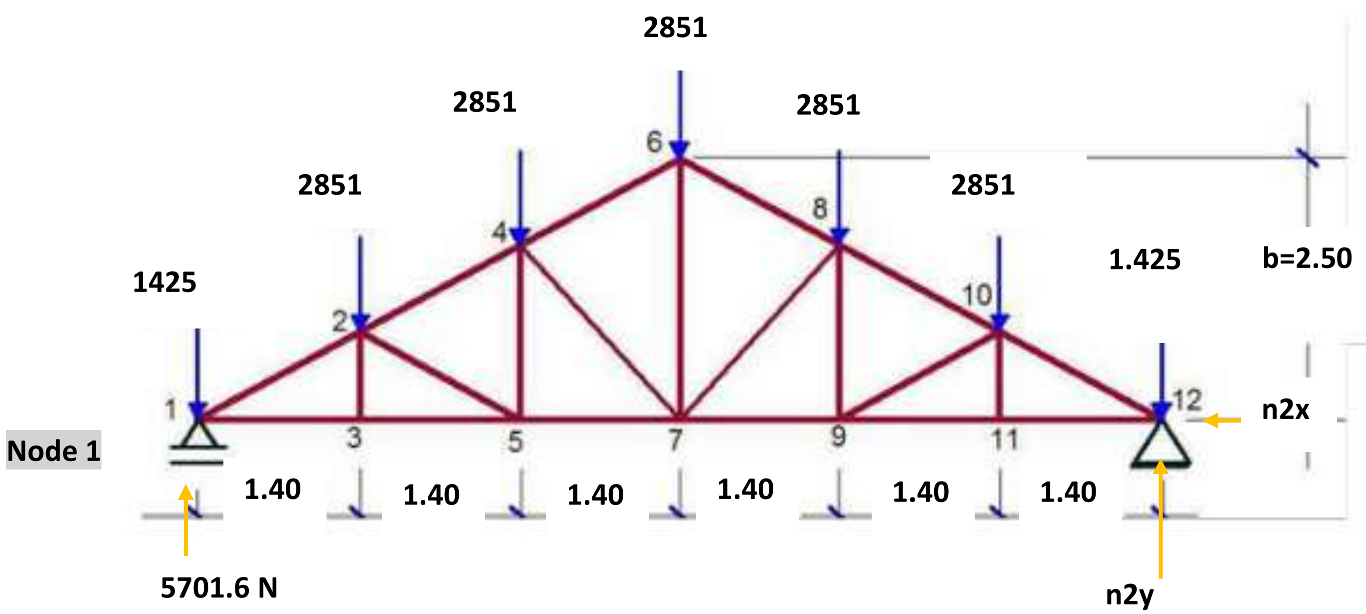


Results Comparison:

From Truss Calculator	From Hand Calculation
(R1y) = 5700 N	(R1y) = 5701.6 N
(R1x) = 0 N	(R1x) = 0 N
(R12y) = 5700 N	(R12y) = 5700.7 N

Task 1 - hand calculation confirmation - Node 1

- $P = 1425.4$ N (at Node 2)
- $\theta = 30.8°$
- $R_{1y} = 5701.6$ N



Equilibrium in the Y-direction

$$R_{1y} + F_{1-2}\sin\,\theta = 0$$

$$F_{1-2} = -\frac{R_{1y}}{\sin\theta}$$

$$F_{1-2} = -\frac{5701.6}{\sin 30.8°} = -11150 \text{ N (Compression)}$$

Equilibrium in the X-direction

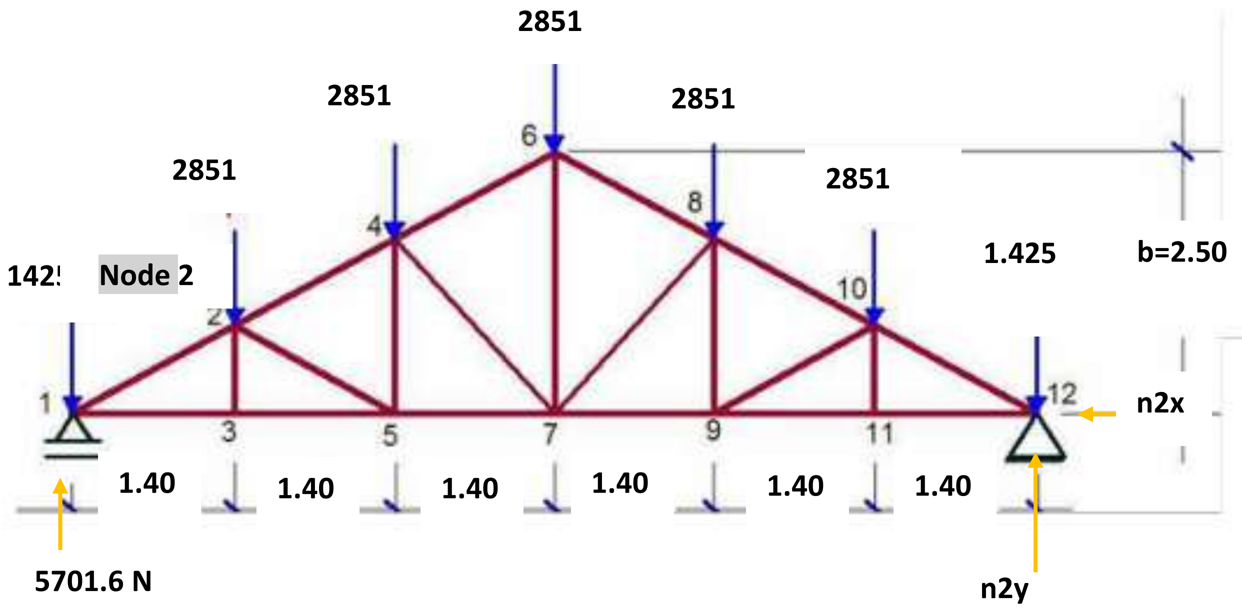
$$F_{1-3} + F_{1-2}\cos\ \theta = 0$$
$$F_{1-3} = -F_{1-2}\cos\ \theta$$
$$F_{1-3} = -(-11150)(\cos 30.8°) = +9580\ n\ (Tension)$$

Result Comparison

From Truss Calculator	From Hand Calculation
(F1-2 = -11180N)	(F1-2 = -11150N)
(F1-3 = +9617N)	(F1-3 = +9580N)

Task 1 – Hand Calculation Confirmation – Node 2

- $P = 1425.4$ N(at Node 2)
- $\theta = 30.8°$
- $F_{1-2} = 11150$ N (Compression)
- F_{2-3}vertical
- F_{2-4}inclined
- F_{2-5}horizontal



Equilibrium in the X-direction

$$F_{2-5} + F_{2-4}\cos\,\theta - F_{1-2}\cos\,\theta = 0$$
$$F_{2-5} + F_{2-4}(0.86) - (11150)(0.86) = 0$$
$$0.86F_{2-5} + 0.86F_{2-4} = 9599$$
$$F_{2-5} + F_{2-4} = 11{,}160$$

(Equation 1)

Equilibrium in the Y-direction

$$-P - F_{2-3} + F_{2-4}\sin\ \theta = 0$$
$$-1425.4 - F_{2-3} + (F_{2-4})(0.50) = 0$$
$$0.50F_{2-4} = F_{2-3} + 1425.4$$
(Equation 2)

Simultaneous Solution

From the truss calculator and symmetry:

$$F_{2-3} \approx 742 \text{ N (Tension)}$$

Substitute into Eq. (2):

$$0.50F_{2-4} = 742 + 1425.4 = 2167.4$$
$$F_{2-4} = 4334.8 \text{ N (Tension)}$$

Then from Eq. (1):

$$F_{2-5} = 11160 - 4334.8 = 6825.2 \text{ N (Compression)}$$

Result Comparison

From Truss Calculator	From Hand Calculation
(F2-4 = 4335N)	(F2-4 = 4334.8N)
(F2-5 = 6825N)	(F2-5 = 6825.2N)
(F2-3 = 742N)	(F2-3 = 742N)

