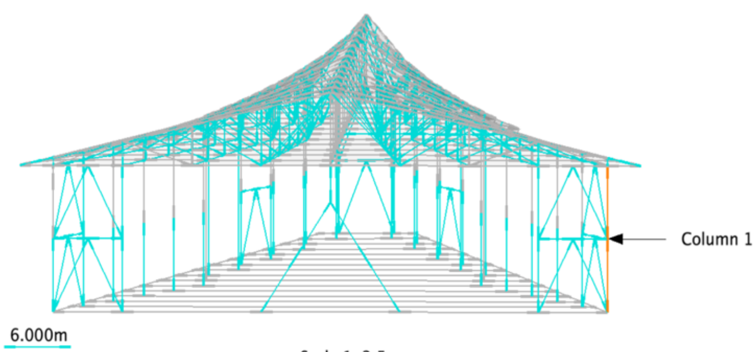
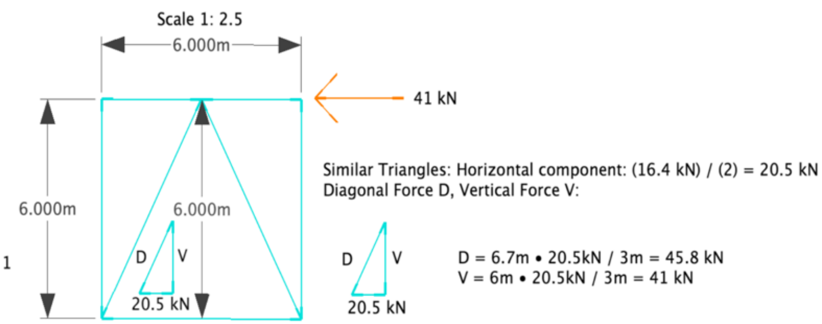
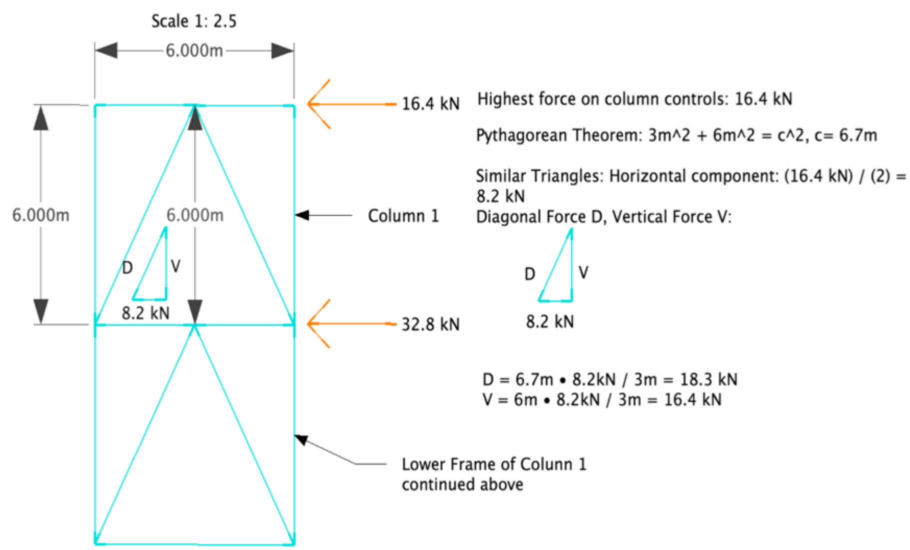
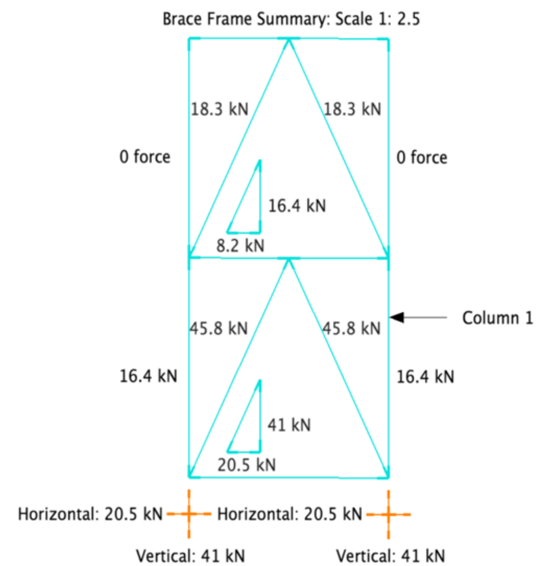
Southeast Elevation; parallel projection; Lateral loads on Column 1, Brace-Frame Design:









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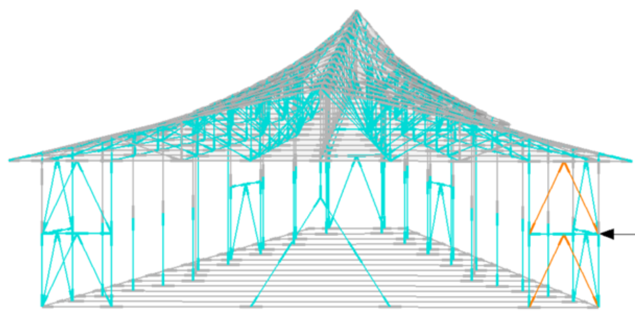
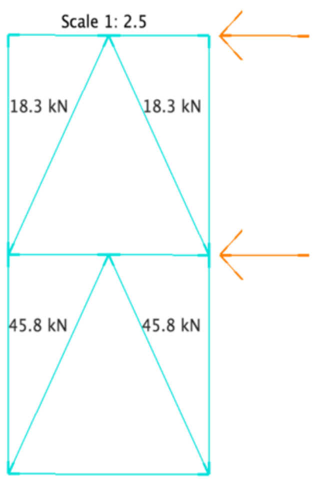
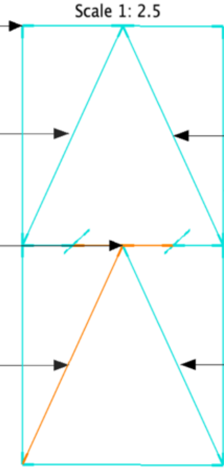
Southeast Elevation; parallel projection; Brace-Frame Design continued:







DESIGN all Struts and Ties to match Brace D; 6.7m in length
@ 48.35 kN in Compression:

Euler Load: PE = (π^2)(E)(I) / (Le)^2 [Cobb 65] Therefore:
Design for Second Moment of Area: I =
PE • (Le)^2 / (π^2)(E) Where: PE = 48.35 kN, L = 670 cm, e = 1, E = 21,000 kN/cm^2
Therefore: I = (48.35 kN • 448,900 cm^2) / (9.87 • 21,000 kN/cm^2) = 105 cm^4
Equal Angle Section: 90 x 90 x 12; I = 149 cm^2

Equal Angle, y-y axis for 90 x 90 x 12, Radius of gyration, ry =
2.71 cm which satisfies Le/ry: 670cm/2.71 =
2.47m < 6.7m, Therefore:

Design Equal Angle Section: 90 x 90 x 12 for Tie A, Strut B, Tie C, Strut D
[Cobb 164]

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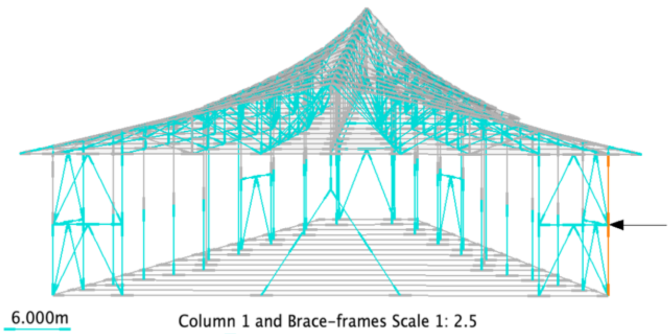
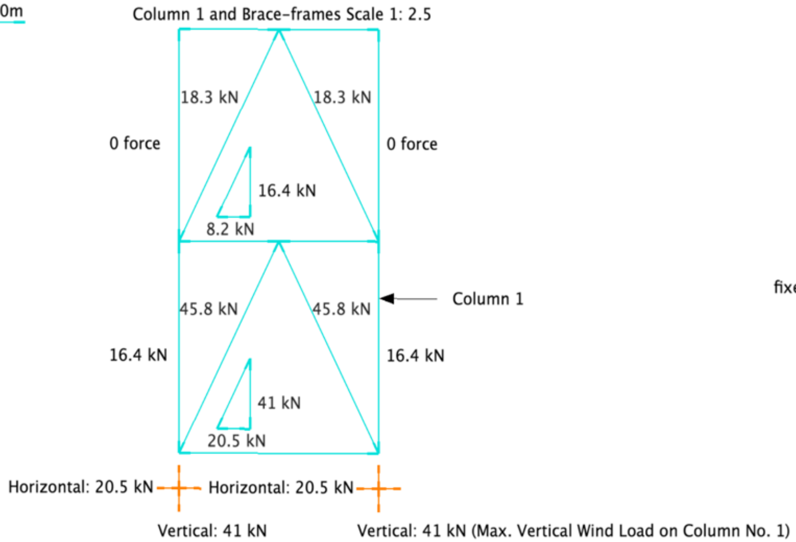
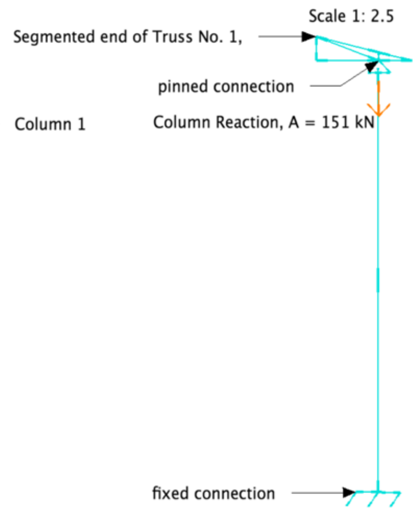
Southeast Elevation; parallel projection; Column No. 1 Design:







Tributary area on Column No. 1: 18 m^2 Therefore:

Dead-Loads: 4 kN/m^2 + Live-loads 1kN/m^2
• 3m • 6m = 90 kN + Max. Wind Load
in the Vertical direction resulting from brace frames =
41 kN = 131 kN + self-weight of member (20 kN) =

151 kN for Column No. 1:

Reaction A = F (applied force)

F/Pc + Mx/Mbs must be ≤ 1 (Load eccentricity for "beam" A only) [Arya190]

However Column No. 2 (highest loading) Controls "Group 1" for Column Design) with 770 kN, As calculated on the next page: Therefore:

Design of Column No. 1: UKC Sections 305 x 305 x 97 [Cobb 158]

Note:

Column loads have been calculated under worst case Dead + Live-Loads only scenario without Lateral-Loads: Uplift from any Duopitch roof negative wind-pressures in this case reduce the gravitational vertical loads on the Columns themselves.

Base Column reactions have also been checked for Overturning Moments where: Dead-Loads only on each Column combined with Lateral-Loads; uplift from negative wind-pressure > vertical reactions @ the base of the Columns, therefore system is stable.

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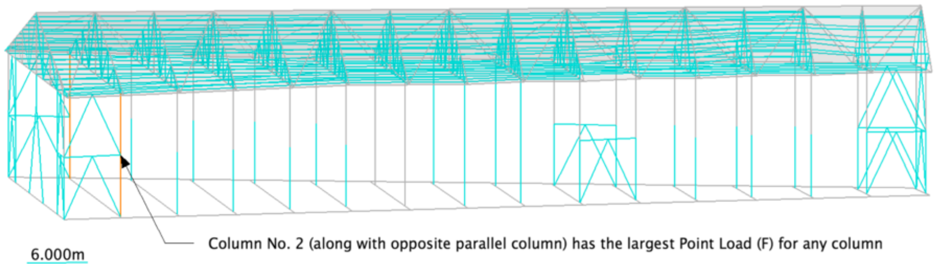
Northeast Elevation; parallel projection; Column No. 2 Member Design:



Column No. 2 (along with opposite parallel column) has the largest Point Load (F) for any column

Tributary area on Column No. 2: 150 m^2 • 5 kN/m^2 = 750 kN
Where:
Dead-Load: 4 kN/m^2 + Snow-Load 1 kN/m^2 =
5 kN/m^2 + self-weight of member:
2, 6.000m 305 x 305 x 97 Sections + plates and bolts = 20 kN

F (applied force) = 770 kN for Column No. 2:

$F/P_c + M_x/M_{bs}$ must be ≤ 1 (Load eccentricity for "beam" A only) [Arya 190]. In this case:
$F/P_c + M_x/M_{bs} = .74 \leq 1$

Where:

F= 770000 N
ex = 254 mm
Le (e =.85; pinned/fixed) = 10200 mm
Pb = 262 N/mm^2
Mx = 19558 • 10^4 Nmm
Mbs = 41658 • 10^4 Nmm
Pc = 322600 N

Design of Column No. 2: UKC Sections 305 x 305 x 97
[Cobb 158]

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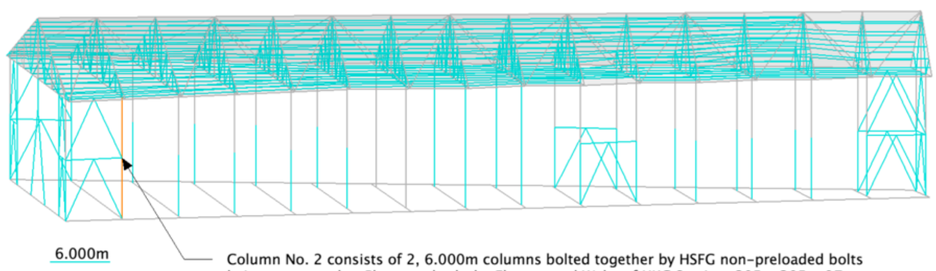
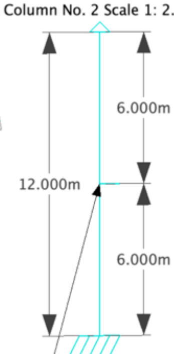
Connections: Northeast Elevation Parallel Projection for detailed Bolted Shear Connection Design on Column No. 2:



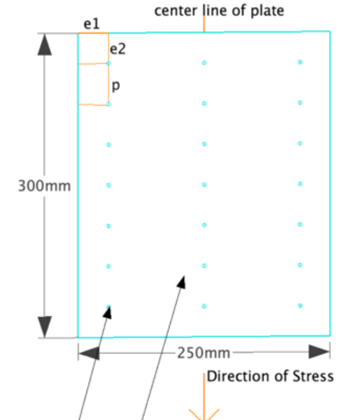
Column No. 2 consists of 2, 6.000m columns bolted together by HSFG non-preloaded bolts being connected to Plates on both the Flanges and Webs of UKC Section: 305 x 305 x 97

Column No. 2 Scale 1: 2.5



Location of the 4 connection plates

Column No. 2 Plan View of one connection plate
Scale 1:100



Bolt Design: S275 Steel, Grade 8.8 @ 16 DIA.

Plate Desgin: S275 Steel 300mm x 250mm @ 15mm thick for both the Web/Web connections and Flange/Flange Connections

For UKC Section: 305 x 305 x 97:
b= 305mm, Tf= 15.4mm Therefore:
hw = h (305mm) – (15.4 • 2) =
274mm, Therefore,
250mm Plate centered on Webs, 10mm off-center on Flanges staggered on both sides (to avoid bolting into the Webs)

Dh = db + 2mm for 16mm ≤ db ≤ 24mm =
18mm
p = 2.5db ≤ p ≤14t = 40mm
e1, e2 = ≥ 1.25Dh = 30mm

Where:
Dh = DIA. of holes
db = bolt DIA.
p = Spacing between bolts (pitch)
t = thickness of inner ply (Web @ 9.9mm)
e1 = minimum edge distance to hole center line
e2 = minimum end distance to hole center line
[Arya 219]

Bolt Connection Capacity check for combined tension and shear:
$f_t/P_t + f_s/P_s \leq 1.4$ [Cobb 241]
In this case shear controls:
$f_s/P_s \leq 1.4$

Where:
fs= applied shear @ Max 770kN
Ps= shear capacity using Grade 8.8 bolts @ 16 DIA:

Shear capacity
for each bolt is 29.4kN • 21 bolts =
617.4kN Therefore: 770kN/617.4kN =
1.25 ≤ 1.4, Capacity Check satisfied:
when thickness of the steel passed through > shearing capacity of the bolt
[Cobb 242]

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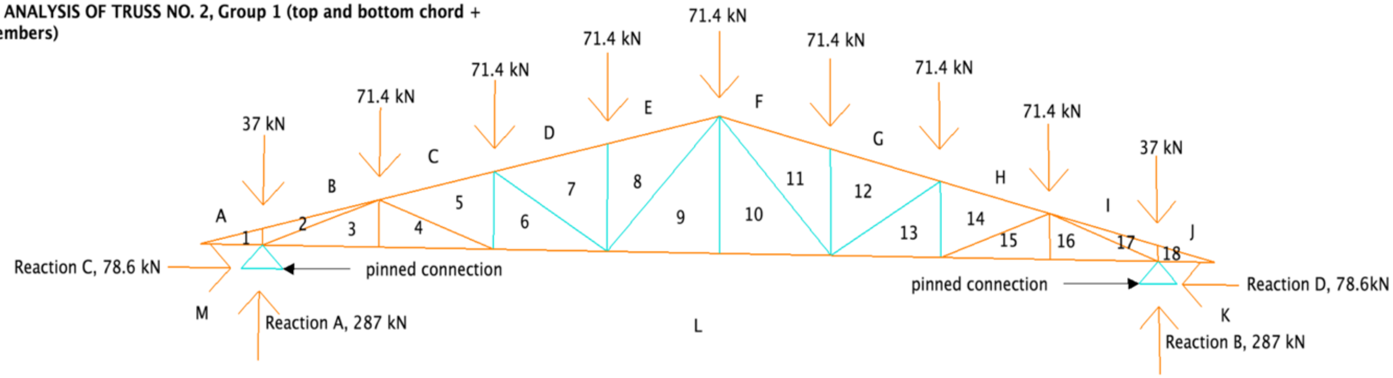
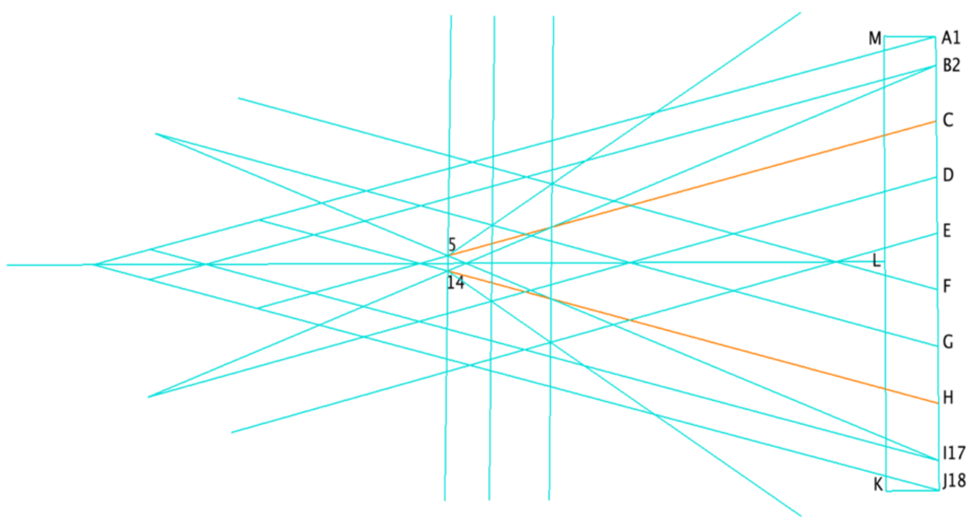
GRAPHICAL ANALYSIS OF TRUSS NO. 2, Group 1 (top and bottom chord + end web members)



