Date: 12th March 2024
Revision – R1

DESIGN CALCULATION REPORT
FOR
GLAZED WALL
AT
CHILD CARE CENTRE,
1458 PACIFIC HIGHWAY,
TURRAMURRA, NSW 2074

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1. INTRODUCTION

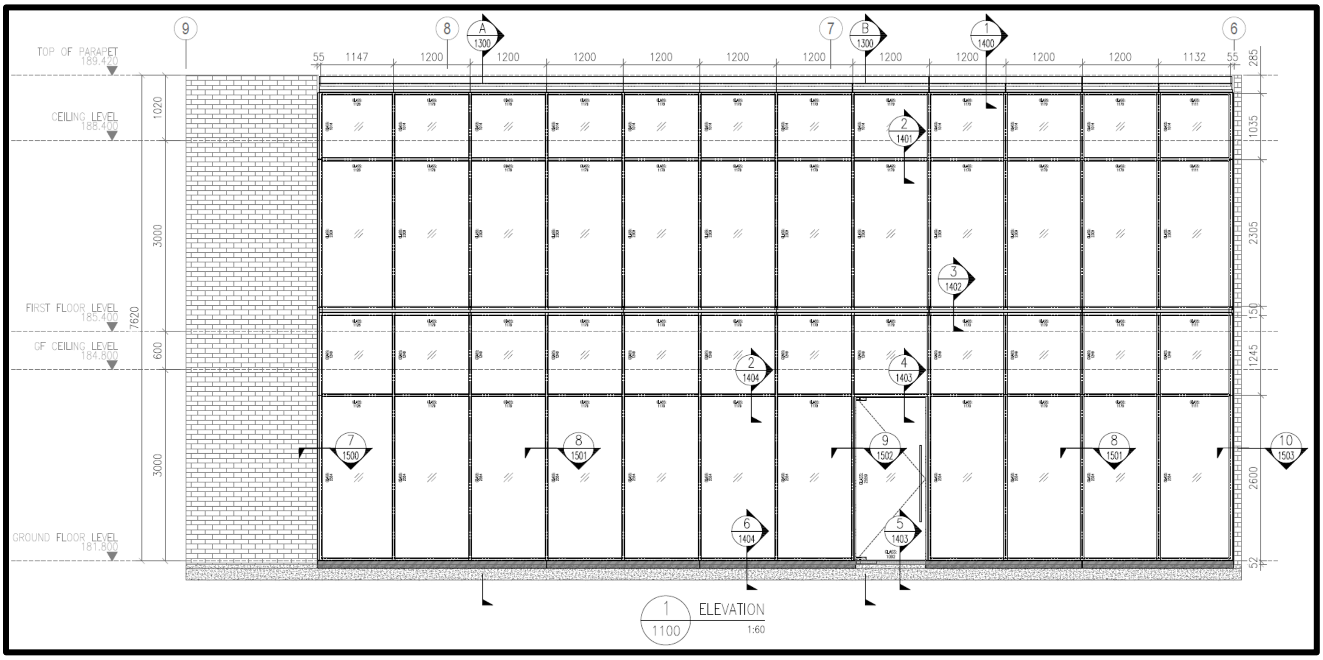
This design calculation is to justify the structural elements of Glazed Wall in the proposed Child Care Centre in Turramurra.

The facade system is designed to sustain the dead load, live load, earthquake load and wind load according to Structural design actions_ Wind actions as per AS/NZS 1170:2:2021.

The facade system will be fixed to parent concrete structure using post fixed anchors.

Load path for Glazed Wall

Load Path ⇨ Loading on Glass ⇨ Aluminum/Steel Frame ⇨ Fixings ⇨ Concrete Structure



Framing Elevation for Glazed Wall

2. MATERIAL

Sr. No.	Member	Remarks	Grade (MPa)
1	RHS_100x50x4	Horizontal Framing Main Member	Steel-350/450
2	RHS_200x100x5	Verticals/Mullions Main Frame	Steel-350/450
3	RHS_200x200x5	Horizontal Framing Member (TBC)	Steel-350/450
4	RHS_100x50x3.2	Horizontal Framing Secondary Member	Aluminium-110

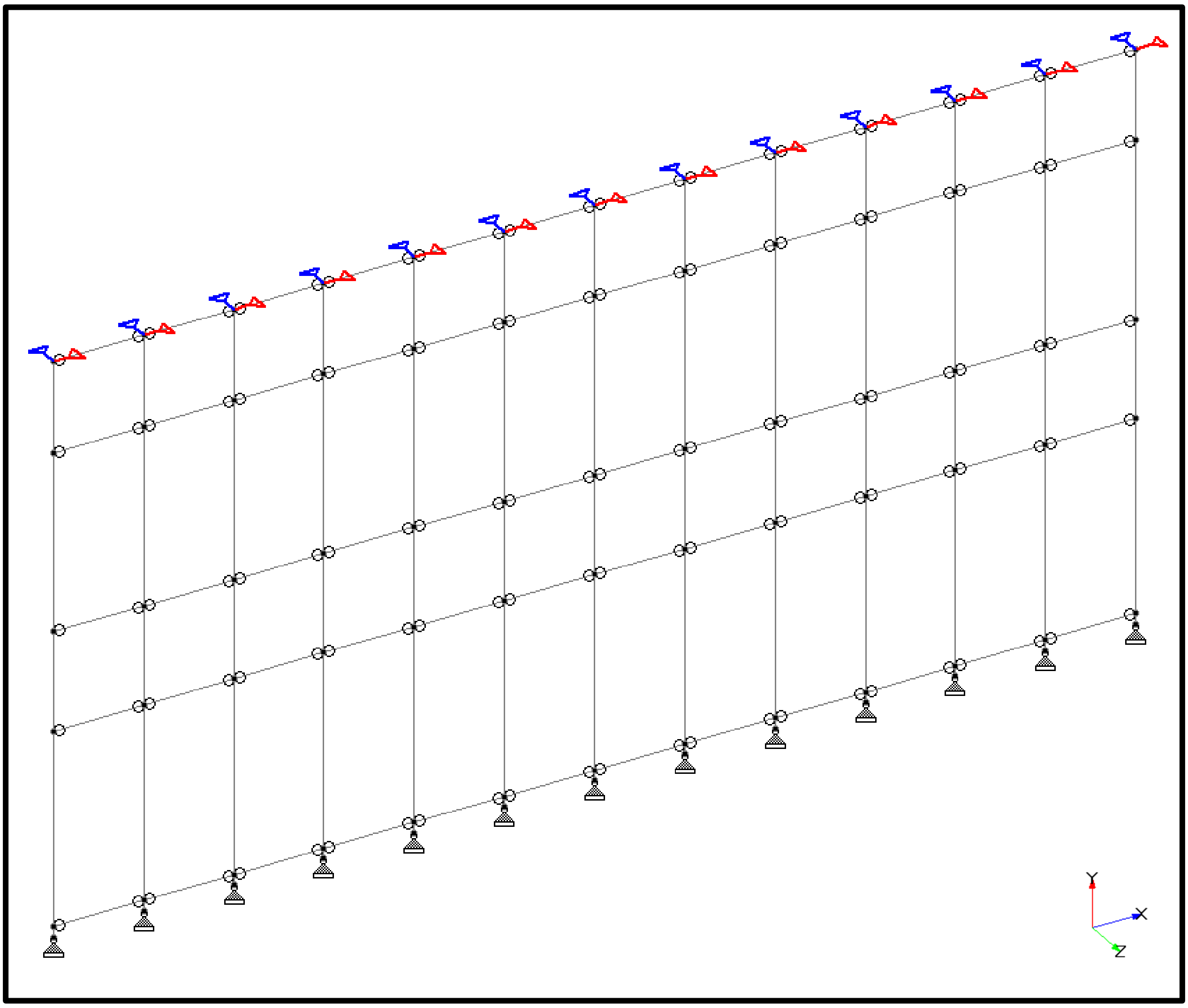
3. CODES CONSIDERED

Following codes are referred for analysis and design of Glazed Wall structure.

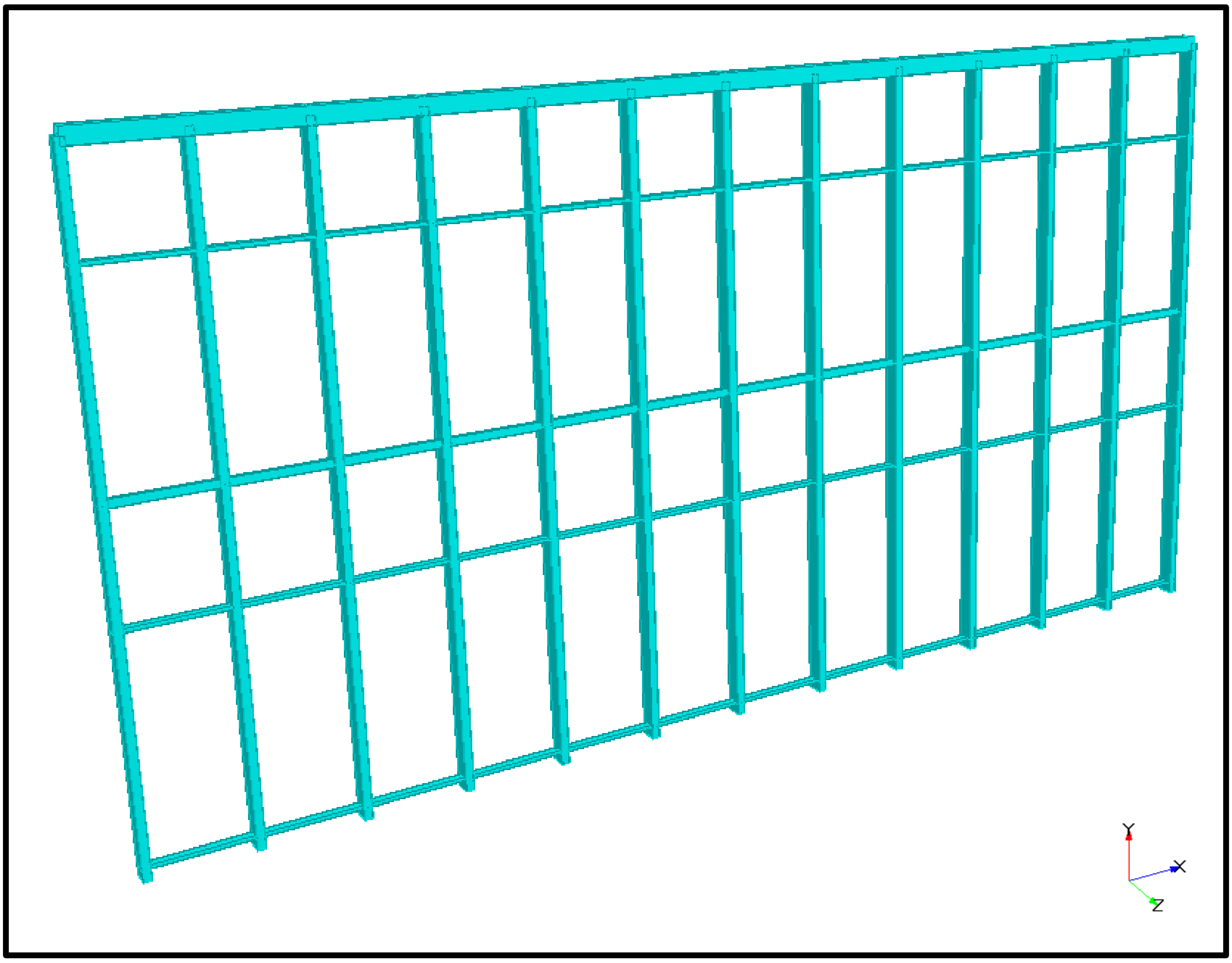
- AS/NZS 1170.0. 2002 - Structural Design Actions Part 0: General principles
- AS/NZS 1170.1. 2002 - Structural Design Actions Part 1: Permanent, imposed, and other actions
- AS/NZS 1170.2.2021 - Structural Design Actions Part 2: Wind Actions
- AS/NZS 4100:1998 - Steel Structures
- AS/NZS 2047:1999 - Windows in Buildings Selection & Installation
- AS/NZS 1664:1997 - Aluminium Structures_Part-1
- AS/NZS 1170.4 - Structural Design Actions Part 4: Earthquake actions
- AS 1288 - Glass Buildings
- AS 5216 - Design of Post Installed & Cast-In Fastening in Concrete
- AS 1530.4 - Fire Resistance Tests for Elements of Construction

4. STAAD MODELLING OF GLAZED WALL

Refer below images showing normal & render 3D view of Glazed Wall modeled in STAAD Pro software