Task 2 - Forces in members due to live load

$a = 3.75 \text{ m} = 3750 \text{ mm}$

$b = 2.50 \text{ m} = 2500 \text{ mm}$

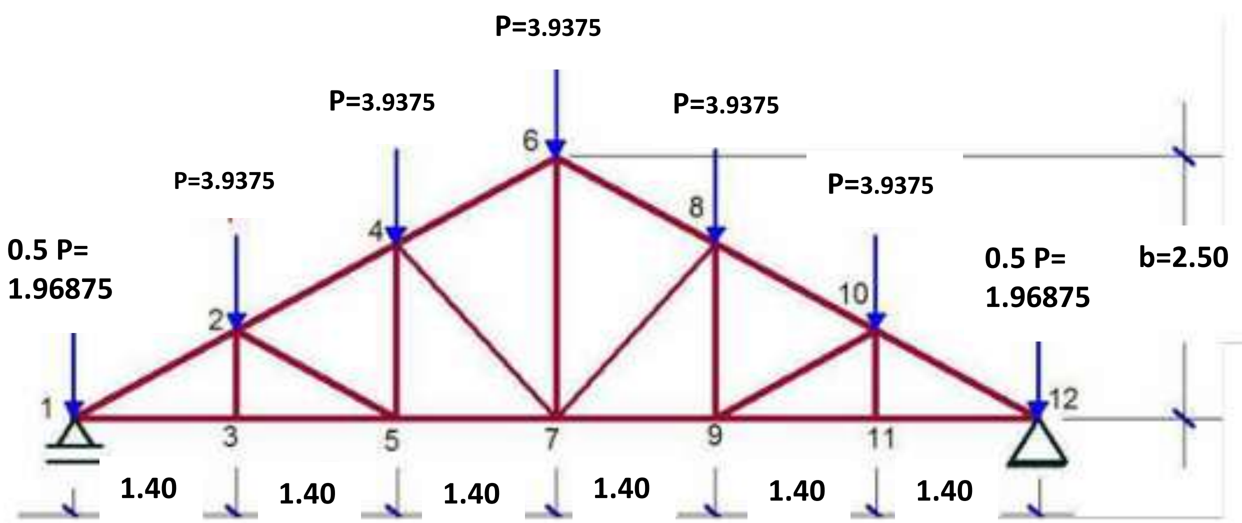
$c = 1.40 \text{ m} = 1400 \text{ mm}$

Livee load calculation

Given $w_{LL} = 0.75 \ \text{KN}/m^2$

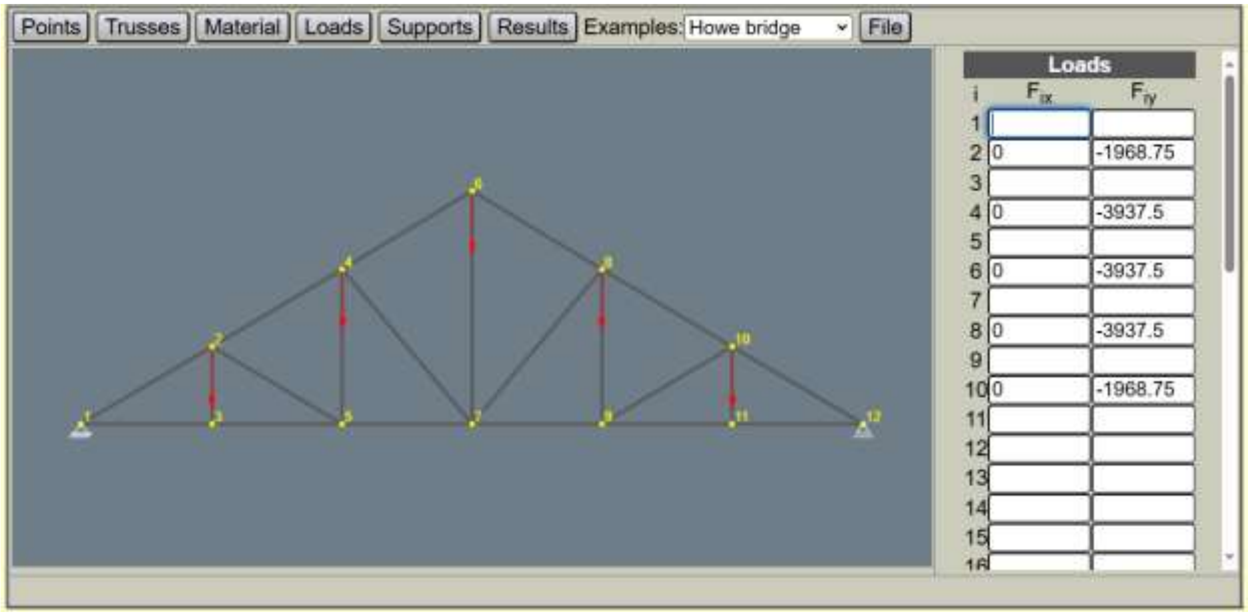
and tributary area $a \times c = 3.75 \times 1.40 = 5.25 m^2$

- **Interior top nodes (4, 6, 8):**
  $P_{LL} = 0.75 \times 5.25 = 3.9375\ KN$=3937.5N
- **End top nodes (2, 10):**
  $0.5 P_{LL} = 1.96875\ KN = 1968.75\ N$