Load Line Scale: 1m = 30 kN



Largest Magnitude, Chord Members:
C 5, H 14: 24.611m
Therefore: Design For
738.3 kN in Compression

F/Pc + Mx/Mbs must be ≤ 1 (Load eccentricity for "beam" A only) [Arya 190] In this case:
F/Pc + Mx/Mbs = .8 ≤ 1

Where:
F= 745000 N (with self-weight)
ex = 230.15 mm
Le (e = 1.0; pinned/pinned) = 5800 mm
Pb = 275 N/mm^2
Mx = 171461750 Nmm
Mbs = 3355 • 10^5 Nmm
Pc = 3107500 N

Design of Top Chord member C-5, H-14:
UKC Section 254 x 254 x 89

Therefore: Design of all Chord and Web members in Group 1 to: UKC Section 254 x 254 x 89
[Cobb 158]

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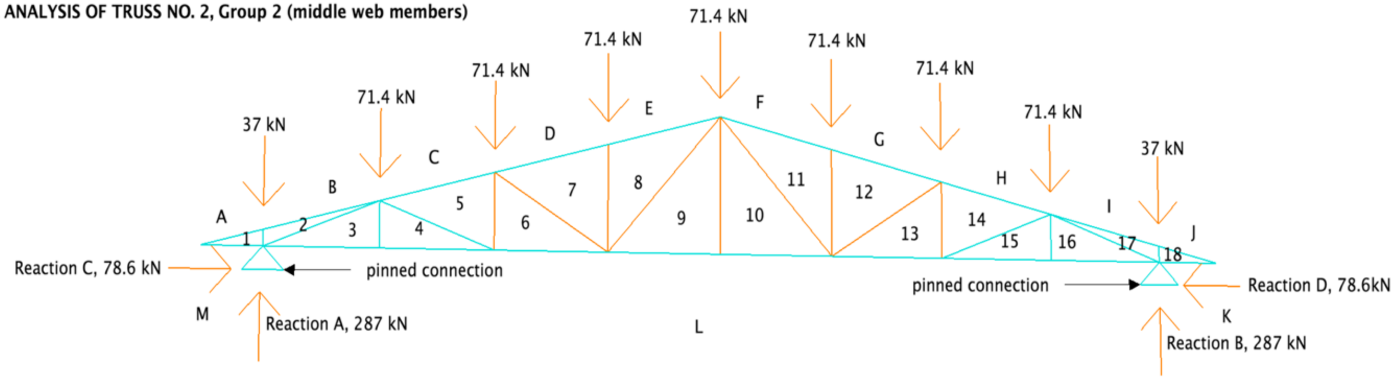
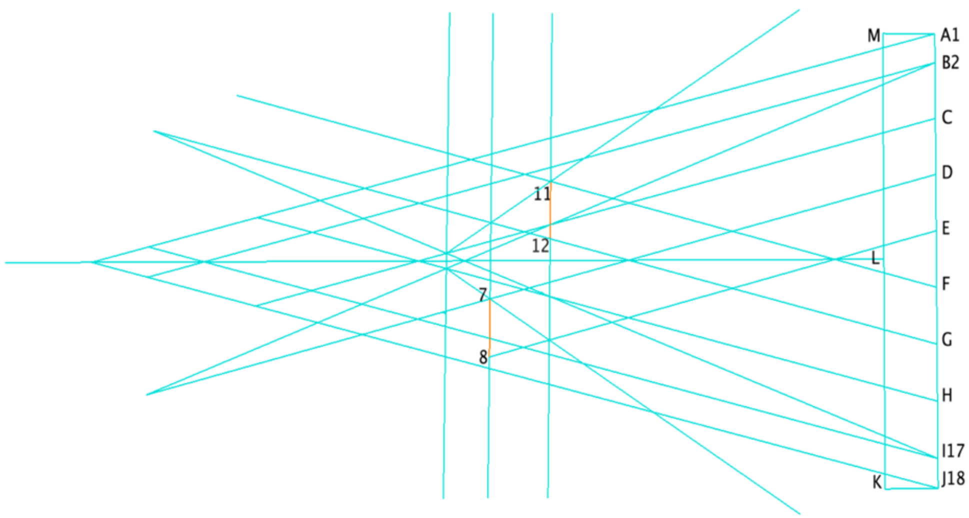
GRAPHICAL ANALYSIS OF TRUSS NO. 2, Group 2 (middle web members)