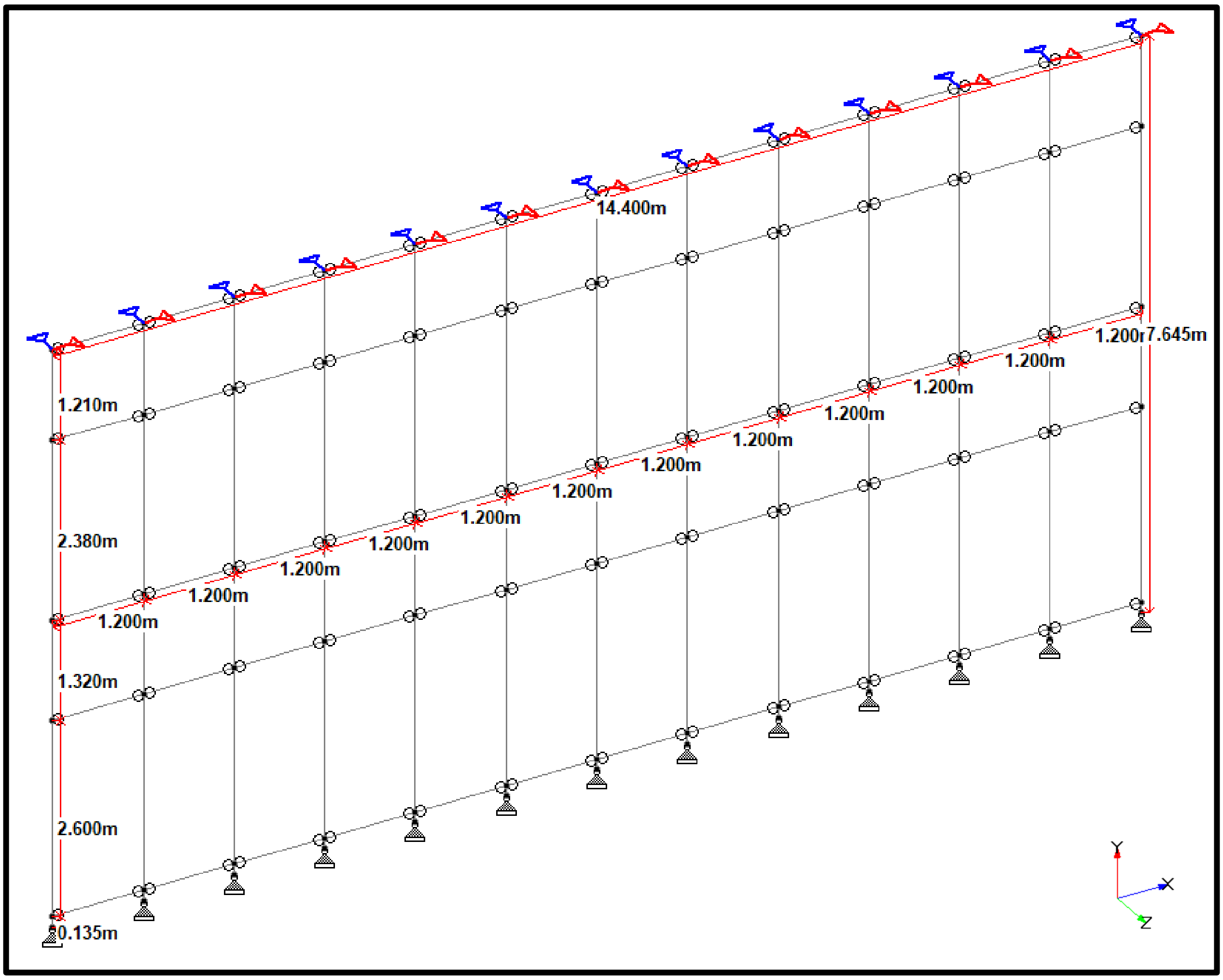


3D view of Glazed Wall



3D render view of entire structure

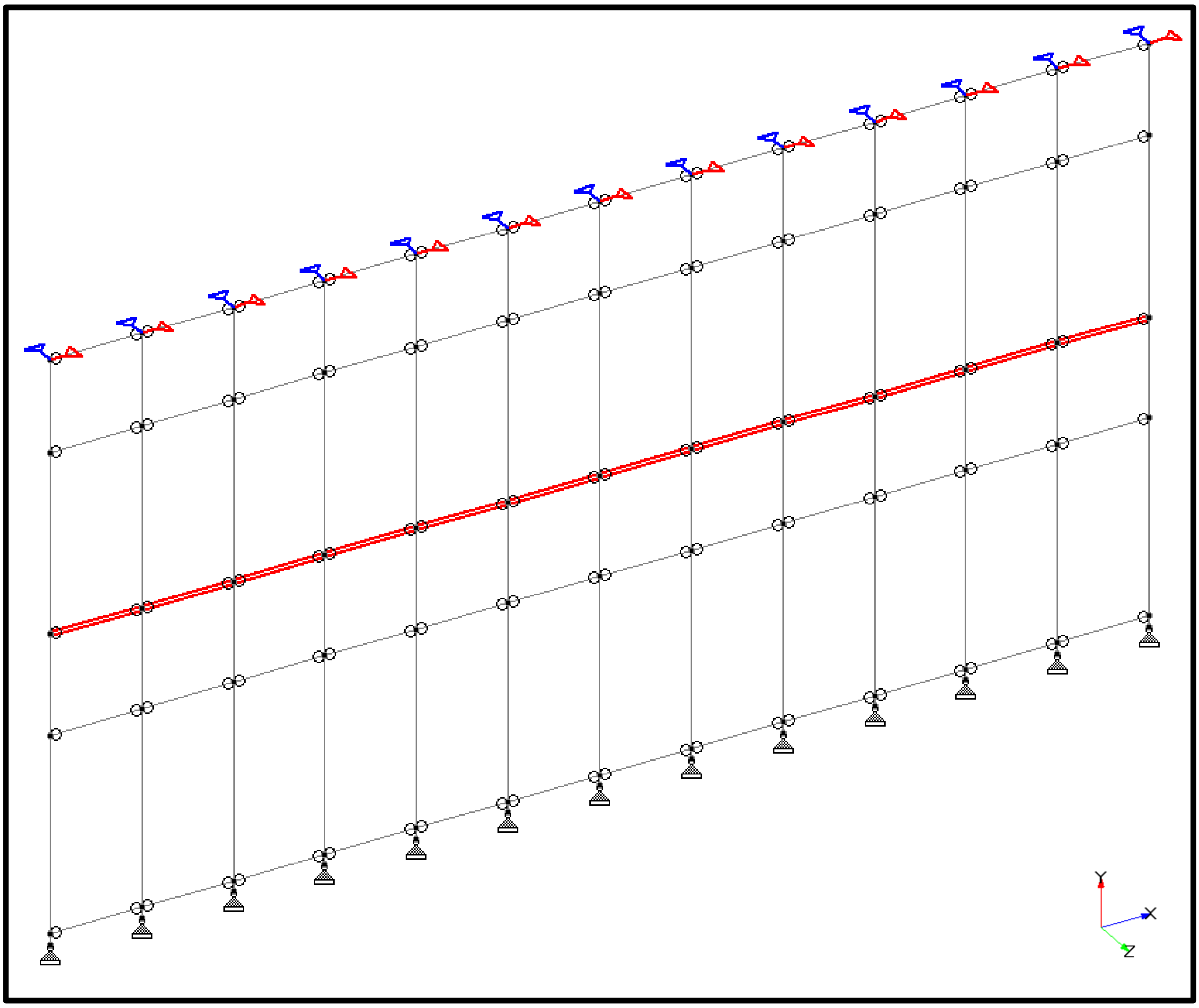
4.1. GEOMETRY DATA



Façade Geometry

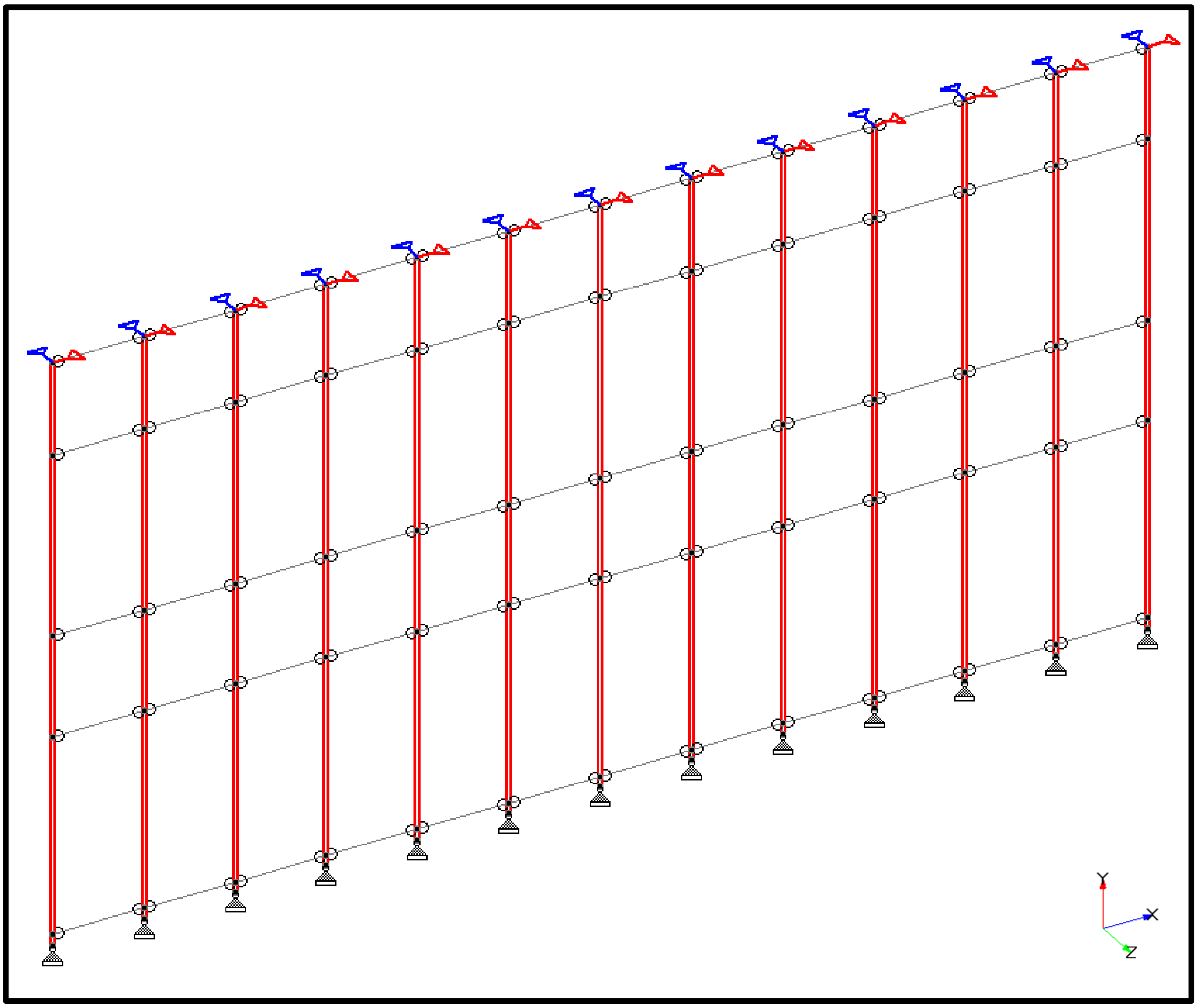
4.2. MEMBER PROPERTIES

1. 100 X 50 X 4.0 RHS – Steel Members:



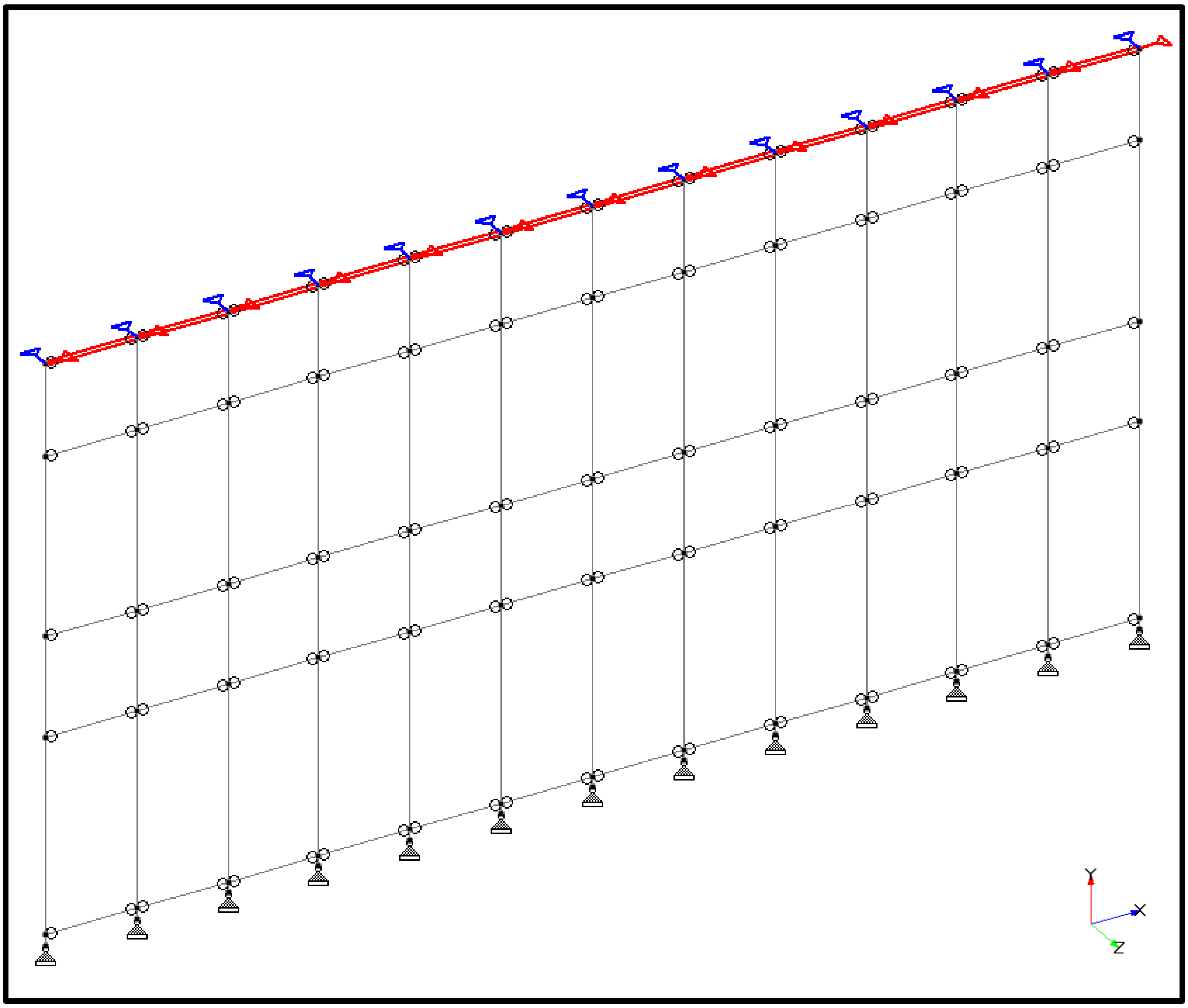
100 X 50 X 4.0 RHS

2. 200 X 100 X 5.0 RHS – Steel Members:



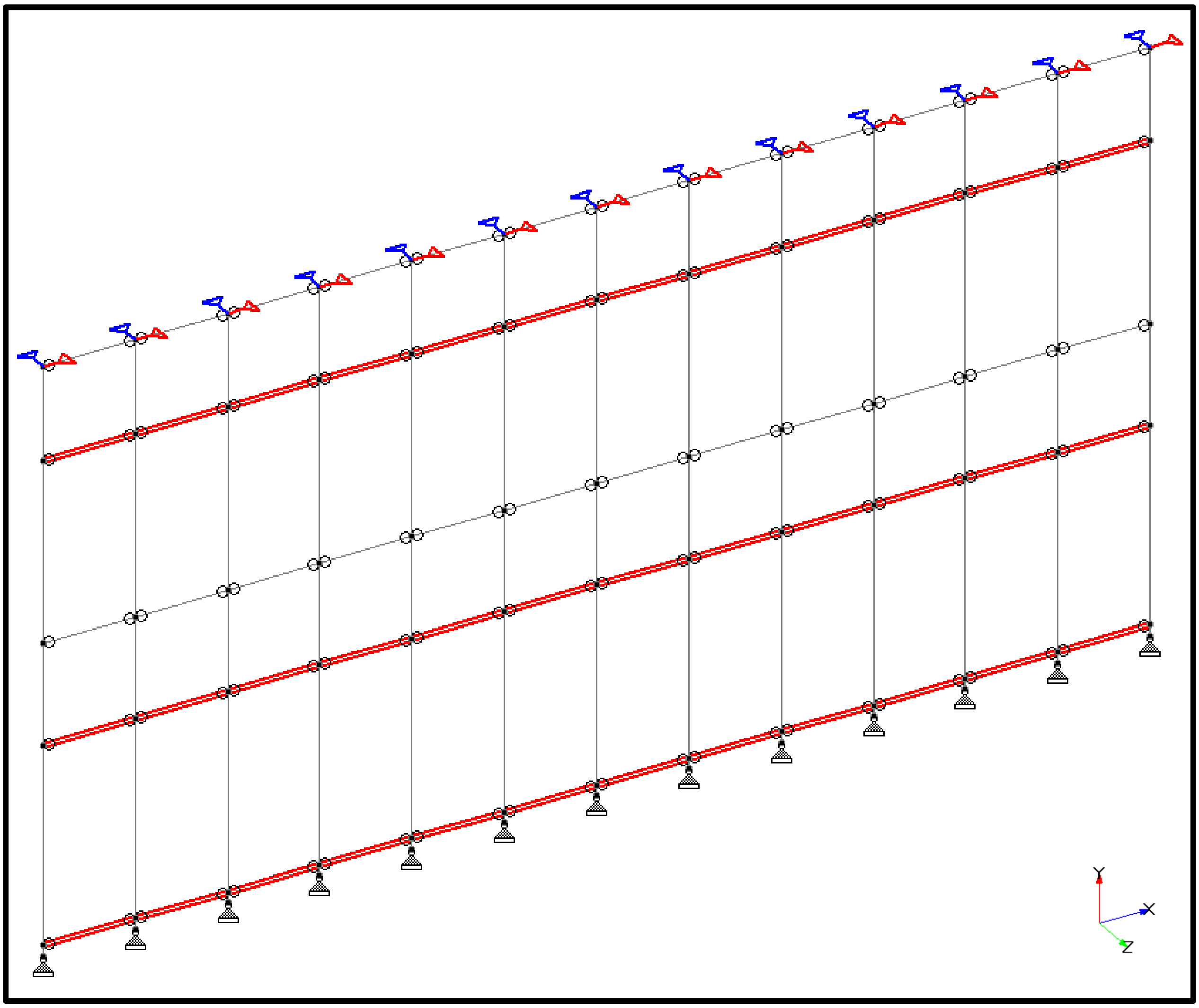
200 X 100 X 5.0 RHS

3. 200 X 200 X 5 RHS – Steel Members :