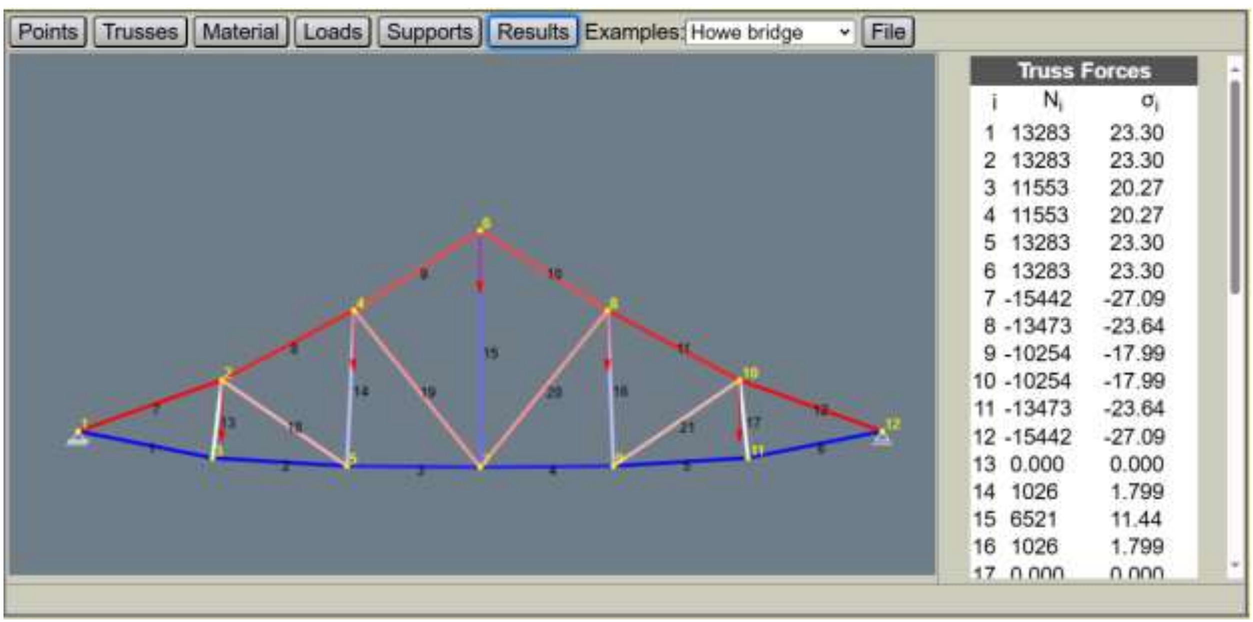


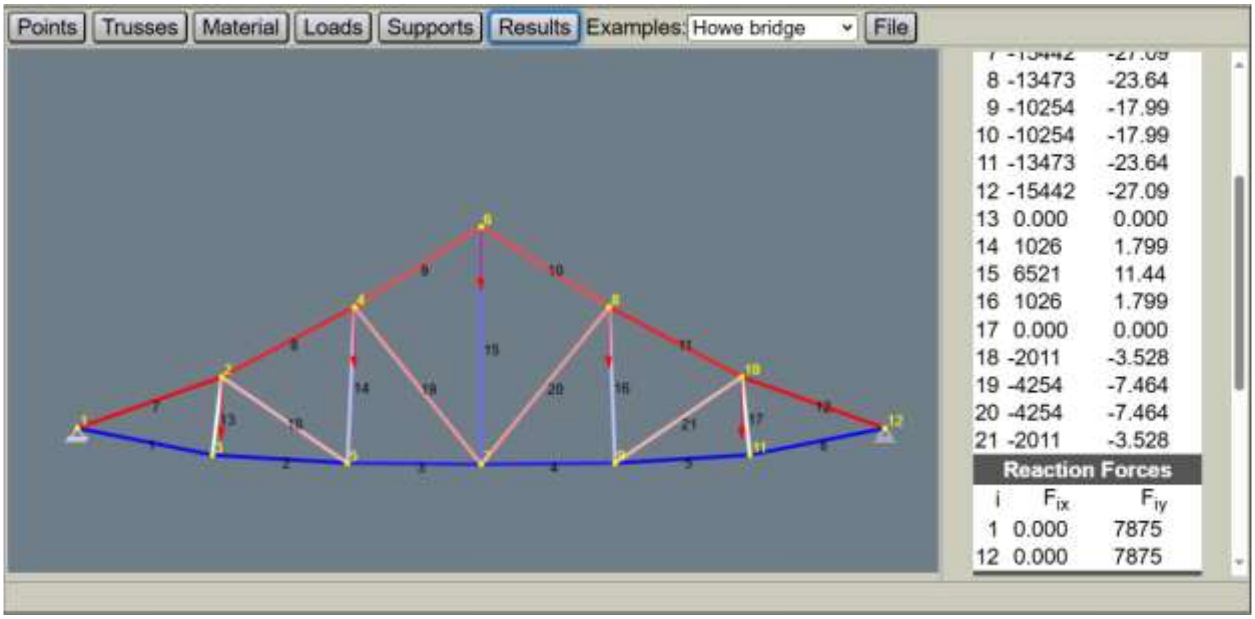
Task 2 - Results from 2D truss calculator

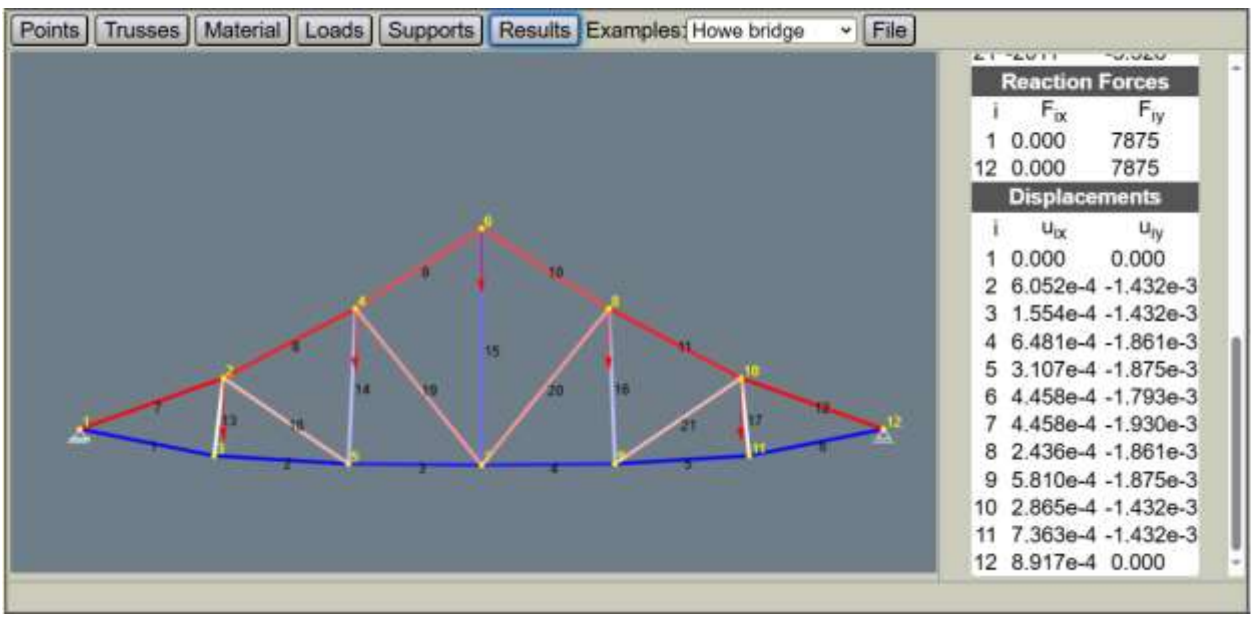
Loads:



Results:




Points
Trusses
Material
Loads
Supports
Results
Examples: Howe bridge
File
8 -13473 -23.64
9 -10254 -17.99
10 -10254 -17.99
11 -13473 -23.64
12 -15442 -27.09
13 0.000 0.000
14 1026 1.799
15 6521 11.44
16 1026 1.799
17 0.000 0.000
18 -2011 -3.528
19 -4254 -7.464
20 -4254 -7.464
21 -2011 -3.528
Reaction Forces
i F_{ix} F_{iy}
1 0.000 7875
12 0.000 7875


Points
Trusses
Material
Loads
Supports
Results
Examples: Howe bridge
File
Reaction Forces
i F_{ix} F_{iy}
1 0.000 7875
12 0.000 7875
Displacements
i u_{ix} u_{iy}
1 0.000 0.000
2 6.052e-4 -1.432e-3
3 1.554e-4 -1.432e-3
4 6.481e-4 -1.861e-3
5 3.107e-4 -1.875e-3
6 4.458e-4 -1.793e-3
7 4.458e-4 -1.930e-3
8 2.436e-4 -1.861e-3
9 5.810e-4 -1.875e-3
10 2.865e-4 -1.432e-3
11 7.363e-4 -1.432e-3
12 8.917e-4 0.000

Task 3: Forces in members due to Wind Load

$a = 3.75 \text{ m} = 3750 \text{ mm}$

$b = 2.50 \text{ m} = 2500 \text{ mm}$

$c = 1.40 \text{ m} = 1400 \text{ mm}$

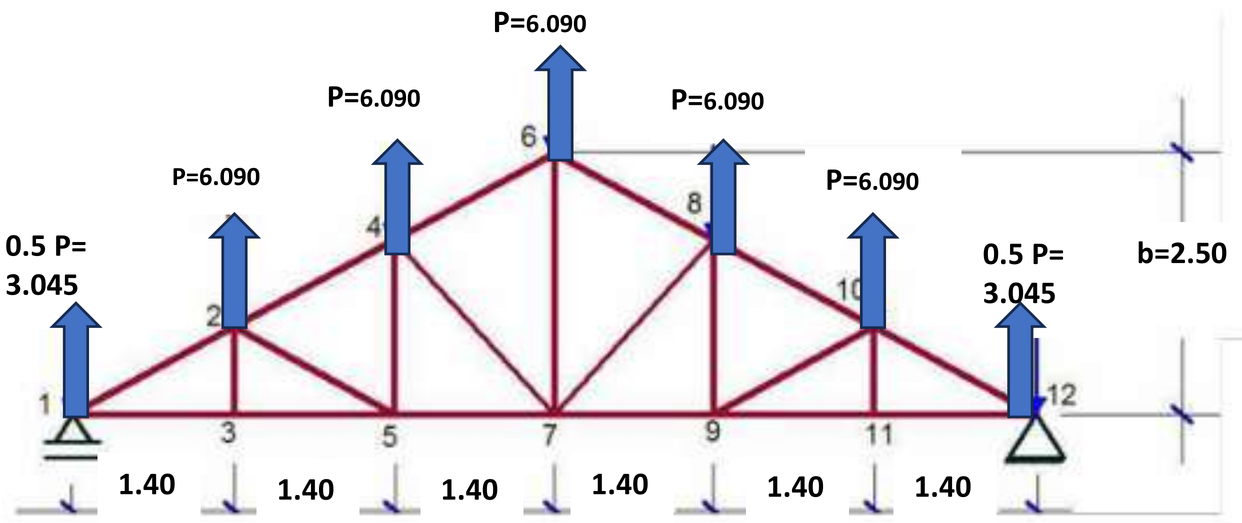
1) Wind load → vertical nodal loads (UPLIFT)