Load Line Scale: 1m = 30 kN



Largest Magnitude, Web member in Second Group: 7-8, 11-12: 2.380m
Therefore: Design For
71.4 kN in Compression

F/Pc + Mx/Mbs must be ≤ 1 (Load eccentricity for "beam" A only) [Arya 190] In this case:
F/Pc + Mx/Mbs = .8 ≤ 1

Where:
F= 73000 N (with self-wight)
ex = 201.6 mm
Le (e = 1.0; pinned/pinned) = 3400 mm
Pb = 275 N/mm^2
Mx = 14716800 Nmm
Mbs = 136675 • 10^3 Nmm
Pc = 1614250 N

Design of Web Members 7-8, 11-12:
UKC Section 203 x 203 x 46

Therefore: Design of all Web members in Group 2:
to: UKC Section 203 x 203 x 46
[Cobb 158]

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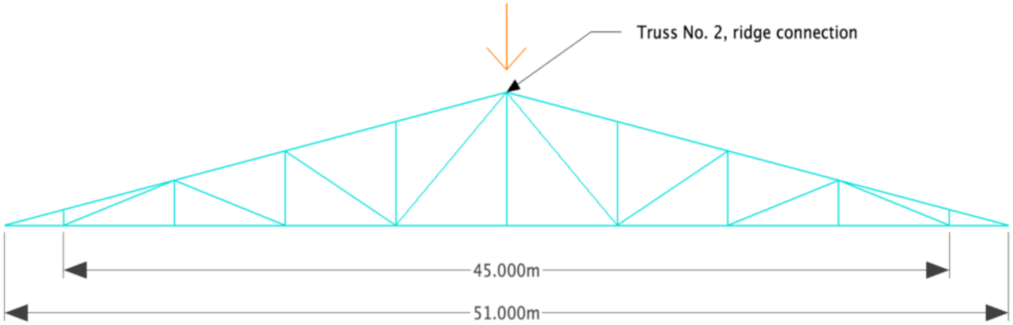
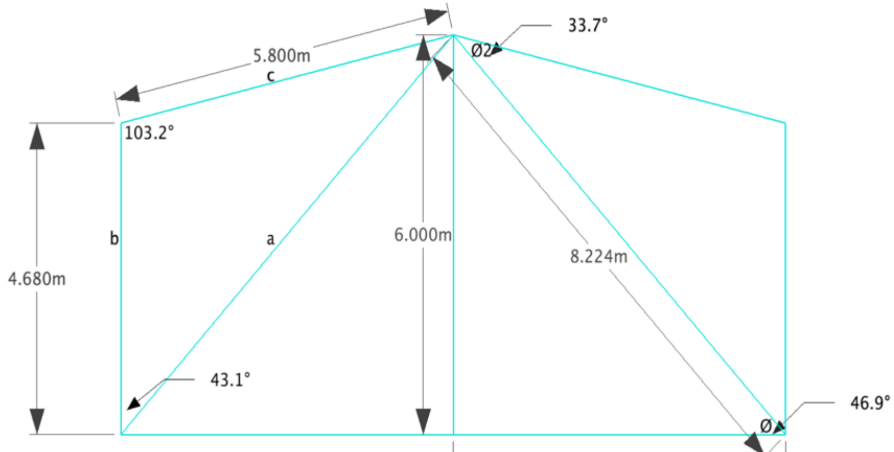
Truss No. 2 Connection



Scale 1: 3



Cos Ø = 5.620m / 8.224m = 46.9°

Law of Cosines for Ø2: a^2 + c^2 - b^2 / 2ac; Arccos = 33.7°

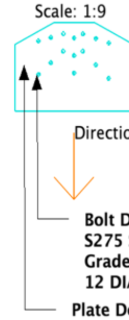
Therefore: 180° - 43.1° - 33.7° = 103.2°

For UKC Section: 254 x 254 x 89 (Truss Group 1):
b= 254mm (designing only for flange/plate connections)

For UKC Section: 203 x 203 x 46 (Truss Group 2):
b= 203mm (designing only for flange/plate connections)

Dh = db + 1mm for 12mm bolt = 13mm
p = 2.5db ≤ p ≤14t = 30mm to 50mm due to 33.7°
to satisfy p (pitch)
e1, e2 = ≥ 1.25Dh = 20mm
Where:
Dh = DIA. of holes
db = bolt DIA.
p = Spacing between bolts (pitch)
t = thickness of inner ply (Flange @ 11mm)
e1 = minimum edge distance to hole center line
e2 = minimum end distance to hole center line
[Arya 219]

Plan view of connection plate
Scale: 1:9



Direction of Stress

Bolt Design:
S275 Steel
Grade 8.8 @
12 DIA.