200 X 200 X 5 RHS

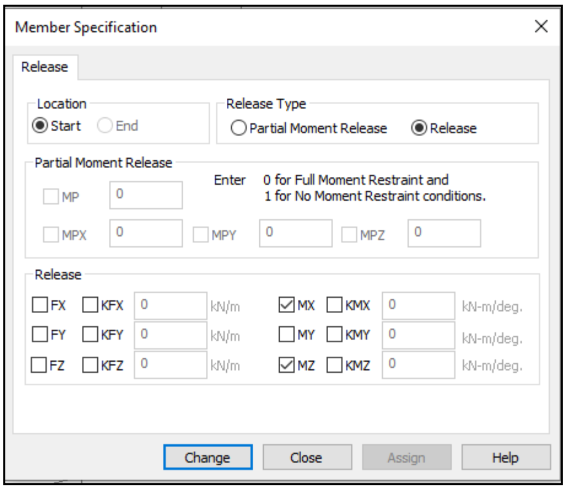
4. 100 X 50 X 3.2 RHS – Aluminium Members :

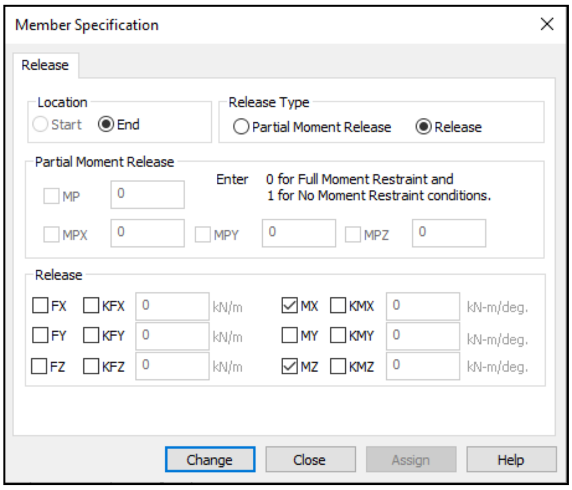


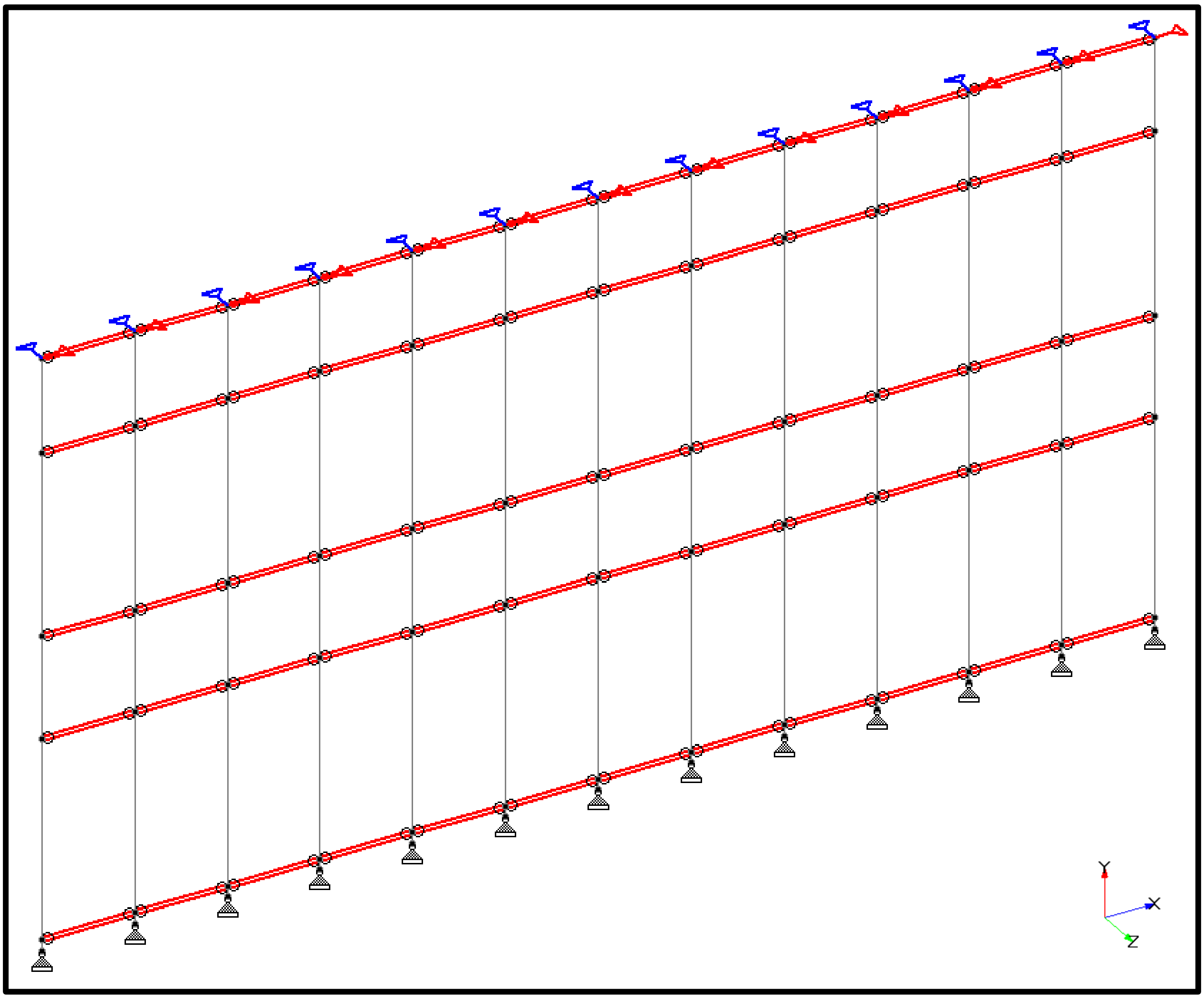
100 X 50 X 3.2 RHS - Aluminum Member Sections

4.3. MEMBER RELEASES

Refer below images shows member has been released at both ends.





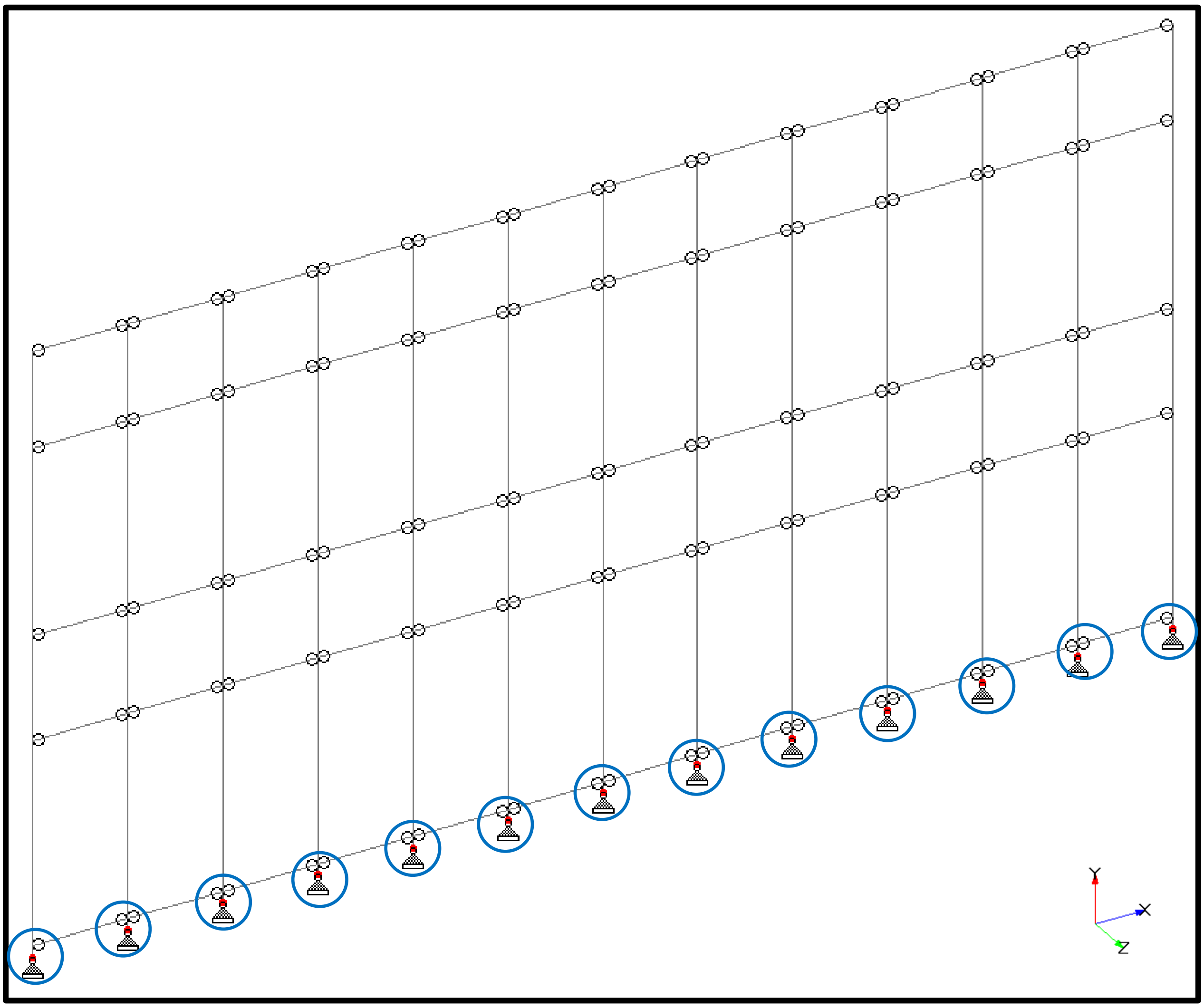


Moment Release

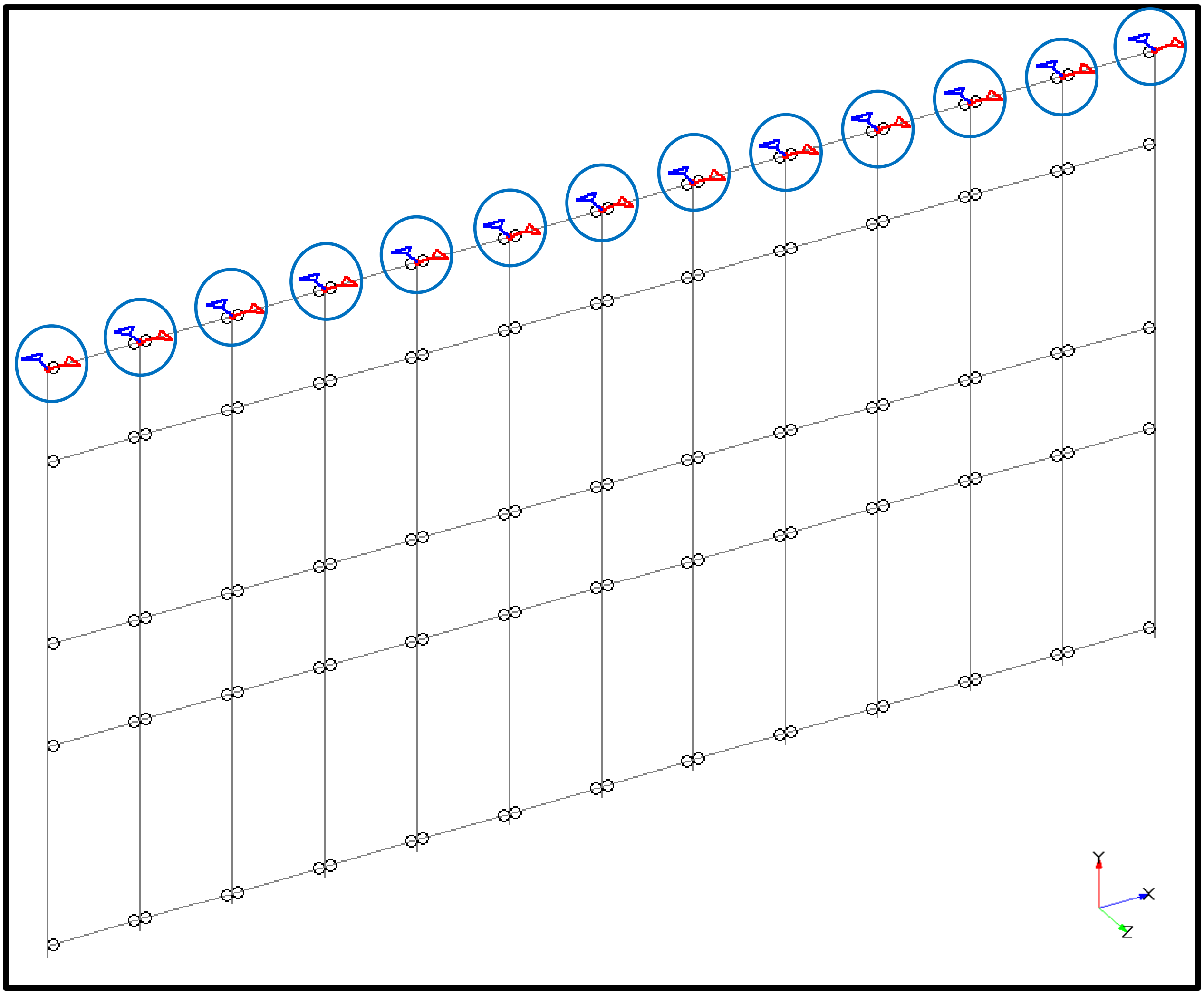
4.4. SUPPORT CONDITION

Pinned supports & fixed but supports have been assigned in StaadPro Model.

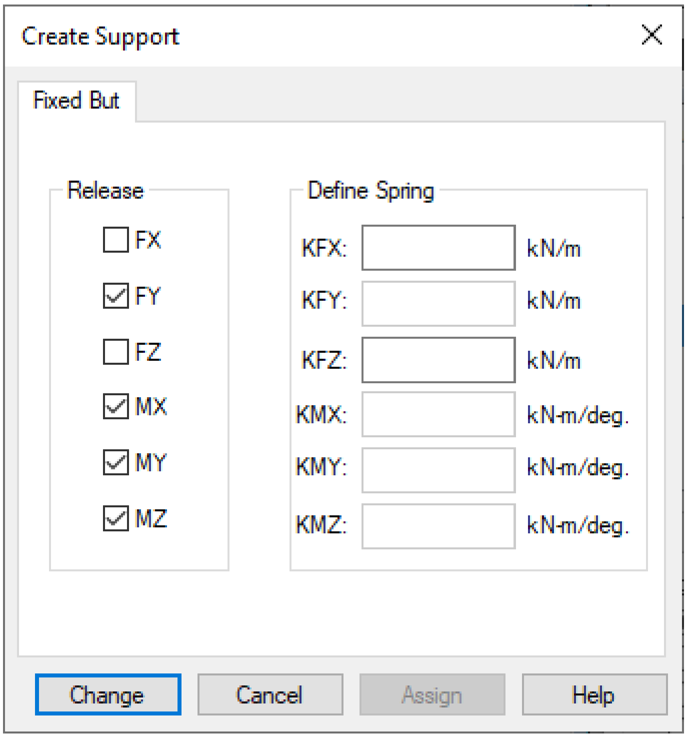
Refer below image showing location of these supports in STAAD model.