- $q_p = 1.5 \text{ KN}/m^2$
- $c_{pe} = -0.9 \Rightarrow q_e = 1.35 \text{ KN}/m^2$
- Roof angle θ: $\tan\ \theta = b/(3c) = 2.50/4.20 \Rightarrow \theta \approx 30.76°$, $\cos\ \theta \approx 0.857$
- **Vertical component**:

  $w_{WL} = q_e \cos\theta = 1.35 \times 0.857 \approx 1.160 \text{ KN}/m^2$(upwards)
- Tributary area per top node: $a \times c = 3.75 \times 1.40 = 5.25\ m^2$

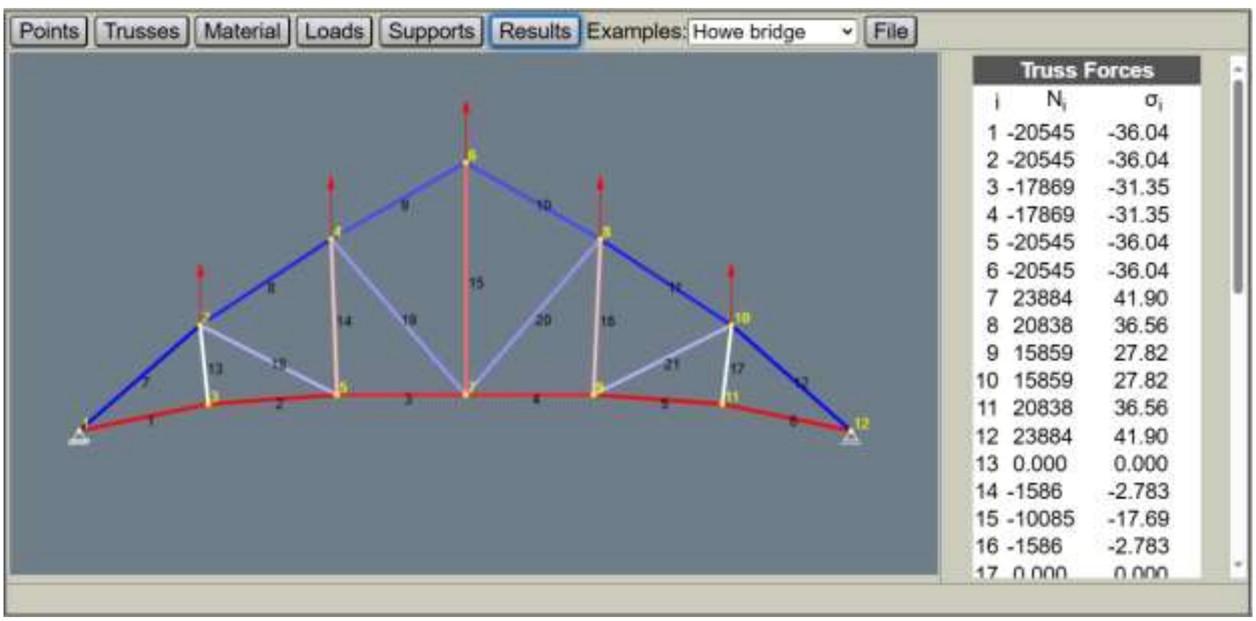
Resulting nodal loads (UP):

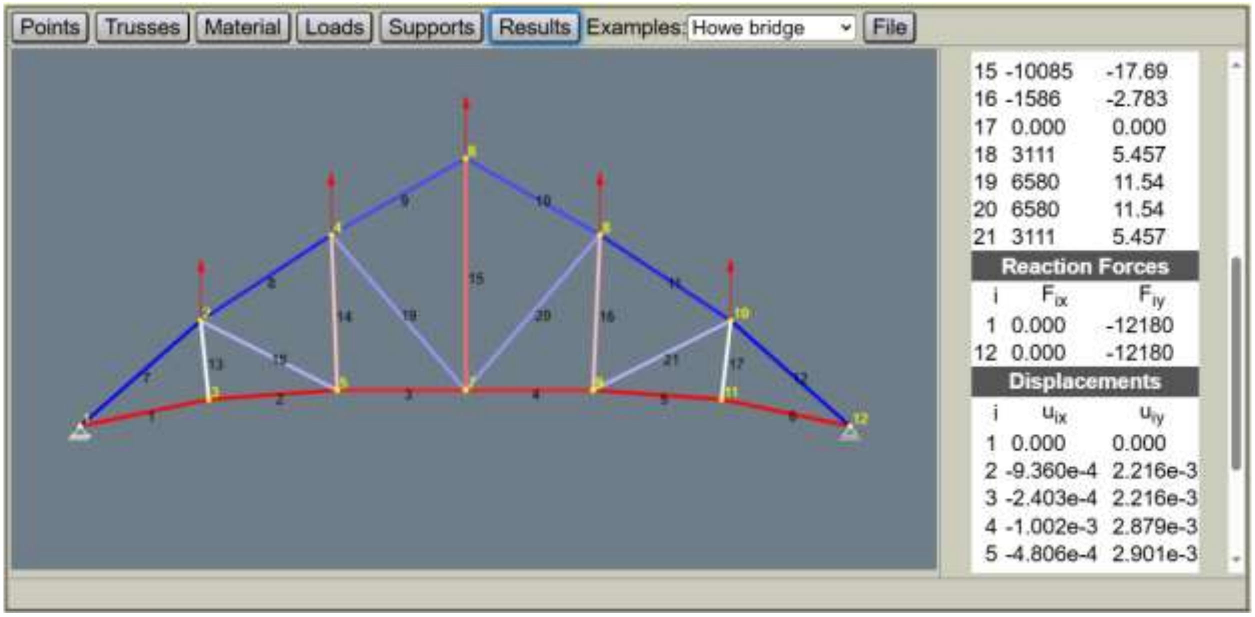
- Interior top nodes (4, 6, 8):
  $$P_{WL} = 1.160 \times 5.25 = 6.090\ KN$$
- End top nodes (2, 10):
  $$0.5P_{WL} = 3.045\ KN$$

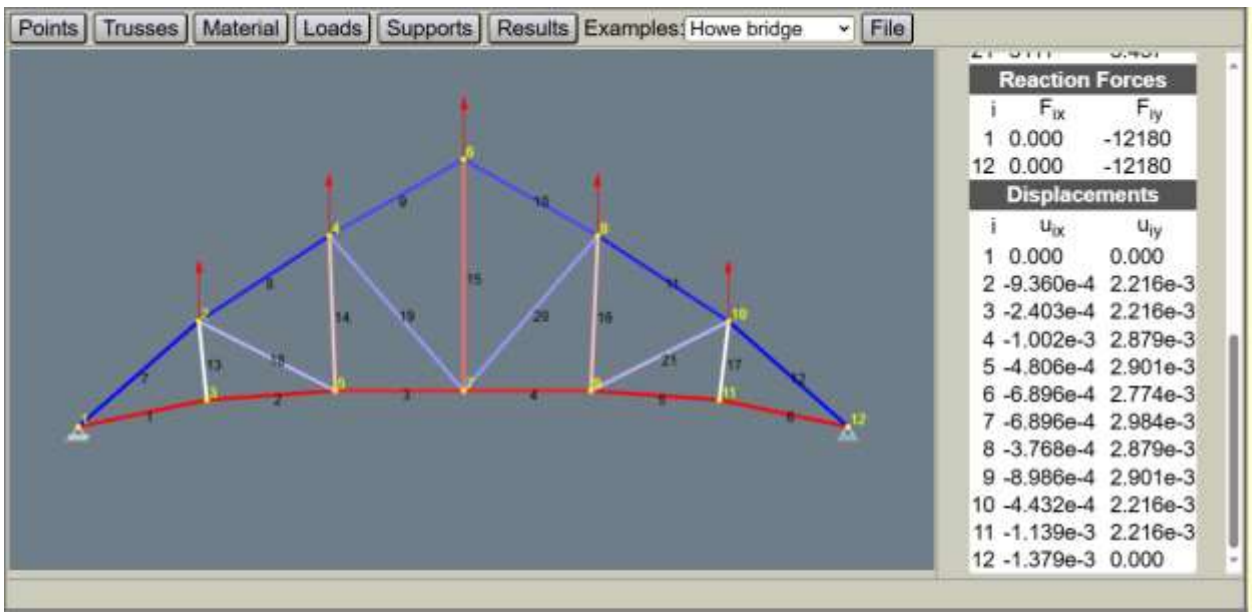


Task 3 - Results from 2D truss calculator

Loads:




Points
Trusses
Material
Loads
Supports
Results
Examples: Howe bridge
File
15 -10085 -17.69
16 -1586 -2.783
17 0.000 0.000
18 3111 5.457
19 6580 11.54
20 6580 11.54
21 3111 5.457
Reaction Forces
i F_ix F_iy
1 0.000 -12180
12 0.000 -12180
Displacements
i u_ix u_iy
1 0.000 0.000
2 -9.360e-4 2.216e-3
3 -2.403e-4 2.216e-3
4 -1.002e-3 2.879e-3
5 -4.806e-4 2.901e-3


Points
Trusses
Material
Loads
Supports
Results
Examples: Howe bridge
File
Reaction Forces
i F_ix F_iy
1 0.000 -12180
12 0.000 -12180
Displacements
i u_ix u_iy
1 0.000 0.000
2 -9.360e-4 2.216e-3
3 -2.403e-4 2.216e-3
4 -1.002e-3 2.879e-3
5 -4.806e-4 2.901e-3
6 -6.896e-4 2.774e-3
7 -6.896e-4 2.984e-3
8 -3.768e-4 2.879e-3
9 -8.986e-4 2.901e-3
10 -4.432e-4 2.216e-3
11 -1.139e-3 2.216e-3
12 -1.379e-3 0.000

3) Checks

- Total uplift: $3 \times 6.090 + 2 \times 3.045 = 24.36\ KN$
- Reactions (symmetric, downward):
  $R_{1y} = R_{12y} = -12.18$ KN the tool may show negative because they act downward).
- Force pattern reverses versus gravity:
  - Top chord goes mainly in tension (blue).
  - Bottom chord goes mainly in compression (red).

$$\text{ratio} = \frac{P_{WL}}{P_{DL}} = \frac{6.090}{2.851} \approx 2.14$$

Check at **Joint 1**:

- $F_{1\,-2}$(was -11.15kN in DL) → $\approx 23.9\ KN$ (tension)
- $F_{1\,-3}$(was $+9.58$kN in DL) → $\approx -20.5\ KN$(compression)

TASK 4:

Task 4 instructions:

Divide the truss into 4 categories as follows:

a. Bottom chord members

b. Top chord members

c. Vertical members

d. Diagonal members

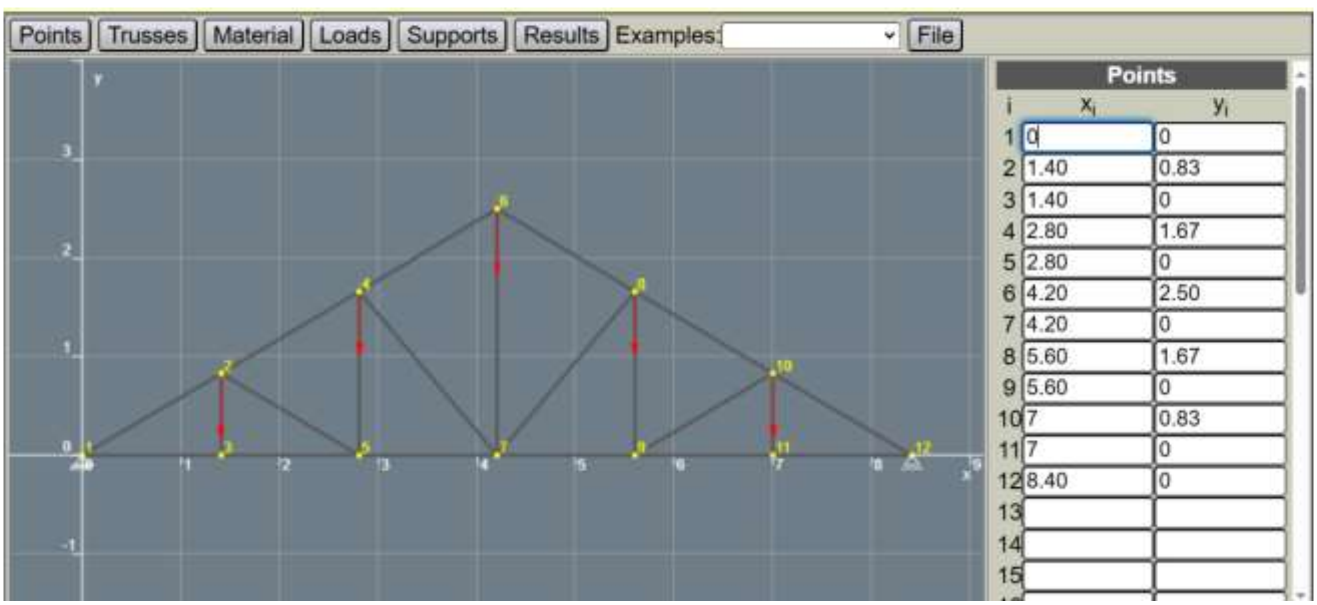
For each category,

calculate the maximum load (tension and compression) that could be sustained by the members considering the following cases:

LOAD CASE 1: 1.35 × DL + 1.5 × LL

LOAD CASE 2: 1.35 × DL + 1.5 × LL + 0.9 × WL LOAD

CASE 3: 1.00 × DL + 1.5 × WL



Bottom Members:

Member	DL	LL (=1.381·DL)	WL (=−2.138·DL)	Case 1	Case 2	Case 3	Max Tension (+)	Max Compression (−)
1	9.62	13.28	−20.56	32.90	14.40	−21.22	32.90	−21.22
2	9.62	13.28	−20.56	32.90	14.40	−21.22	32.90	−21.22
3	8.37	11.55	−17.88	28.62	12.53	−18.46	28.62	−18.46
4	8.37	11.55	−17.88	28.62	12.53	−18.46	28.62	−18.46
5	9.62	13.28	−20.56	32.90	14.40	−21.22	32.90	−21.22
6	9.62	13.28	−20.56	32.90	14.40	−21.22	32.90	−21.22

Top Members:

Member	DL	LL (=1.381·DL)	WL (=−2.138·DL)	Case 1	Case 2	Case 3	Max Tension (+)	Max Compression (−)
7	−11.18	−15.44	+23.90	−38.25	−16.74	+24.67	+24.67	−38.25
8	−9.76	−13.47	+20.86	−33.38	−14.61	+21.53	+21.53	−33.38
9	−7.42	−10.25	+15.87	−25.40	−11.12	+16.38	+16.38	−25.40
10	−7.42	−10.25	+15.87	−25.40	−11.12	+16.38	+16.38	−25.40
11	−9.76	−13.47	+20.86	−33.38	−14.61	+21.53	+21.53	−33.38
12	−11.18	−15.44	+23.90	−38.25	−16.74	+24.67	+24.67	−38.25

Vertical members

Member	DL	LL (=1.381·DL)	WL (=−2.138·DL)	Case 1	Case 2	Case 3	Max Tension (+)	Max Compression (−)
13	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
14	0.742	1.025	−1.586	2.539	1.112	−1.637	2.539	−1.637
15	4.721	6.521	−10.088	16.155	7.075	−10.411	16.155	−10.411
16	0.742	1.025	−1.586	2.539	1.112	−1.637	2.539	−1.637
17	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Diagonal members

Member	DL	LL (=1.381·DL)	WL (=−2.138·DL)	Case 1	Case 2	Case 3	Max Tension (+)	Max Compression (−)
18	−3.48	−4.81	+7.45	−11.92	−5.22	+7.69	+7.69	−11.92
19	−4.73	−6.53	+10.11	−16.18	−7.08	+10.44	+10.44	−16.18
20	−4.73	−6.53	+10.11	−16.18	−7.08	+10.44	+10.44	−16.18
21	−3.48	−4.81	+7.45	−11.92	−5.22	+7.69	+7.69	−11.92

Task 5:

(a) Design for tension (EC3) First, assume a section (you could assume double steel angles). Second, check that the following applies: A steel element subjected to tension must satisfy the following expression:

$$\frac{N_T}{N_{t,Rd}} \leq 1.0$$

Where:

$N_T, is\ the\ maximum\ tensile\ force\ from\ the\ load\ cases.$

$N_{t,Rd}$ is the lesser value of $\frac{0.65\ A_{gross} * F_y}{1.0}$ and $\frac{0.9 * 0.65 * A_{gross} * F_u}{1.25}$

the assumed section satisfies the expression, then the section is ok, otherwise change the section to a bigger one until the section is satisfied.