Plate Design:
S275 Steel
@ 750mm x
375mm x 315 mm
250 mm x 315 x
375 mm x 15mm
thick for
Flange/Plate
connections for
both members:
254 x 254 x 89
and
203 x 203 x 46

Bolt Connection Capacity check for combined tension and shear:
ft/Pt + fs/Ps ≤ 1.4 [Cobb 241]
In this case shear controls:
fs/Ps ≤ 1.4

Where:
fs= applied shear @ Max 71.4kN
Ps= shear capacity using Grade 8.8 bolts @ 12 DIA:

Shear capacity:
for each bolt is 15.8kN • 15 bolts = 237 kN Therefore: 74.1kN/237kN = .31 ≤ 1.4, Capacity Check satisfied: when thickness of the steel passed through > shearing capacity of the bolt [Cobb 242] avoiding connections along the center line of the Flanges (not to interfere with the Webs) for both UKC sections.

Connection note: The distribution of forces for the other unexamined connections:

Truss to Column due to in-factory fillet welds of rectangular plates to the top sections of columns, plus strut and tie to Truss and Column connections will be analyzed in further detail; eliminating unnecessary bending moments through center-line load-path connections wherever possible, while conducting capacity checks for both combined tension and shear where applicable.

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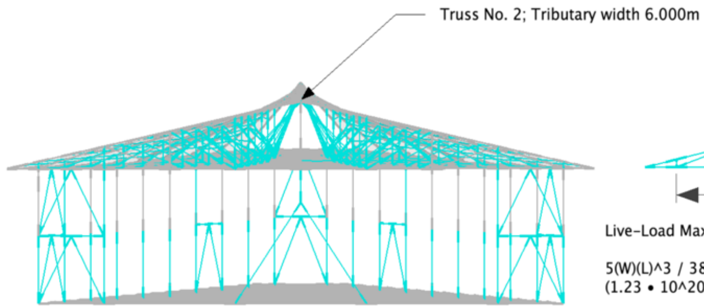
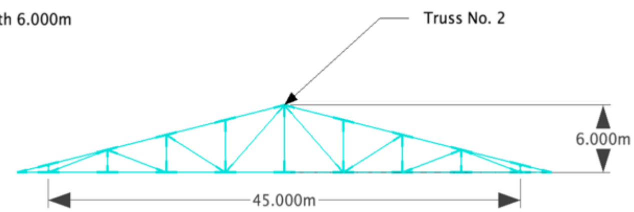
Vertical Deflection: Live-Load (snow-load @ 1kN/m^2) on Truss No. 2



6.000m



Live-Load Max. Deflection < span/360 [Cobb 58]

5(W)(L)^3 / 384(E)(I) =
(1.23 • 10^20) / (7.51 • 10^18) =

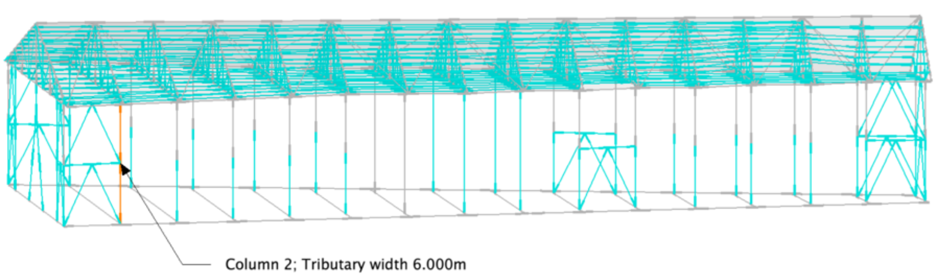
Where:
W = 270,000 N (1kN/m^2 • 6m • 45m)
L = 45,000mm
E = 210000 N/mm^2
I = 9.965 • 10^10mm^4

16.4mm < 125mm;
Deflection Satisfied

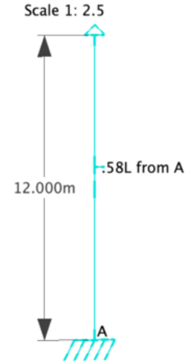
Determining the Second Moment of Area for the Truss No. 2:
Parallel Axis Theorem: I = ∑ A • y^2 + I local