Pinned Supports



Fixed But Supports



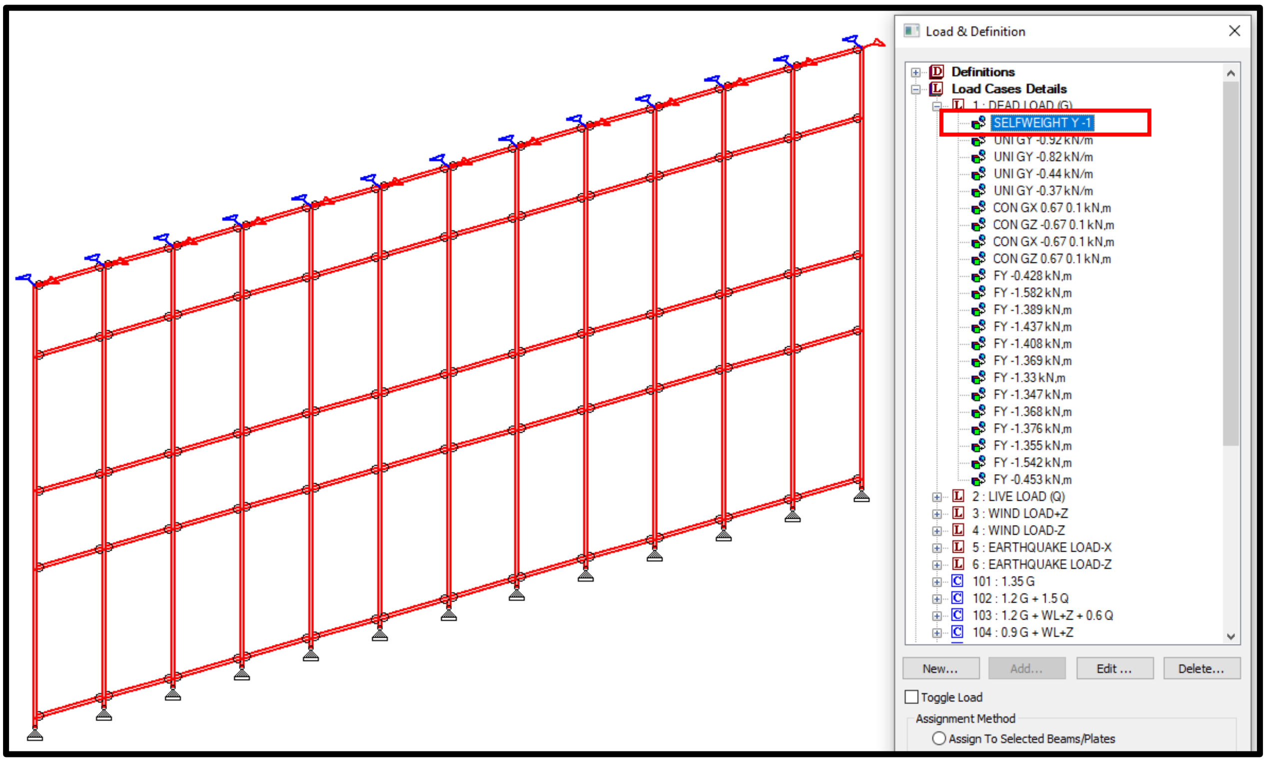
5. LOADING

Load cases:

1. DL: Dead Load
2. LL: Live load
3. WL: Wind Load
4. EL: Earthquake Load

5.1. DL: Dead Load

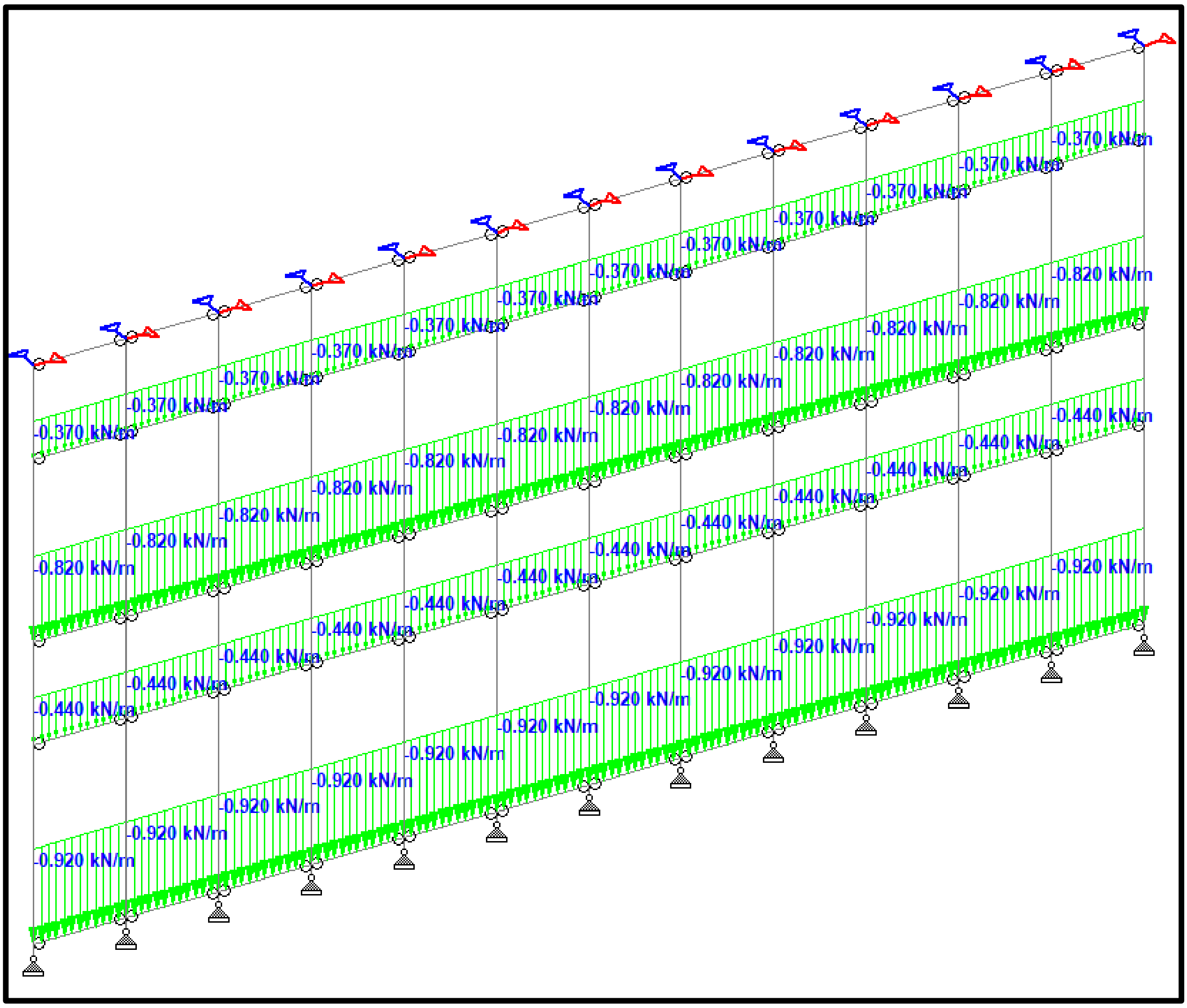
1. Self-weight of framing members



Self-weight of Members

2. Glass Panel load for Glaze Wall 13.52mm glass,

Glass panel = 26 kN/m^3 x 13.52 mm x 2.6m height	= 0.92 kN/m
Glass panel = 26 kN/m^3 x 13.52 mm x 2.305m height	= 0.82 kN/m
Glass panel = 26 kN/m^3 x 13.52 mm x 1.245m height	= 0.44 kN/m
Glass panel = 26 kN/m^3 x 13.52 mm x 1.035m height	= 0.37 kN/m



Glass Panel Load

3. Glass Door load on top & bottom pivot

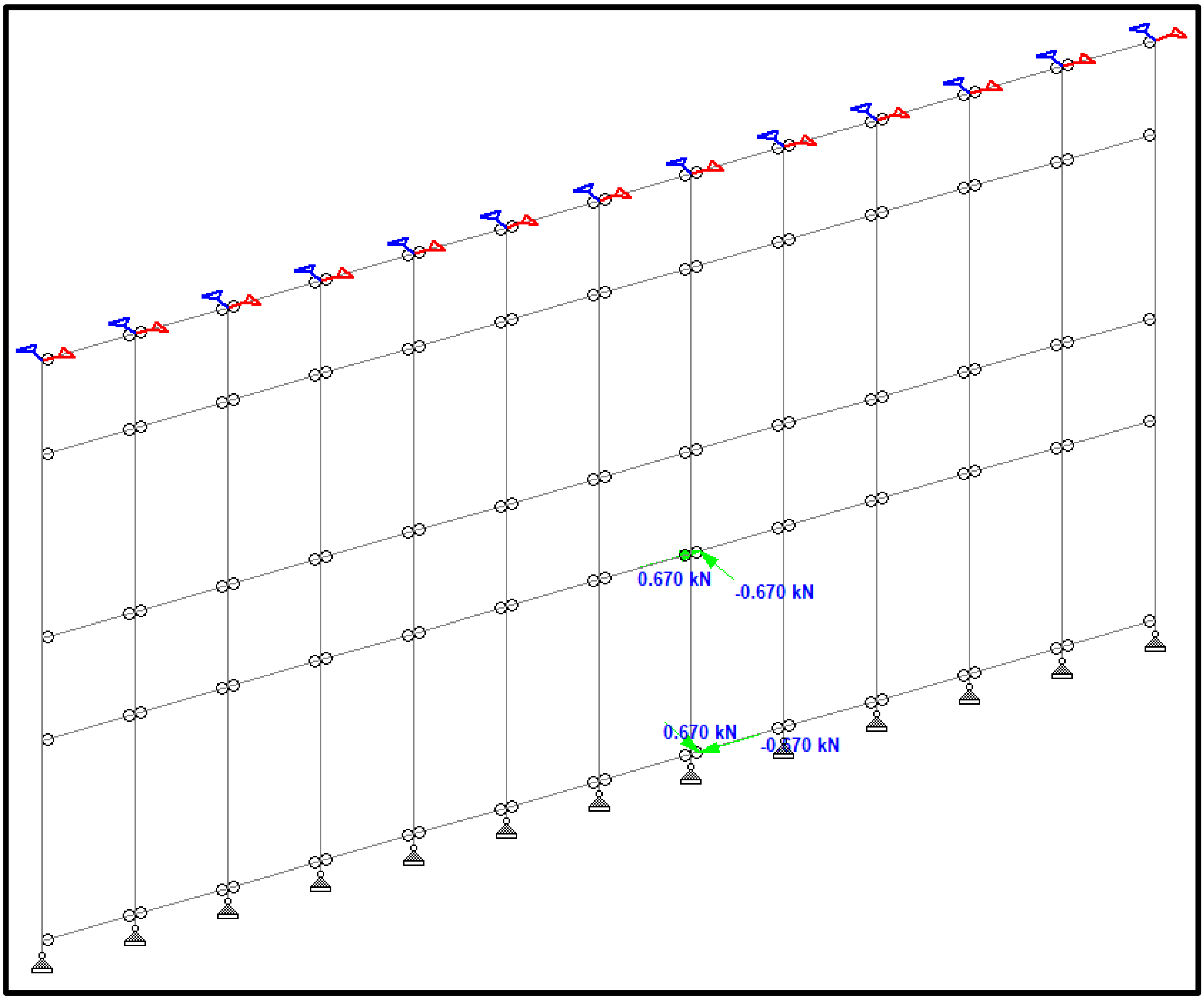
Self-weight of 17.52mm thick glass door = 26 kN/m^3 x 17.52 mm x1.2m width x 2.6m height

= 1.43 kN

Moment due to self-weight = 1.43 kN x 1.2m width
= 1.72 kN.m

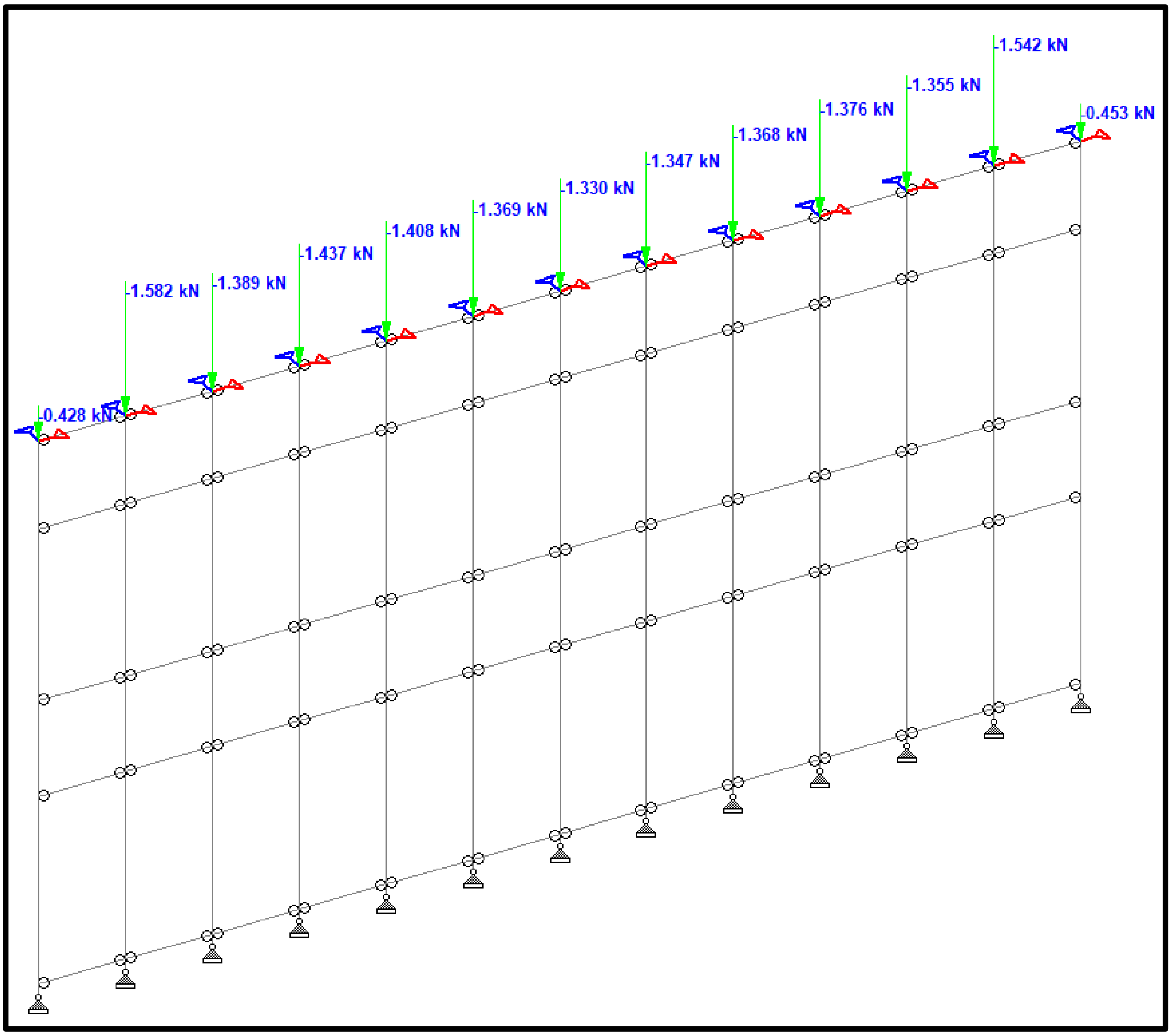
Moment converted to couple = 1.72 kN.m / 2.6m height
= 0.67 kN

Above derived 0.67 kN point load has been applied on pivot point in both direction as to satisfy both condition of door open & door closed.