(b) Design for compression (EC3)

For the section chosen above, check that the following applies: A steel element subjected to Compression must satisfy the following expression:

$$\frac{N_C}{N_{t,Rd}} \leq 1.0$$

Where:

$N_C, is\ the\ maximum\ compressive force\ from\ the\ load\ cases.$

$N_{c,Rd}$ is the lesser value of $\frac{A_{gross} * F_y}{1.0}$

(c) Design for buckling (EC3) For the section chosen above, check that the following applies: A steel element subjected to compression must satisfy the following expression:

$$\frac{N_C}{N_{t,Rd}} \leq 1.0$$

Where:

$N_C, is\ the\ maximum\ compressive force\ from\ the\ load\ cases.$

$N_{c,Rd}$ is the lesser value of $\frac{\alpha * A_{gross} * F_y}{1.0}$

(d) Check for deflection (EC3) Allowable deflection for roofs (Table 4.1 ENV 1993-1-1:1992) = L/250 for the LL case only. Therefore, deflection for LL case needs to be less than L/250.

Task 5 - Bottom Chord Members

Member length (panel): $L = c = 1400$ mm
Steel: $f_y = 275$ N/mm^2,

$f_u = 43$ N/mm^2
Partial factors: $\gamma_{M0} = 1.0$, $\gamma_{M1} = 1.0$, $\gamma_{M2} = 1.25$
Trial chord section (example values, OK for checking):
Gross area $A_g = 2800\ m^2$
Radius of gyration about weak axis $i = 19.4$ mm
Elastic modulus $E = 210\,000$ N/mm^2

Governing forces (from Task 4)
Maximum tension $N_{T,Ed} = 32.90$ kN
Maximum compression $N_{C,Ed} = 21.22$ kN

A) Design in tension

Ultimate tensile resistance per EC3:

$$N_{t,Rd} = \min\left(\frac{A_g f_y}{\gamma_{M0}}, \frac{0.9\, A_{net}\, f_u}{\gamma_{M2}}\right)$$

Assuming $A_{net} \approx A_g$ for the member check (connection dealt with later):

$$\frac{A_g f_y}{\gamma_{M0}} = \frac{2800 \times 275}{1.0} = 770\,000\ \text{N} = 770\ \text{kN}$$
$$\frac{0.9\, A_{net} f_u}{\gamma_{M2}} = \frac{0.9 \times 2800 \times 430}{1.25} = 866.9\ \text{kN}$$
$$\Rightarrow N_{t,Rd} = 770\ \text{kN}$$

Utilisation:

$$\frac{N_{T,Ed}}{N_{t,Rd}} = \frac{32.90}{770} = 0.043 \ < 1.0)$$

B) Design in compression (plastic resistance, no buckling)

$$N_{c,Rd} = \frac{A_g f_y}{\gamma_{M0}} = 770 \text{ kN}$$

$$\frac{N_{C,Ed}}{N_{c,Rd}} = \frac{21.22}{770} = 0.028 < 1.0)$$

C) Buckling check (EC3, member in compression)

Slenderness:

$$\lambda = \frac{L}{i} = \frac{1400}{19.4} = 72.2$$

Non-dimensional slenderness:

$$\bar{\lambda} = \frac{L}{i}\sqrt{\frac{f_y}{\pi^2 E}} = 72.2 \times \sqrt{\frac{275}{\pi^2 \times 210000}} = 0.832$$

For angles/built-up chords, adopt buckling curve c ($\alpha = 0.49$):

$$\phi = \frac{1}{2}\,[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] = \frac{1}{2}\,[1 + 0.49(0.632) + 0.832^2] = 1.001$$

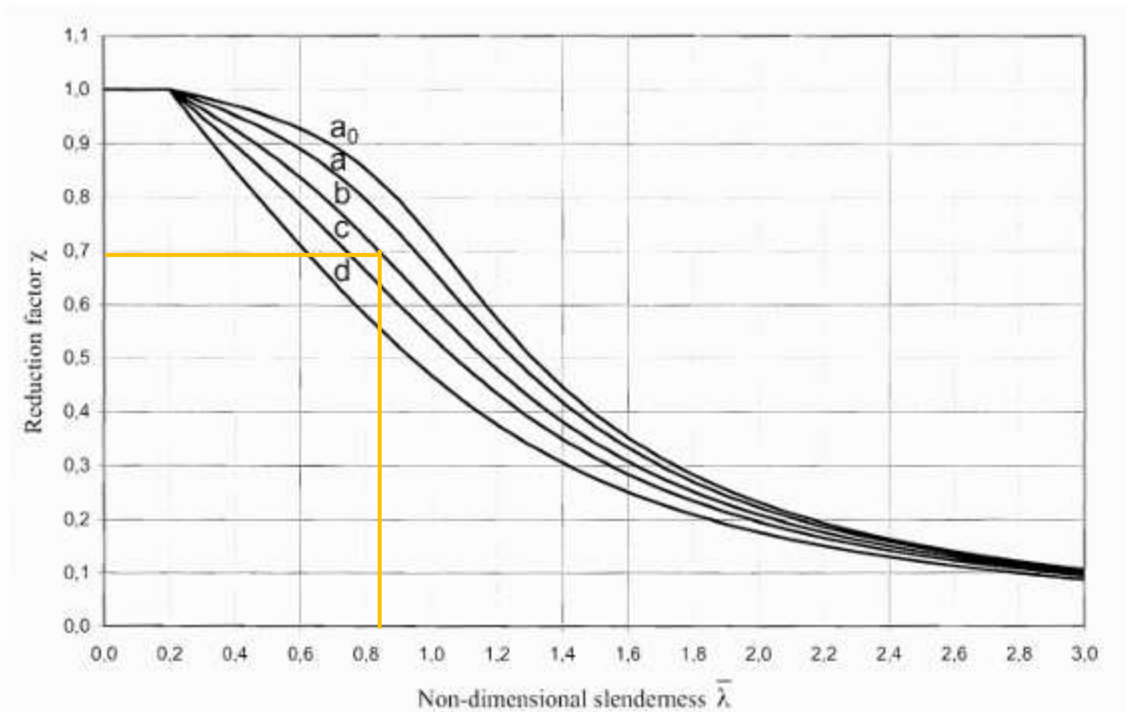
$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{1.001 + \sqrt{1.001^2 - 0.832^2}} = 0.642$$

Buckling resistance:

$$N_{b,Rd} = \frac{\chi\, A_g f_y}{\gamma_{M1}} = \frac{0.642 \times 2800 \times 275}{1.0} = 494 \text{ kN}$$

Utilisation:

$$\frac{N_{C,Ed}}{N_{b,Rd}} = \frac{21.22}{494} = 0.043\ (< 1.0)$$



Top Chord Members

$$L = \sqrt{c^2 + (\frac{b}{3})^2} = \sqrt{1.40^2 + 0.833^2} = 1.629 \text{ m} = 1629 \text{ mm}$$

Steel grade: $f_y = 275 \text{ N}/mm^2$,, $f_u = 430 \text{ N}/mm^2$,
Trial section (same as bottom chord for illustration): $A_g = 2800\ mm^2$,

radius of gyration $i = 19.4$ mm
Partial factors: $\gamma_{M0} = 1.0$,

$\gamma_{M1} = 1.0$,

$\gamma_{M2} = 1.25$

Governing forces from Task 4 (Top chord):

- Maximum compression: 38.25 kN

  (members 7 & 12)

- Maximum tension: 24.67 kN

  (members 7 & 12)

a) Design in tension

Ultimate tensile resistance (EC3):

$$N_{t,Rd} = \min\ (\frac{A_g f_y}{\gamma_{M0}}, \frac{0.9\ A_{net} f_u}{\gamma_{M2}})$$