Top and Bottom Chord members designed to UKC Section: 254 x 254 x 89
Where:
A = 11300mm^2
y = 3m (half the depth of the truss) – h(depth of section)/2
I local = 143 • 10^5
h = 260.3mm
Therefore y = 3m – (260.3/2) = 2870mm
I = ∑ (11,300mm^2) • (2870^2) + (143 • 10^5mm^4) = 9.31 • 10^10mm^4

Horizontal Deflection: for Lateral-Load (wind-load, Zone D: @ 1.8kN/m^2) on Column No. 2



6.000m



Live-Load Max. Deflection < height/300
[Cobb 61]
(W)(L)^3 / 185(E)(I) @ .58L from A =
2.25 • 10^17 / 8.63 • 10^15 =

Where:
W = 130000 N (1.8kN/m^2 • 6000mm • 12000mm)
L = 12000mm
E = 210000 N/mm^2
I = 222 • 10^6mm^2 (UKC 305 x 305 x 97)

26.1mm < 40mm;
Deflection Satisfied @ .58L from location A

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Geotechnical Design: Main Reference [Tomlinson, M. J and John Woodward. Pile Design And Construction Practice. 5th ed. 2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN: Taylor & Francis. Print.]

To be drilled by CFA method to mitigate neighborhood disruption:

Total Force = 770 kN on Column No. 2

Design for **Pile length 15m and 750mm in DIA. [Cobb 331]** (Terrace Gravels) Granular soil down to 15 meters in Lewisham, London, angle of repose: ø = 30°

Tan ø = .58° Vertical Stress p = soil density • gravity • d (depth of pile)

Where: soil density = 2000kg/m^3 • 9.81(g) • 15m(depth of pile) =

295 kN/m^2

u (water pressure) Where: Water = 1000 kg/m^3

water density = 1000 kg/m^3 • 9.81 • 15m = 147 kN/m^2

Effective Stress = soil density – water density = 148 kN/m^2, For Total Pile 148 kN/m^2: the average pressure (0 pressure at the top, 148 kN/m^2 @ the bottom = 148 kNm^2 / 2= 74 kN/m^2

74 kN/m^2 • .58 (Tan ø) • .7 (FOS) = 30 kN/m^2

Apply to Shaft area:

n = π • d • L = 3.14 • 750mm (or .75m) • 15m = 35.34m^2

Therefore Total shaft = 30kN/m^2 • 35.34m^2 = 1060 kN

Applied FOS = 2.5; therefore 1060kN / 2.5 = 424 kN

Shaft Resistance Satisfied [Cobb 331]

Base Capacity:

A = πd^2/4 (d = DIA @ .75m) = .4416m^2

qb = 10000 kN/m^2

Where Qb = base resistance: A • qb = 4416kN / 2.5 (FOS) = 1766 kN

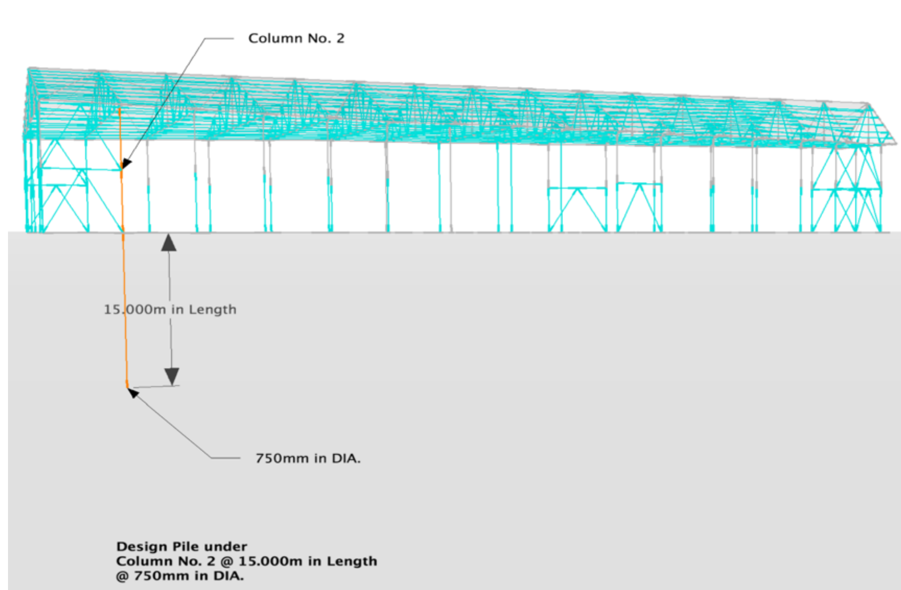
Base Resistance Satisfied [Cobb 331]

Pile Caps: In Plan, projects ≥ 150mm beyond the pile face; for pile DIA. @

750mm: pile cap depth will be 1600mm [Cobb 335].

NOTE: fck of concrete and steel rebar sizing for both the piles and the pile caps will be analyzed in further detail under [BS 8110].

Northeast Elevation; parallel projection; Geotechnical Design for column No. 2



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