Door Open & Closed Position

4. Roof Façade Dead Load



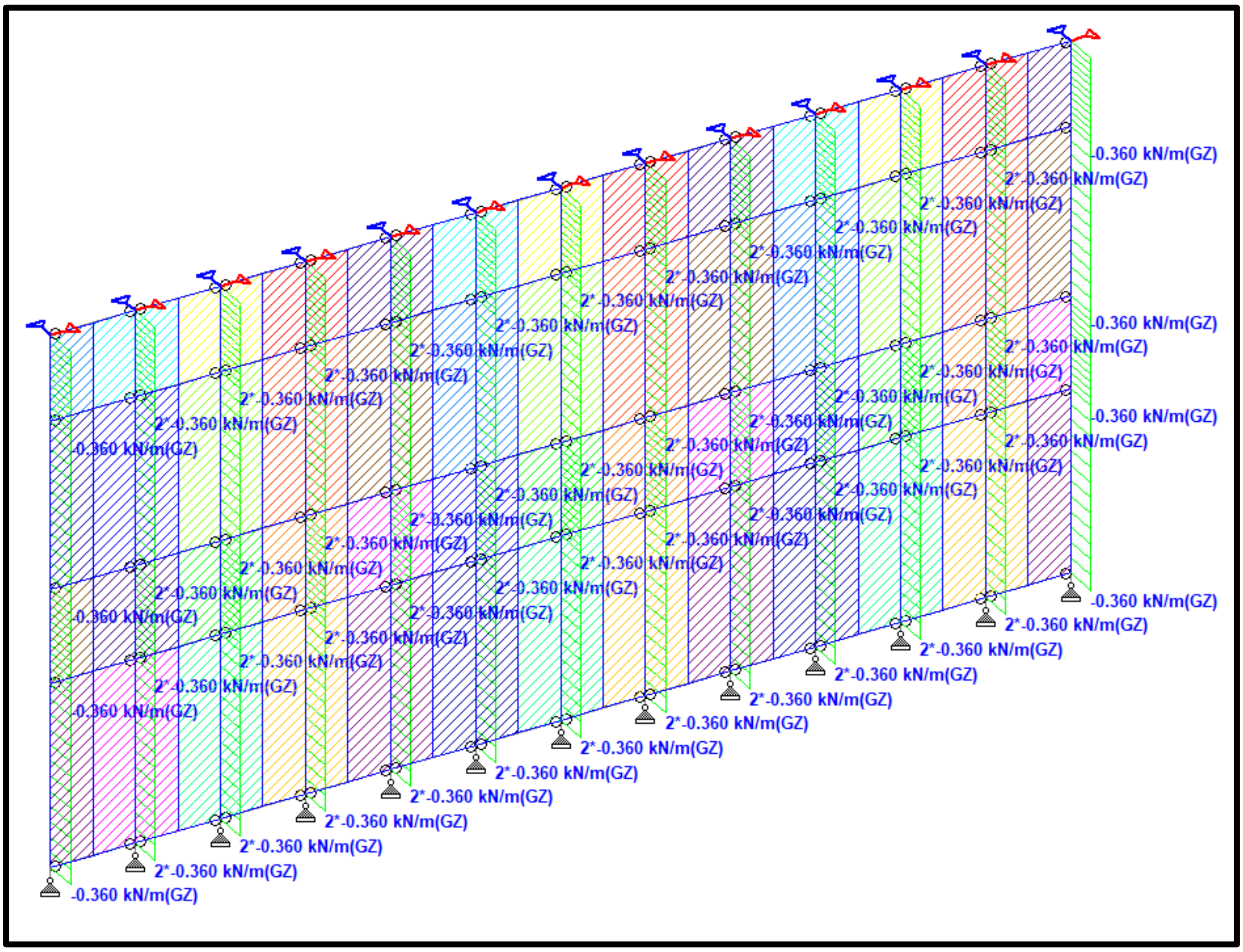
Roof Façade Dead Load

5.2. LL: Live Load

1. Live load on Facade

3.5.4 Loads from Maintenance and Building occupants	
Loads from Building Occupants	Vertical point load of 1kN applied anywhere or a uniformly distributed load of 0.6kN/m2 whichever is the most onerous to internal ledges, horizontal framing members and horizontal surfaces
Horizontal/near horizontal surfaces	Vertical uniformly distributed load of 0.6kPa, and a concentrated load of 1.1kN acting separately on a 150mm diameter contact area applied separately to any gutters, copings or flat and near flat surfaces.
Vertical/near vertical surfaces	500N applied horizontally through a 150mm diameter contact area on any vertical or near vertical surface which is accessible by building occupants or maintenance staff.

For live load application, 0.6 kN/m^2 applied as UDL.

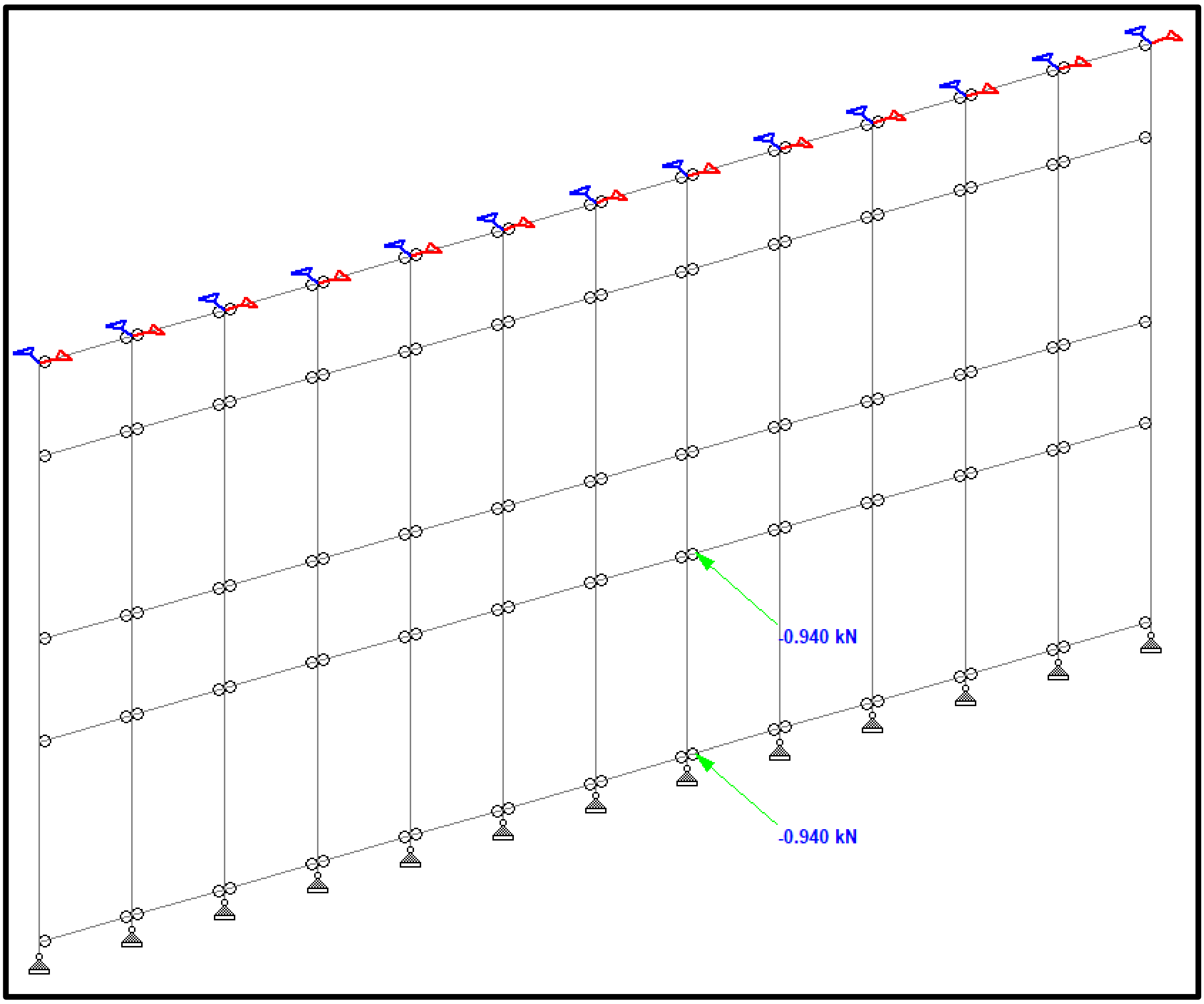


Live load on Facade

2. Glass Door live load on top & bottom pivot

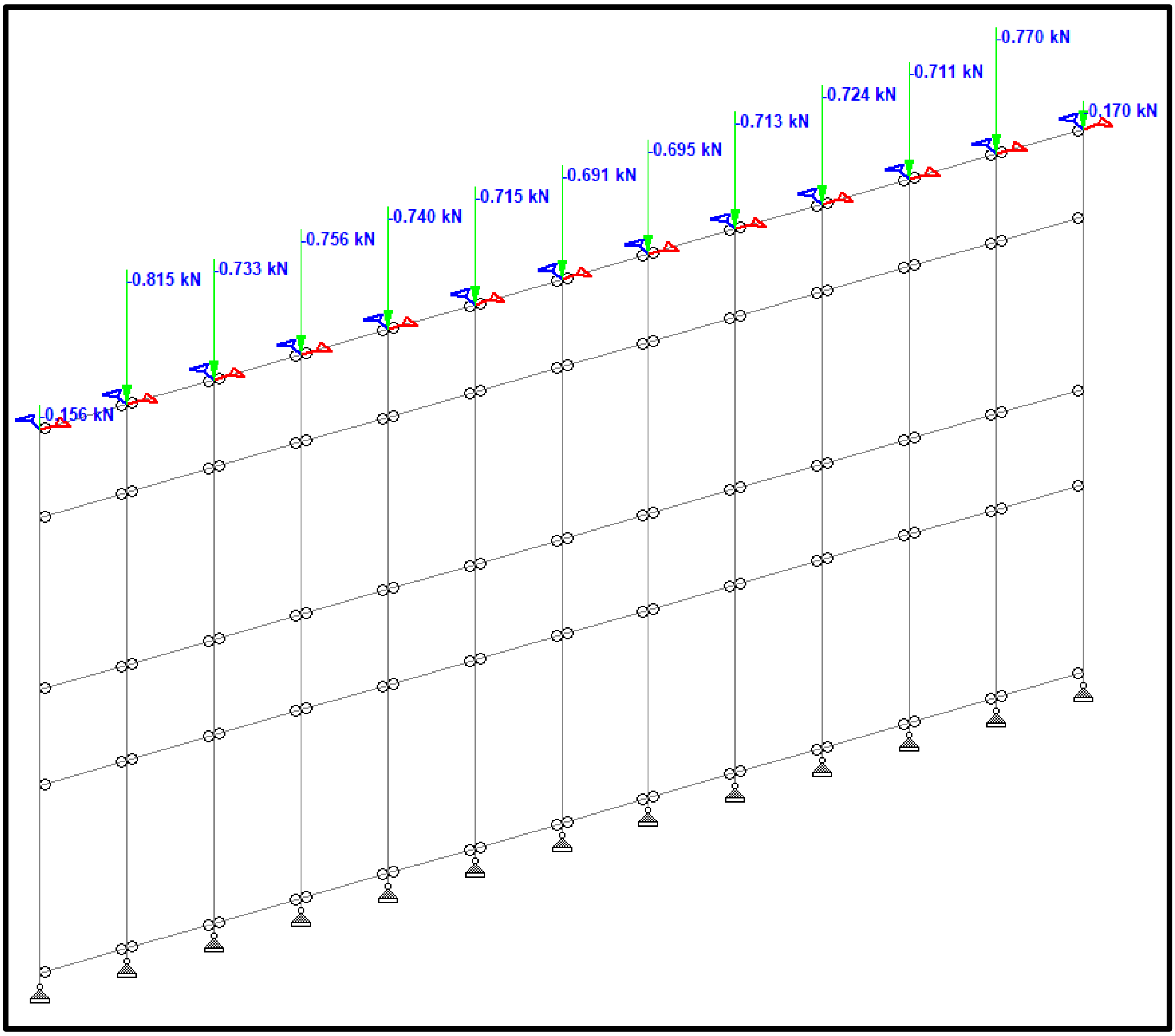
Live load on glass door = 0.6 kN/m^2 x1.2m width x 2.6m height
= 1.872 kN

Assuming half load on top pivot & half load on bottom pivot has been transferred
= 1.872 kN / 2
= 0.94 kN



Live load on Door

3. Roof Façade Live Load



Roof Façade Live Load

5.3. <u>WL: Wind Load</u>

AS/NZS_1170-2-2021
WIND LOAD CALCULATION

Wind Load as per AS 1170 Part 2 :-

Regional Wind Speed :

$V_{sit,\beta}$ = V_R X Md X (Mz,cat X Ms X Mt)

Where,

Regional Wind Speed :

V_R = Regional gust wind speed (m/s)

M_d = wind directional multipliers

$M_{z,cat}$ = terrain/height multiplier

Ms = shielding multiplier

Mt = topographic multiplier

As building is in Australia, Region of Wind is A2 (As per AS 1170.2:2021)

AS/NZS 1170.2:2021 20

Figure 3.1(A) — Wind regions — Australia

As mentioned earlier, The importance level of building considered 2.

for Importance level and annual probability of exceedence, Regional wind speed considered.

Table 3.1(A) — Regional wind speeds — Australia

Regional wind speed (m/s)	Region			
	Non-cyclonic		Cyclonic	
	A (0 to 5)	B1, B2	C (maximum)	D (maximum)
V_1	30	26	23	23
V_5	32	28	33	35
V_{10}	34	33	39	43
V_{20}	37	38	45	51
V_{25}	37	39	47	53
V_{50}	39	44	52	60
V_{100}	41	48	56	66
V_{200}	43	52	61	72
V_{250}	43	53	62	74
V_{500}	45	57	66	80
V_{1000}	46	60	70	85
V_{2000}	48	63	73	90
V_{2500}	48	64	74	91
V_{5000}	50	67	78	95
V_{10000}	51	69	81	99
V_R ($R \geq 5$ years)	$67-41R^{-0.1}$	$106-92R^{-0.1}$	$122-104R^{-0.1}$	$156-142R^{-0.1}$

NOTE 1 The peak gust has an equivalent moving average time of approximately 0.2 s (Holmes and Ginger, 2012).

NOTE 2 Values for V_1 have not been calculated by the formula for V_R in the Australian regions.

NOTE 3 For ultimate or serviceability limit states, refer to the National Construction Code (Australia) or AS/NZS 1170.0 for information on values of importance level and annual probability of exceedance appropriate for the design of structures. For buildings in townships in cyclonic regions, users should consider overall risk to a community when selecting importance levels.

NOTE 4 For Regions C and D, only the maximum values for the region are tabulated. Lower values of V_R may apply in those regions, depending on the distance of the site from the smooth coastline.

VR considered with, Importance level as 2 and Design workign life as 50 year.

V500 =	45	m/s	Table 3.1 AS1170 Part 2 (For Ultimat Limit State)
V25 =	37	m/s	Table 3.1 AS1170 Part 2 (For serviceability Limit State)
Md =	1	from Clause 3.3.2 AS1170 Part 2 by considering any direction.	