Take $A_{net} \approx A_g$ for member check (connections checked later):

$$\frac{A_g f_y}{\gamma_{M0}} = \frac{2800 \times 275}{1.0} = 770\,000 \text{ N} = 770 \text{ kN}$$
$$\frac{0.9\ A_{net} f_u}{\gamma_{M2}} = \frac{0.9 \times 2800 \times 430}{1.25} = 866.9 \text{ kN}$$
$$N_{t,Rd} = 770 \text{ kN}$$

Utilisation:

$$\frac{N_{T,Ed}}{N_{t,Rd}} = \frac{24.67}{770} = 0.032 < 1.0$$

b) Design in compression (plastic resistance)

$$N_{c,Rd} = \frac{A_g f_y}{\gamma_{M0}} = 770 \text{ kN}$$
$$\frac{N_{C,Ed}}{N_{c,Rd}} = \frac{38.25}{770} = 0.050 < 1.0$$

c) Buckling check (EC3)

Slenderness:

$$\lambda = \frac{L}{i} = \frac{1629}{19.4} = 83.96$$

Non-dimensional slenderness:

$$\bar{\lambda} = \frac{L}{i}\sqrt{\frac{f_y}{\pi^2 E}} = 83.96\sqrt{\frac{275}{\pi^2 \times 210000}} = 0.967$$

Adopt buckling curve c for angles/chords ($\alpha = 0.49$):

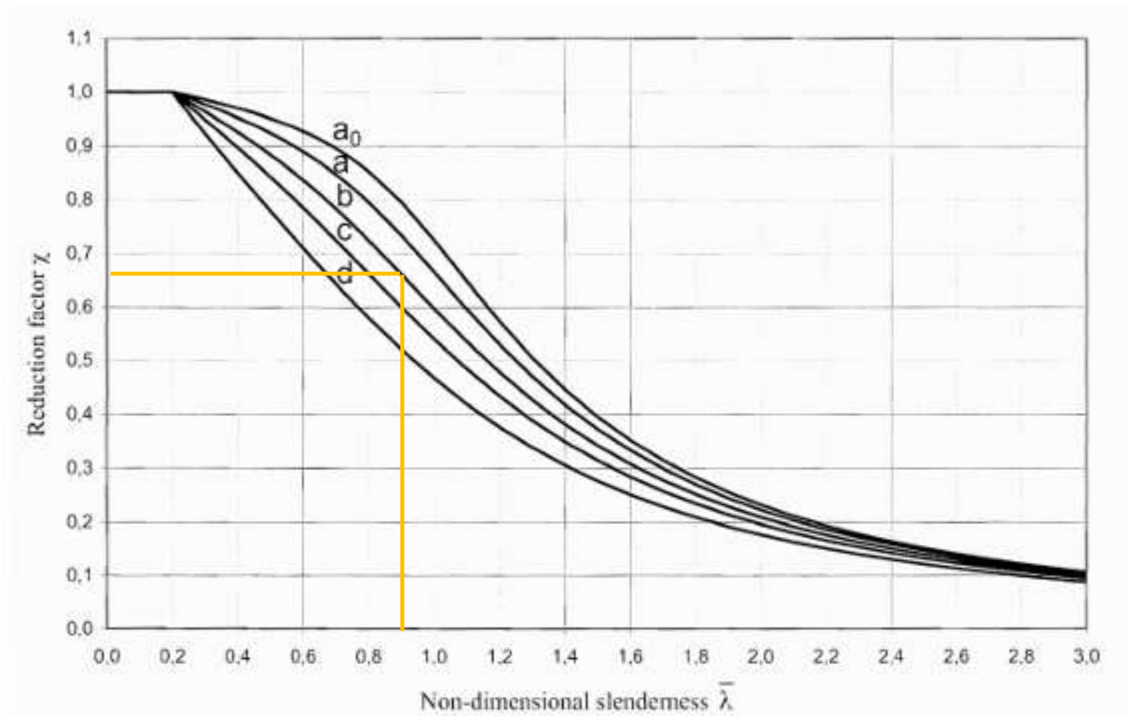
$$\phi = \frac{1}{2}\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] = \frac{1}{2}\left[1 + 0.49(0.767) + 0.967^2\right] = 1.155$$
$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{1.155 + \sqrt{1.155^2 - 0.967^2}} = 0.559$$

Buckling resistance:

$$N_{b,Rd} = \frac{\chi A_g f_y}{\gamma_{M1}} = \frac{0.559 \times 2800 \times 275}{1.0} = 430 \text{ kN}$$

Utilisation:

$$\frac{N_{C,Ed}}{N_{b,Rd}} = \frac{38.25}{430} = 0.089 < 1.0$$



Vertical Chord Members

Vertical lengths (clear panel heights)

- $2–3 = 0.833$ m $= 833$ mm
- $4–5 = 1.667$ m $= 1667$ mm
- $6–7 = 2.50$ m $= 2500$ mm$\longleftarrow$ longest $\rightarrow$ governs
- $8–9 = 1.667$ m
- $10–11 = 0.833$ m

Steel: S275 $\rightarrow f_y = 275$ N/mm^2, $f_u 430$ N/mm^2
Partial factors: $\gamma_{M0} = \gamma_{M1} = 1.0, \gamma_{M2} = 1.25$
Elastic modulus: $E = 210{,}000$ N/mm^2

Governing design actions from Task 4 (verticals):

- Members 13 & 17: $N_{Ed} = 0$
- Members 14 & 16: $+2.539$ kN(T) and -1.637 kN(C)
- Member 15 (centre, 6–7): $+16.155$ kN(T) and -10.411 kN(C)

A) Design in tension

$$N_{t,Rd} = \min\ (\frac{A_g f_y}{\gamma_{M0}}, \frac{0.9\ A_{net} f_u}{\gamma_{M2}})$$

Assume $A_{net} \approx A_g$ for the member check (connection checked later):

$$\frac{A_g f_y}{\gamma_{M0}} = 469 \times 275 = 129 \text{ kN}$$
$$\frac{0.9\ A_{net} f_u}{\gamma_{M2}} = \frac{0.9 \times 469 \times 430}{1.25} = 145 \text{ kN}$$
$$N_{t,Rd} = 129 \text{ kN}$$

$$\Rightarrow \frac{N_{T,Ed}}{N_{t,Rd}} = \frac{16.155}{129} = 0.125 < 1.0$$

B) Design in compression (plastic resistance, no buckling)

$$N_{c,Rd} = \frac{A_g f_y}{\gamma_{M0}} = 129 \text{ kN} \Rightarrow \frac{N_{C,Ed}}{N_{c,Rd}} = \frac{10.411}{129} = 0.081 < 1.0$$

C) Buckling check (EC3, member 15 governs)

Slenderness:

$$\lambda = \frac{L}{i} = \frac{2500}{14.7} = 170$$

Non-dimensional slenderness:

$$\bar{\lambda} = \frac{L}{i}\sqrt{\frac{f_y}{\pi^2 E}} = 170 * \sqrt{\frac{275}{\pi^2 \times 210000}} = 1.96$$

Adopt buckling curve c for angles ($\alpha = 0.49$):

$$\phi = \frac{1}{2}[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] = \frac{1}{2}[1 + 0.49(1.76) + 1.96^2] = 2.85$$

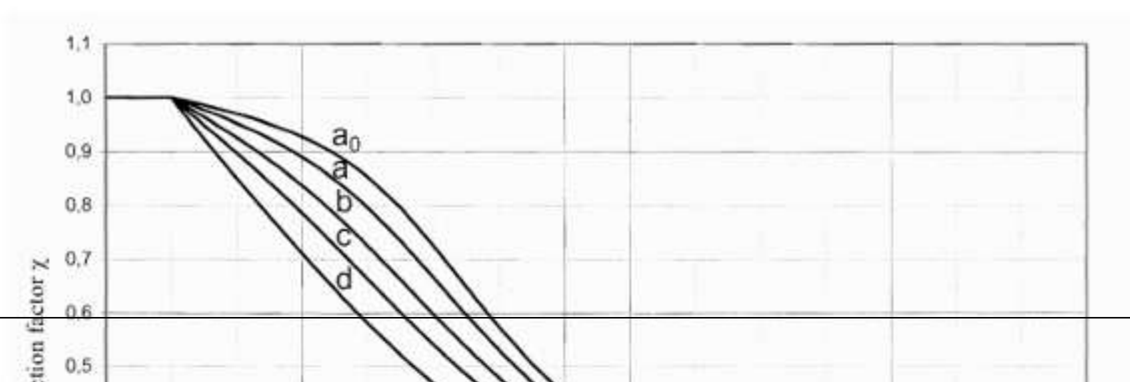
$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{2.85 + \sqrt{2.85^2 - 1.96^2}} = 0.203$$