Table 3.2(A) — Wind direction multiplier (M_d) — Australia

Cardinal directions	Region A0	Region A1	Region A2	Region A3	Region A4	Region A5	Region B1	Regions B2, C, D
N	0.90	0.90	0.85	0.90	0.85	0.95	0.75	0.90
NE	0.85	0.85	0.75	0.75	0.75	0.80	0.75	0.90
E	0.85	0.85	0.85	0.75	0.75	0.80	0.85	0.90
SE	0.90	0.80	0.95	0.90	0.80	0.80	0.90	0.90
S	0.90	0.80	0.95	0.90	0.80	0.80	0.95	0.90
SW	0.95	0.95	0.95	0.95	0.90	0.95	0.95	0.90
W	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.90
NW	0.95	0.95	0.95	0.95	1.00	0.95	0.90	0.90

NOTE In Region A0 non-synoptic winds are dominant. In Regions A1 and A4, extra-tropical synoptic winds are dominant. Extreme winds in Regions A2, A3, A5 and B1 are caused by a mixture of synoptic (extra-tropical large-scale pressure systems, or tropical cyclones in the case of B1) and non-synoptic (thunderstorm) events. In Regions B2, C, and D, extreme winds from tropical cyclones are dominant.

Table 3.2 NZS1170 Part 2 (WIND DIRECTION MULTIPLIER Md) (For region A2)

wind direction in X+, Wind direction multiplier for NE =	0.75
wind direction in X-, Wind direction multiplier for SW =	0.95
wind direction in Y+, Wind direction multiplier for NW =	0.95
wind direction in Y-, Wind direction multiplier for SE =	0.95

wind direction in N, Wind direction multiplier =	0.85
wind direction in S, Wind direction multiplier =	0.95
wind direction in E, Wind direction multiplier =	0.85
wind direction in W, Wind direction multiplier =	1

Wind applied at 45° angle,	Vsit,β =	45°
	Cos45° =	0.71
	Sin45° =	0.71

Wind applied at 45° angle for NE	=	NE X Cos45°/Sin45°
	=	0.5325
Wind applied at 45° angle for SW	=	SW X Cos45°/Sin45°
	=	0.6745
Wind applied at 45° angle for NW	=	NW X Cos45°/Sin45°
	=	0.6745
Wind applied at 45° angle for SE	=	SE X Cos45°/Sin45°
	=	0.6745

Hence, Above all direction multiplier take a critical at E and W direction.

Wind direction multiplier Md considered with worst case with considering maximum cardinal direction within a sector 45 degree in both side.

Terrain Category (AS1170 Part 2)

Based upon the site condition, Terrain category considered = 3

Table 4.1 — Terrain/height multipliers for gust wind speeds in fully developed terrains — All regions except A0

Height (z) (m)	Terrain/height multiplier ($M_{z,cat}$)				
	Terrain Category 1	Terrain Category 2	Terrain Category 2.5	Terrain Category 3	Terrain Category 4
≤ 3	0.97	0.91	0.87	0.83	0.75
5	1.01	0.91	0.87	0.83	0.75
10	1.08	1.00	0.92	0.83	0.75
15	1.12	1.05	0.97	0.89	0.75
20	1.14	1.08	1.01	0.94	0.75
30	1.18	1.12	1.06	1.00	0.80
40	1.21	1.16	1.10	1.04	0.85
50	1.23	1.18	1.13	1.07	0.90
75	1.27	1.22	1.17	1.12	0.98
100	1.31	1.24	1.20	1.16	1.03
150	1.36	1.27	1.24	1.21	1.11
200	1.39	1.29	1.27	1.24	1.16

NOTE 1 In Region A0, use $M_{z,cat\,2}$ for all $z \leq 100$ m in all terrains. For 100 m $< z \leq 200$ m, take $M_{z,cat}$ as 1.24 in all terrains.

NOTE 2 For all other regions, for intermediate terrains use linear interpolation.

NOTE 3 For intermediate values of height z, use linear interpolation.

By linear interpolation,

Height of building,h = 7.62 m

Determination of terrain/height multiplier (Mz,cat) = 0.83

Table 4.1 AS 1170 Part 2

Ms = Shielding multiplier = 1 AS1170 Part 2

Mt= Topographic multiplier = As per AS.1170.2:2021, Mlee can be taken as 1.0

Mt=Mh

Mt= Topographic multiplier = 1

Site Wind Speed

Vsit,β =	Vr X Md X (Mz,cal X Ms X Mt)
V500 =	45 x 1 x 0.83 x 1 x 1
=	37.35
V25 =	37 x 1 x 0.83 x 1 x 1
=	30.71

Design wind pressure :

$$p = (0.5\ \rho_{\text{air}})\ [V_{\text{des},\theta}]^2\ C_{\text{fig}}\ C_{\text{dyn}}$$

C_{dyn} = 1

$C_{\text{fig,e}} = C_{\text{p,e}}\ K_a\ K_{c,e}\ K_\ell\ K_p$, for external pressures

$C_{\text{fig,i}} = C_{\text{p,i}}\ K_{c,i}$, for internal pressures

ρ_{air} = density of air, which shall be taken as 1.2 kg/m^3

Cpe = external pressure coefficient		Table 5.2 (A)/(B)/(C) AS-1170 Part 2
Cpi = Internal pressure coefficient		Table 5.1 (A) AS-1170 Part 2
Ka =	0.9	Table 5.4 AS-1170 Part 2
Kce=	0.9	Table 5.5 AS-1170 Part 2
Kci=	1	Table 5.5 AS-1170 Part 2
Kl=	1.5	Table 5.6 AS-1170 Part 2
Kp =	0.9	Table 5.8 AS-1170 Part 2

Puls = 0.6 X 37.35^2 X (Cfig.e+Cfig,i)

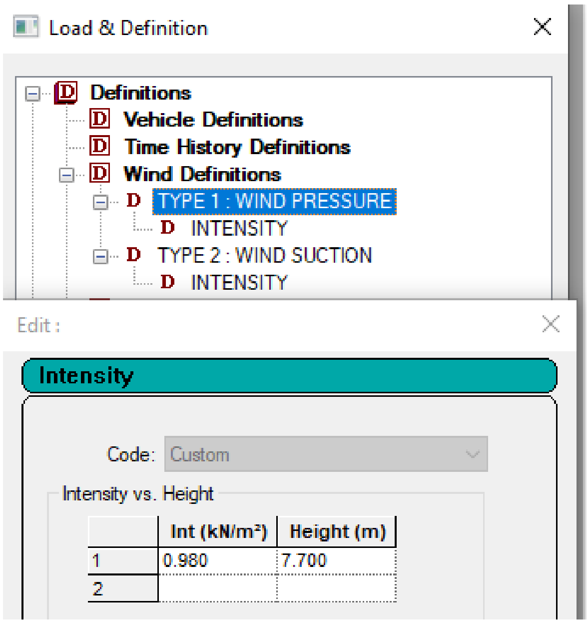
Psls = 0.6 X 30.71^2 X (Cfig.e+Cfig,i)

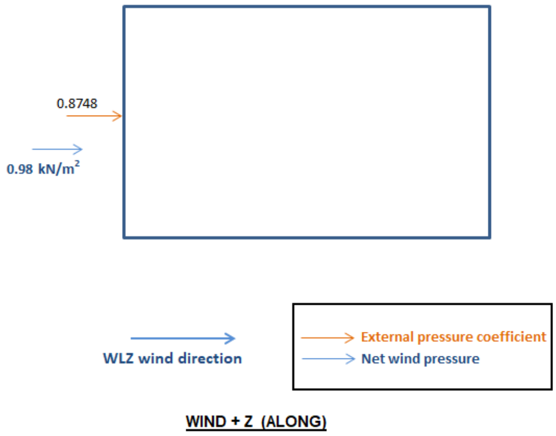
External wind coeficients

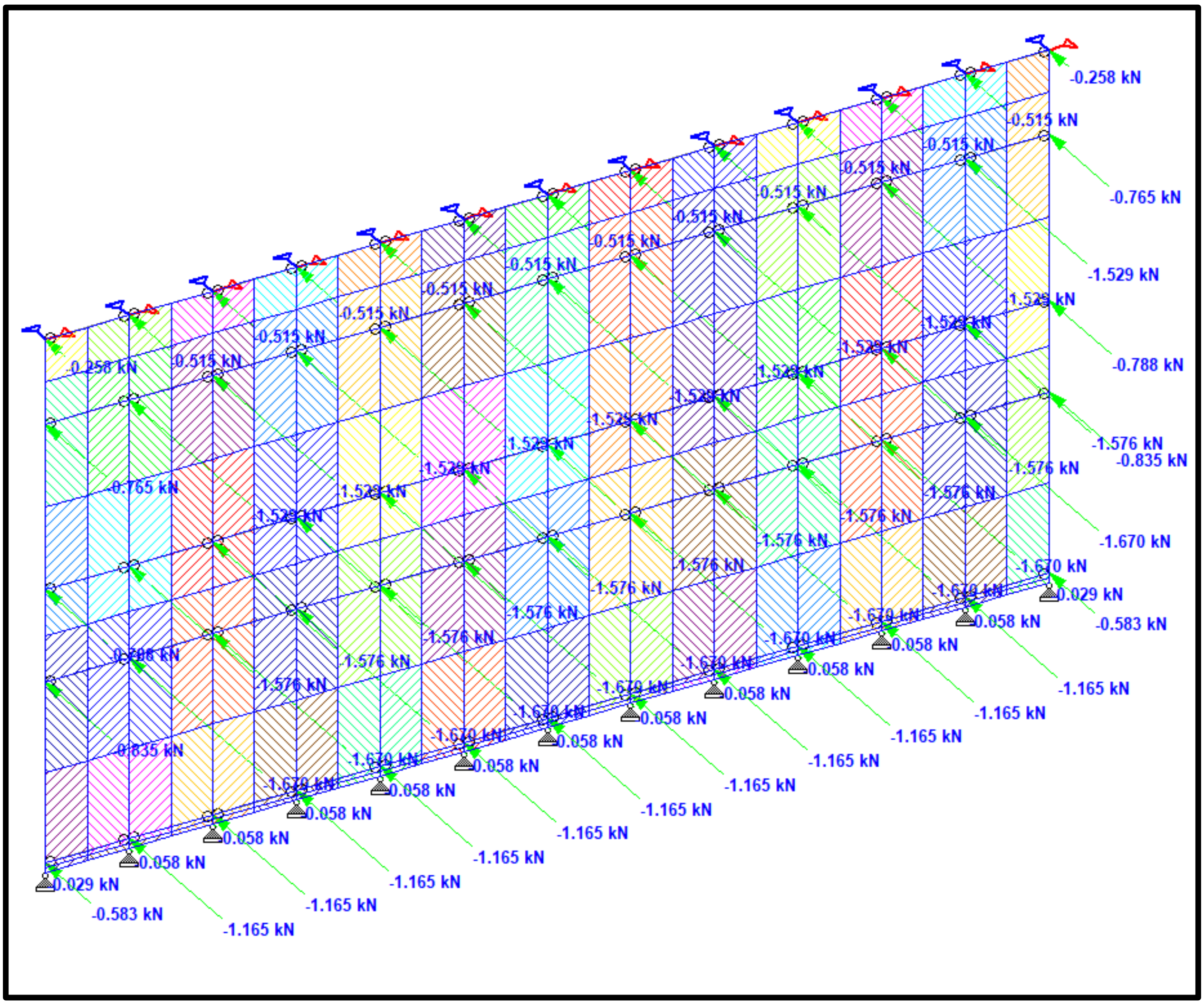
Windward	C_{pe} =	0.8	
Leeward	C_{pe} =	-0.5	
Side wall	C_{pe} =	-0.65	
Roof	C_{pe} =	-1.3	-0.6

Windward	$C_{fig\ e}$ =	0.87	
Leeward	$C_{fig\ e}$ =	-0.55	
Side wall	$C_{fig\ e}$ =	-0.71	
Roof	$C_{fig\ e}$ =	-1.42	-0.66

1. Wind Load (Pressure)





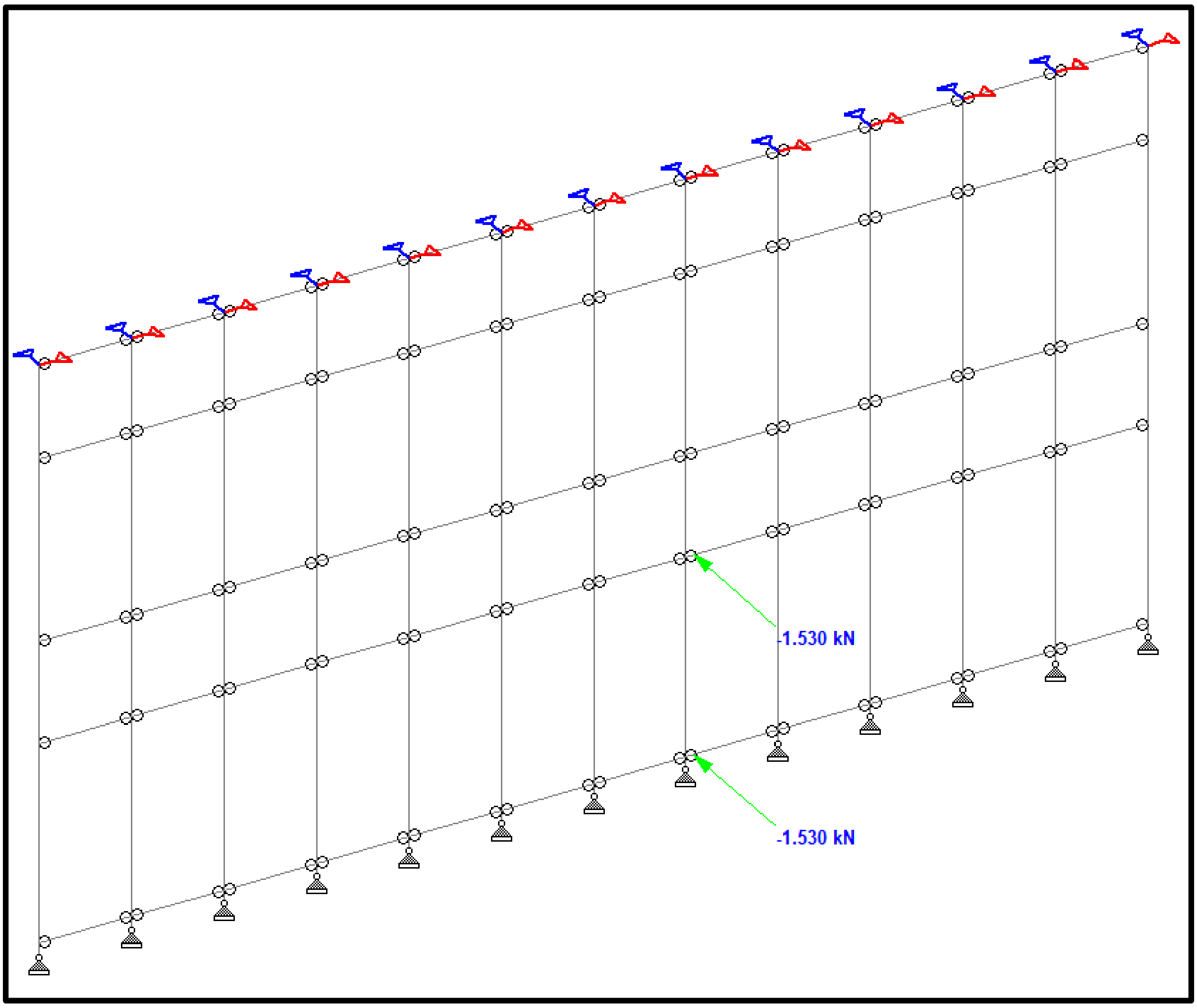


Wind load on Facade

2. Wind Load (Pressure) on Glass Door

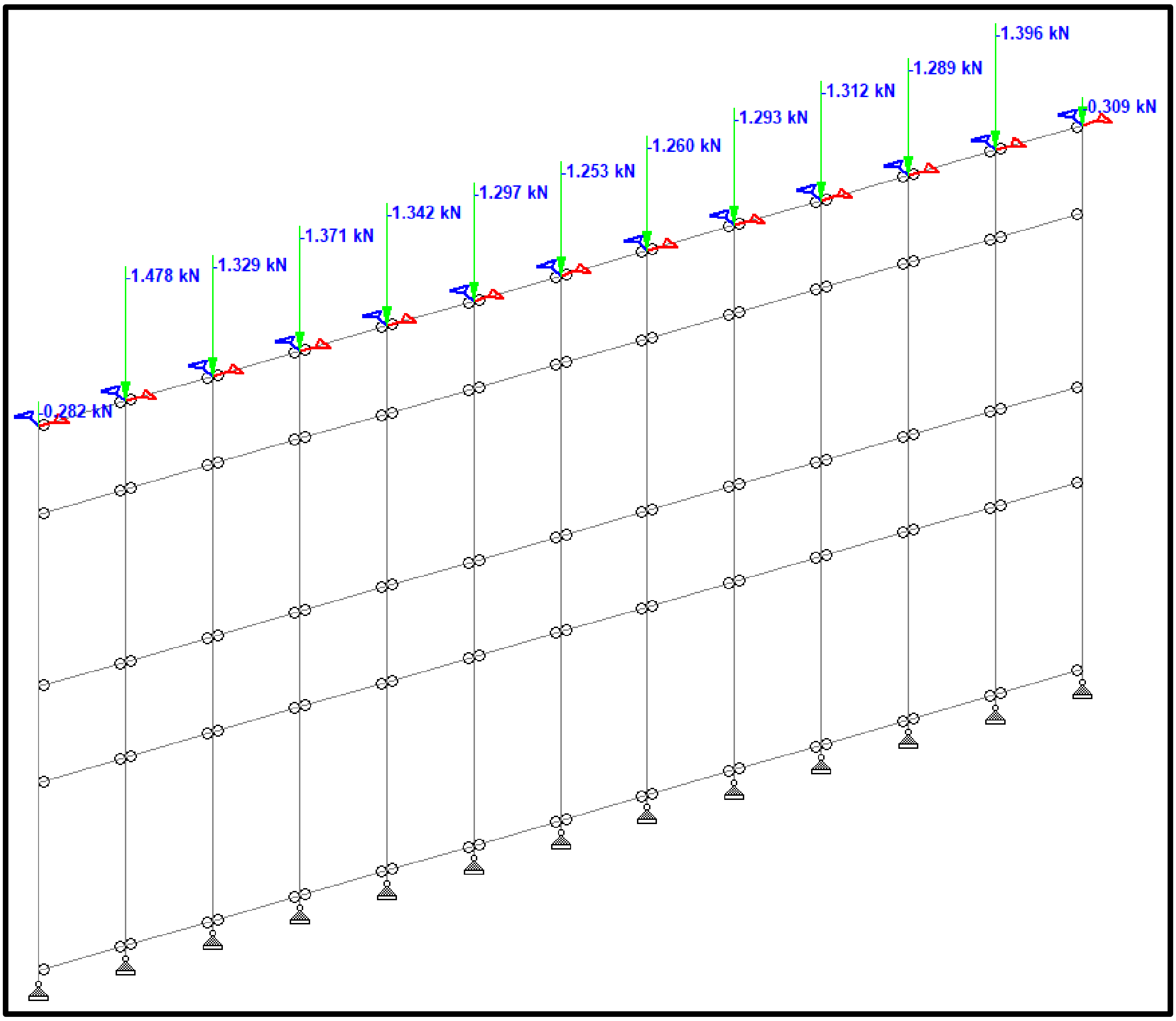
Wind load on glass door $= 0.98\ kN/m^2$ x1.2m width x 2.6m height
= 3.06 kN

Assuming half load on top pivot & half load on bottom pivot has been transferred
= 3.06 kN / 2
= 1.53 kN



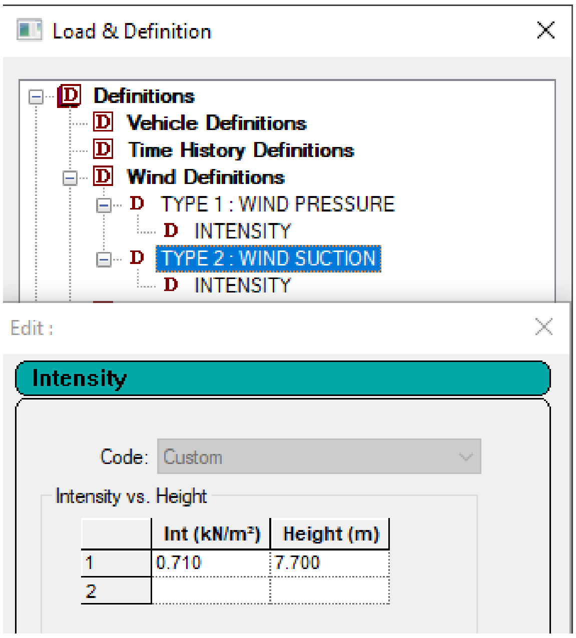
Wind load on Door

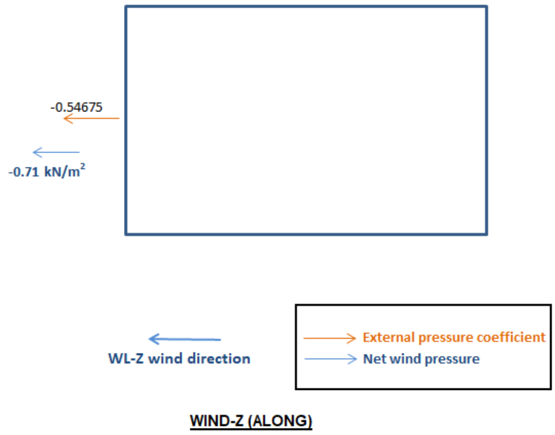
3. Wind Load (Pressure) of Roof Façade

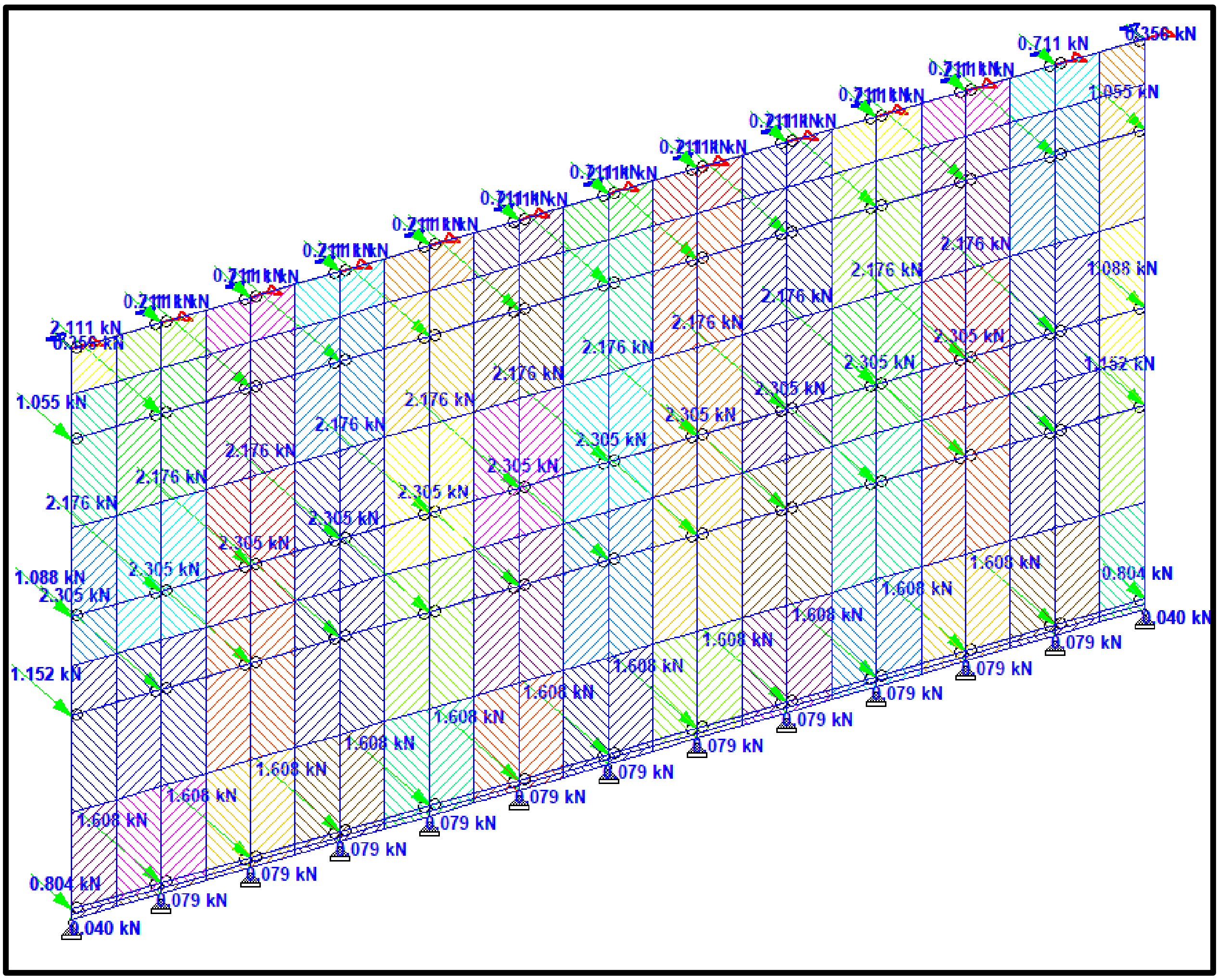


Roof Façade Wind Load (Pressure)

4. Wind Load (Suction)





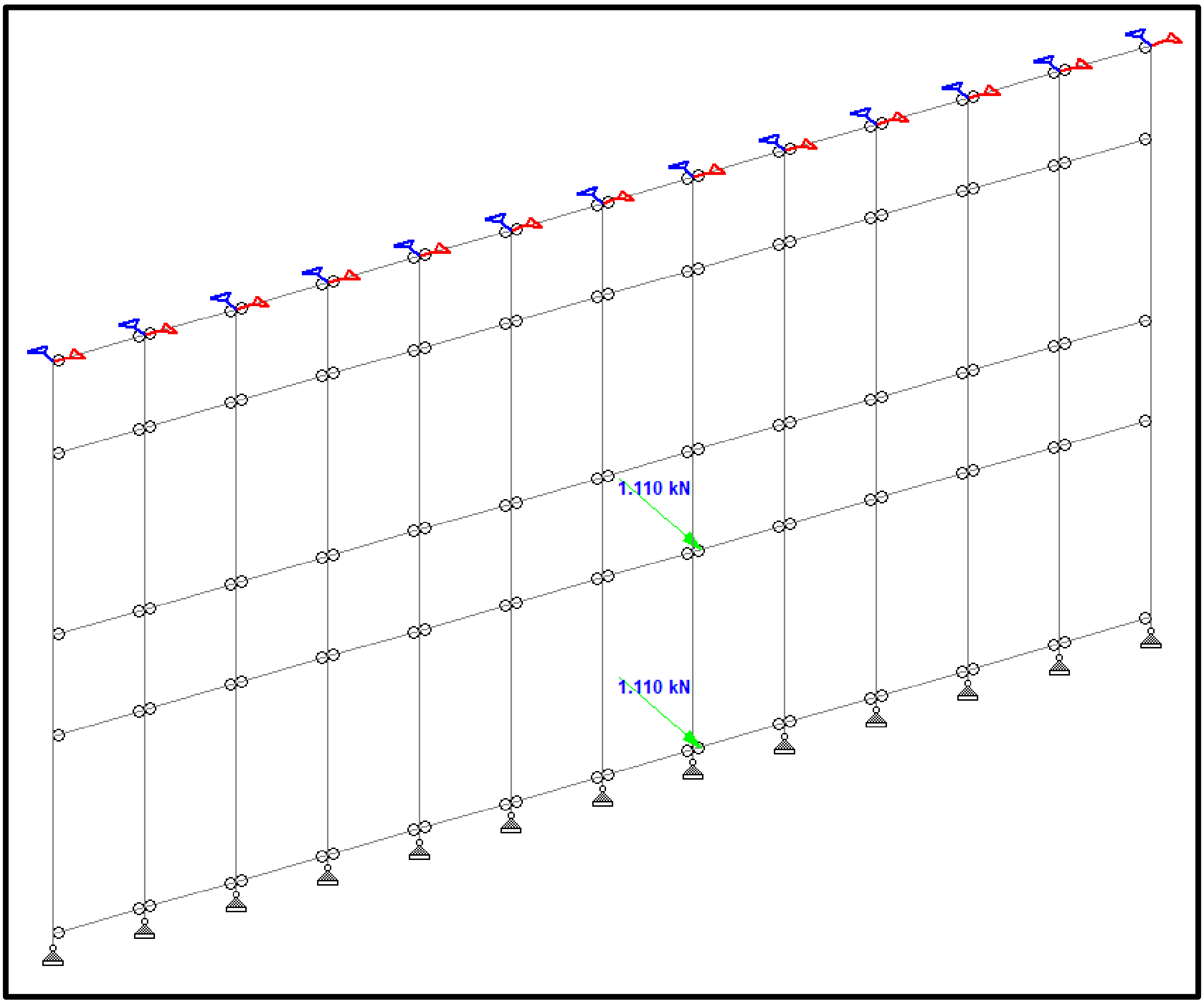


Wind load on Facade

5. Wind Load (Suction) on Glass Door

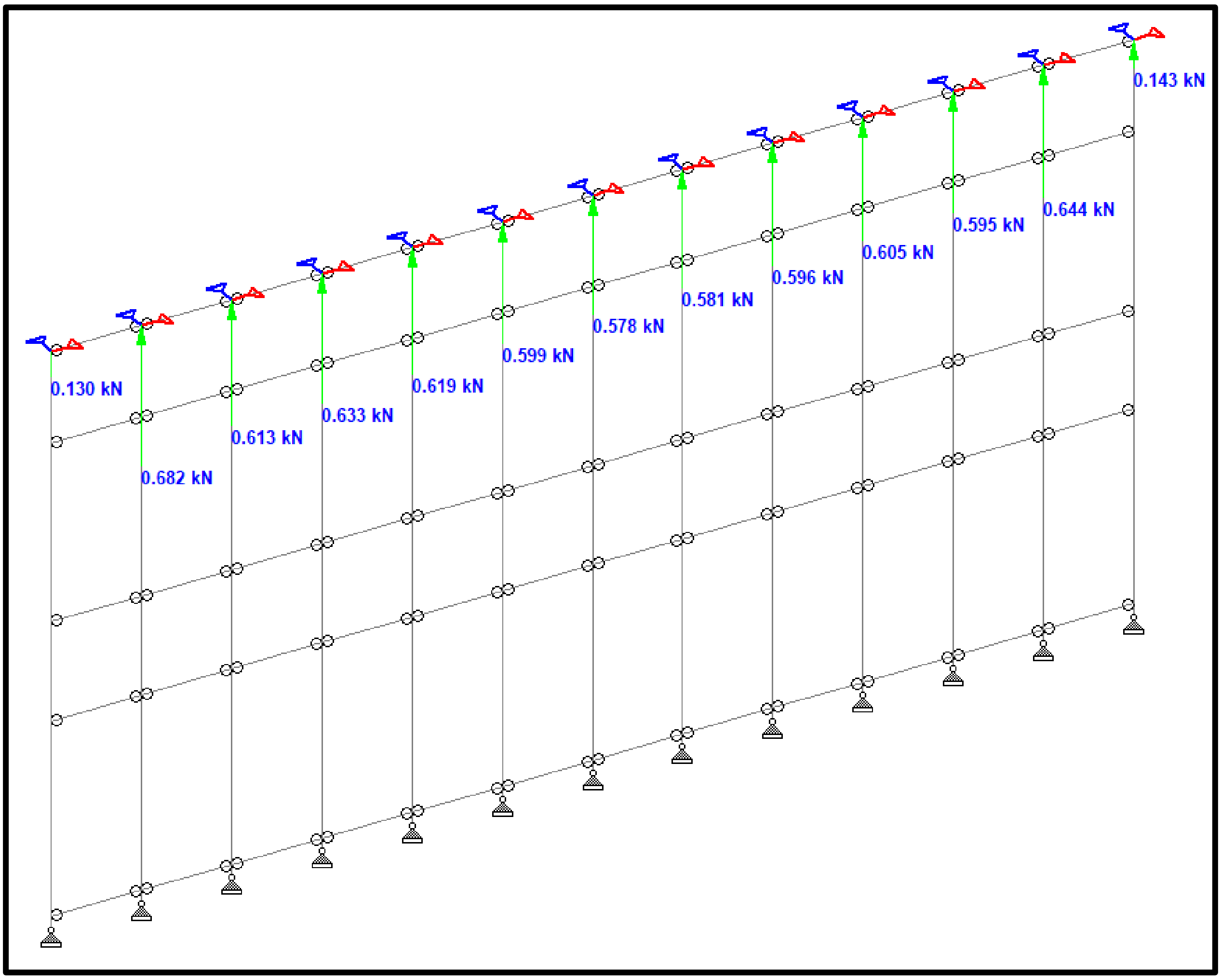
Wind load on glass door = 0.71 kN/m^2 x1.2m width x 2.6m height
= 2.22 kN