Buckling resistance:

$$N_{b,Rd} = \frac{\chi A_g f_y}{\gamma_{M1}} = \frac{0.203 \times 469 \times 275}{1.0} = 26.2 \text{ kN}$$

Utilisation:

$$\frac{N_{C,Ed}}{N_{b,Rd}} = \frac{10.411}{26.2} = 0.40 < 1.0$$



Diagonal Chord Members

- Short diagonals (e.g., 3–4, 7–8, 9–10, 11–10):

$$L_s = \sqrt{c^2 + (2b/3)^2} = \sqrt{1.40^2 + 1.667^2} = 2.179 \text{ m} = 2179 \text{ mm}$$

- Long diagonals to apex (like, 5–6, 7–6):

$$L_\ell = \sqrt{c^2 + b^2} = \sqrt{1.40^2 + 2.50^2} = 2.865 \text{ m} = 2865 \text{ mm}$$

Steel: S275 → $f_y = 275 \text{ N}/mm^2$, $f_u = 430 \text{ N}/mm^2$

Elastic modulus: $E = 210{,}000 \text{ N}/mm^2$
Partial factors: $\gamma_{M0} = \gamma_{M1} = 1.0$, $\gamma_{M2} = 1.25$

Governing forces from Task 4 (kN):

- Short diagonals (Members 18 & 21):
  $N_{Ed}^{\max T} = +7.69$, $N_{Ed}^{\max C} = -11.92$
- Long diagonals (Members 19 & 20):
  $N_{Ed}^{\max T} = +10.44$, $N_{Ed}^{\max C} = -16.18$

$A_g = 469 \text{ mm}^2$, $i_{\min} = 14.7$ mm.

A) Design in tension

$$N_{t,Rd} = \min\left(\frac{A_g f_y}{\gamma_{M0}}, \frac{0.9\, A_{net} f_u}{\gamma_{M2}}\right)$$

Taking $A_{net} \approx A_g$ for member capacity (connections checked later):

$$\frac{A_g f_y}{\gamma_{M0}} = 469 \times 275 = 129 \text{ kN}$$

$$\frac{0.9 A_{net} f_u}{\gamma_{M2}} = \frac{0.9 \times 469 \times 430}{1.25} = 145 \text{ kN}$$

$$\Rightarrow N_{t,Rd} = 129 \text{ kN}$$

Checks:

- Short diagonal: $7.69/129 = 0.06 < 1.0$
- Long diagonal: $10.44/129 = 0.08 < 1.0$

B) Plastic compression resistance (no buckling)

$$N_{c,Rd} = \frac{A_g f_y}{\gamma_{M0}} = 129 \text{ kN}$$

- Short: $| -11.92 |/129 = 0.09 < 1.0$
- Long: $| -16.18 |/129 = 0.13 < 1.0$

C) Buckling (EC3 curve c, angles; $\alpha = 0.49$)

Use $\bar{\lambda} = \frac{L/i}{\pi}\sqrt{\frac{f_y}{E}}$.

1) Short diagonal $L_s = 2179$ mm

$$\lambda = \frac{L_s}{i} = \frac{2179}{14.7} = 148.3, \bar{\lambda} = 148.3\sqrt{\frac{275}{\pi^2 \cdot 210000}} = 1.71$$

$$\phi = \frac{1}{2}[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] = \frac{1}{2}[1 + 0.49(1.51) + 1.71^2] = 2.33$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{2.33 + \sqrt{2.33^2 - 1.71^2}} = 0.256$$

$$N_{b,Rd} = \frac{\chi A_g f_y}{\gamma_{M1}} = 0.256 \times 469 \times 275 = 33.0 \text{ kN}$$

$$\frac{| N_{C,Ed} |}{N_{b,Rd}} = \frac{11.92}{33.0} = 0.36 < 1.0$$

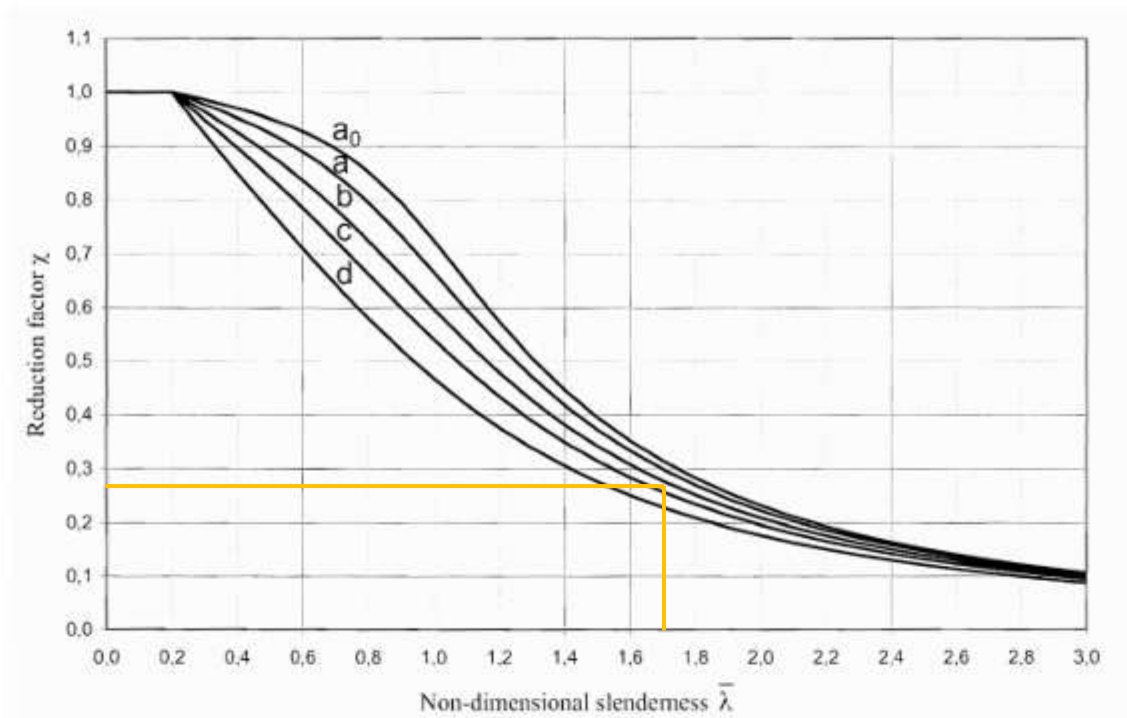
2) Long diagonal $L_\ell = 2865$ mm

$$\lambda = \frac{2865}{14.7} = 195.0, \bar{\lambda} = 195.0\sqrt{\frac{275}{\pi^2 \cdot 210000}} = 2.25$$

$$\phi = \frac{1}{2}[1 + 0.49(2.05) + 2.25^2] = 3.52, \chi = \frac{1}{3.52 + \sqrt{3.52^2 - 2.25^2}} = 0.160$$

$$N_{b,Rd} = 0.160 \times 469 \times 275 = 20.6 \text{ kN}$$

$$\frac{|\,N_{C,Ed}\,|}{N_{b,Rd}} = \frac{16.18}{20.6} = 0.79 < 1.0$$



Checking for deflection

- Total (DL+LL) vertical deflection: L/200–L/250
- Imposed load only (LL): L/300–L/360
- Under wind (uplift): often L/200–L/250 (absolute value)

Your span $L = 8.40$ m $= 8400$ mm:

- $L/200 = 42.0$ mm
- $L/250 = 33.6$ mm
- $L/300 = 28.0$ mm
- $L/360 = 23.3$ mm
- $r_{LL} = 1.381$, so $w_{LL} = 1.381\, w_{DL}$
- $r_{WL} = 2.138$, so $w_{WL} = -2.138\, w_{DL}$