Assuming half load on top pivot & half load on bottom pivot has been transferred
= 2.22 kN / 2
= 1.11 kN



Wind load on Door

6. Wind Load (Suction) of Roof Façade



Roof Façade Wind Load (Suction)

5.4. EQ: Earthquake Load

EQUIVALENT STATIC METHOD

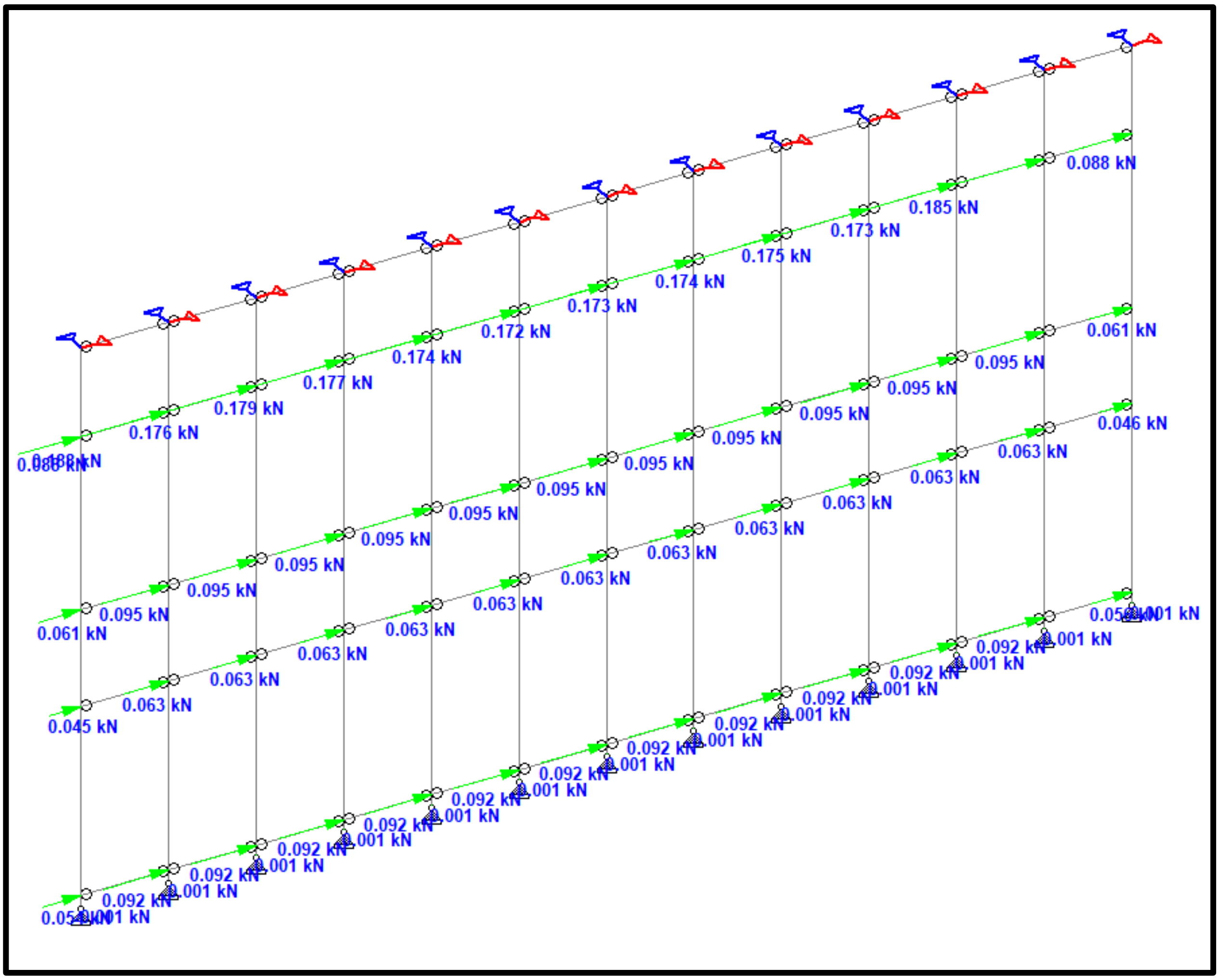
The horizontal equivalent static shear force (V) acting at the base of the structure (base shear) in the direction being considered shall be calculated from the following equations:

Earthquake base shear $V = C_d(T_1)W_t$

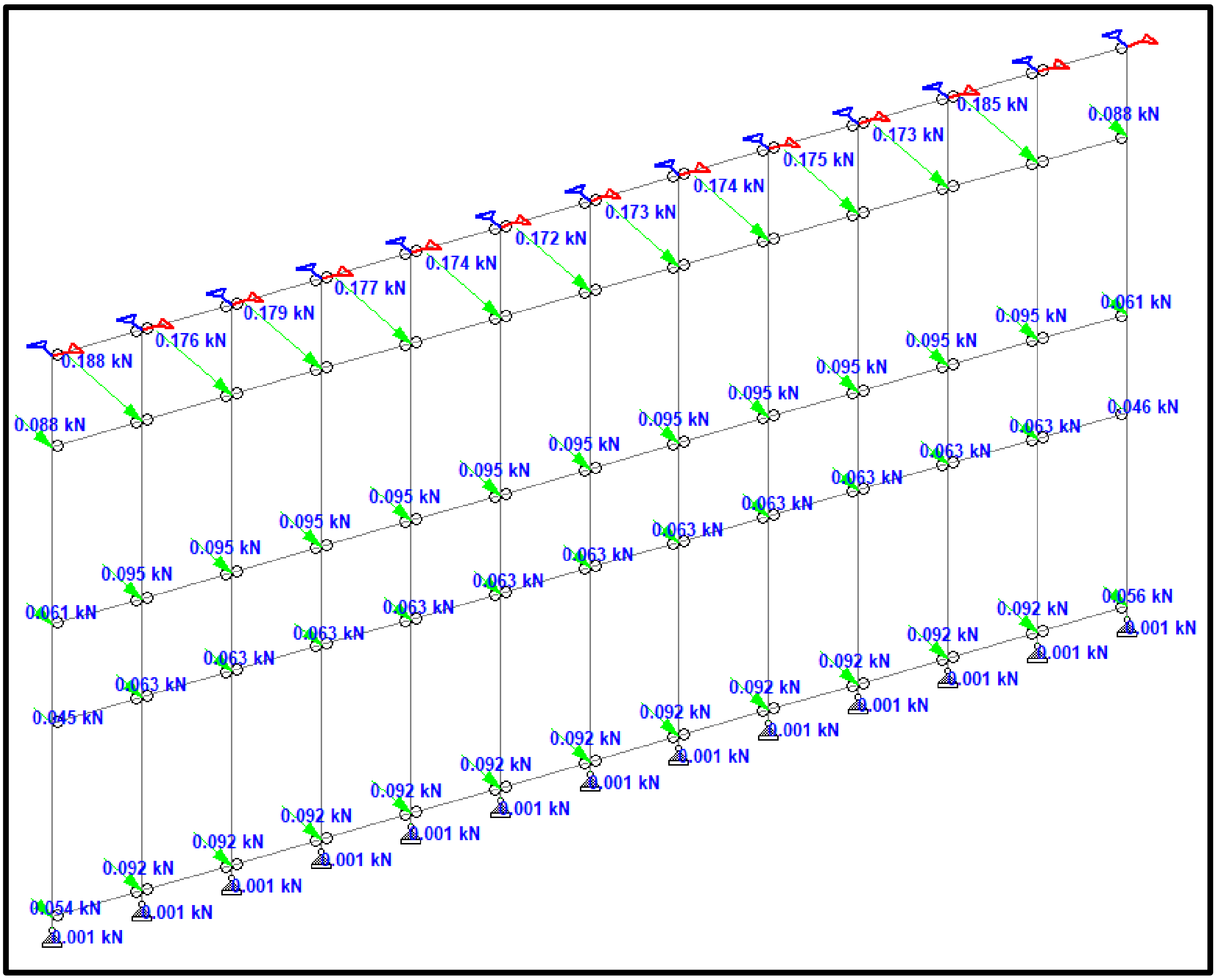
$$= [C(T_1)S_p/\mu]W_t$$

$$= [k_pZC_h(T_1)S_p/\mu]W_t$$

Factors	Abbrivi.	Value	Unit	Remarks
Probability Factor	K_p =	1	-	As per Annual probabolity P = 1/500
Hazard factor	Z =	0.08	-	As per Sydney location
Spectral time period for T_1	C_h(T1) =	2.08	sec	As per class C soil
Structural performance factor	S_p =	0.77	-	For Steel OMRF
Structural ductility factor	μ =	2	-	For Steel OMRF
Seismic weight of structure	W_t =	58	kN	Considering Dead load + 0.3x Imposed load
Horizontal design action coefficient	C_d(T1) =	0.0641	-	-
Horizontal equivelent static base shear	V =	3.7	kN	-



Earthquake Load in X direction



Earthquake Load in Z direction

6. LOAD COMBINATIONS

Load combinations as per AS/NZS 1170.0.2002 Structural Design Actions

Design load combinations

101. 1.35 G
102. 1.2 G + 1.5 Q
103. 1.2 G + WL-Z (PRESSURE) + 0.6Q
104. 0.9 G + WL-Z (PRESSURE)
105. 1.2 G + WL-Z (SUCTION) + 0.6Q
106. 0.9 G + WL-Z (SUCTION)
107. 1.0 G + EQX + 0.6 Q
108. 1.0 G - EQX + 0.6 Q
109. 1.0 G + EQZ + 0.6 Q
110. 1.0 G - EQZ + 0.6 Q

Service load combinations

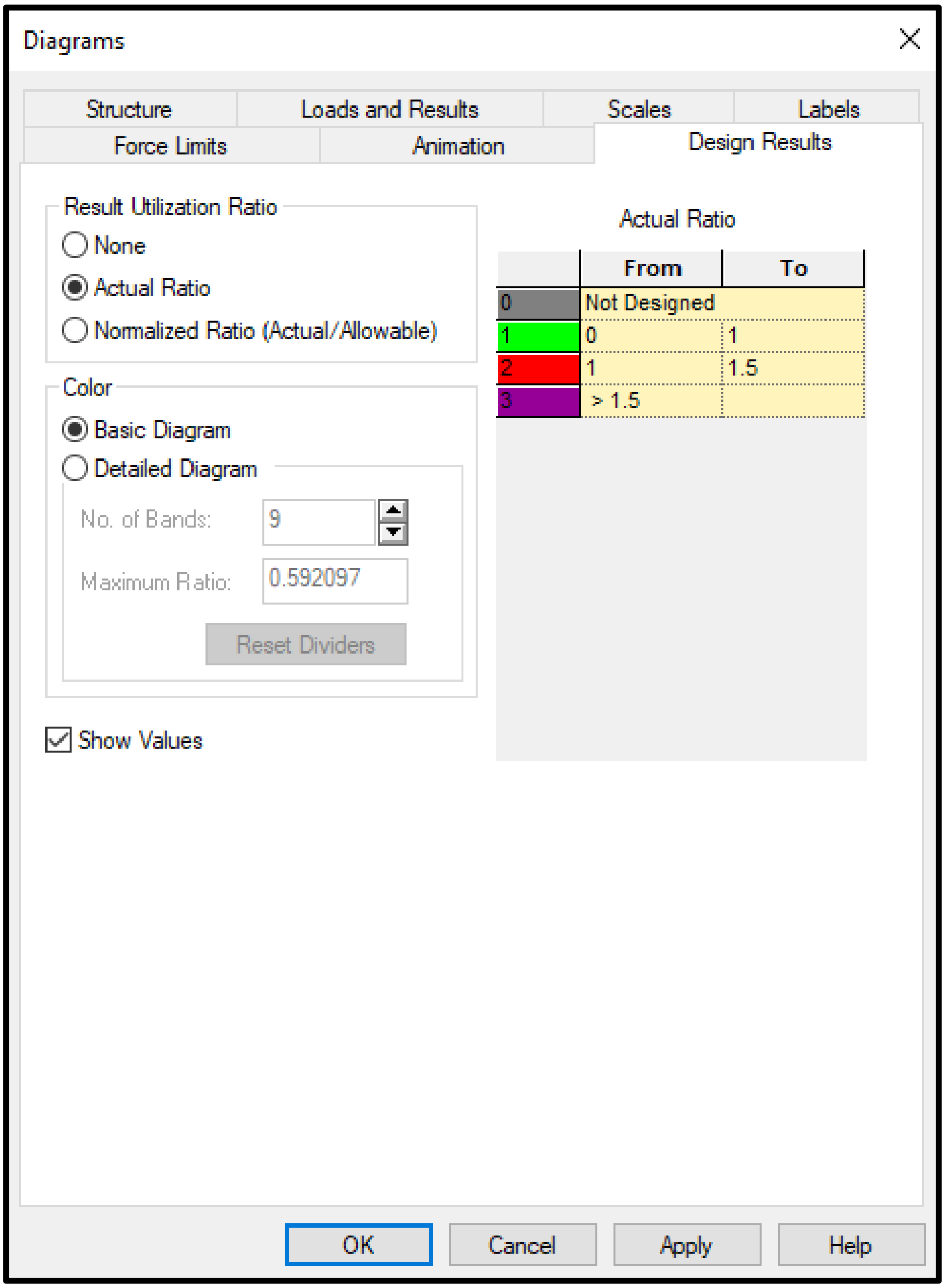
201. 1.0 G
202. 1.0 G + 0.7 Q
203. 1.0 G + 0.65WL-Z (PRESSURE) + 0.6 Q
204. 1.0 G + 0.65WL-Z (SUCTION) + 0.6 Q
205. 1.0 G + WL-Z (PRESSURE)
206. 1.0 G + WL-Z (SUCTION)

207 1.0 G + EQX
208 1.0 G - EQX
209 1.0 G + EQZ
210 1.0 G – EQZ

7. ANALYSIS & DESIGN RESULTS

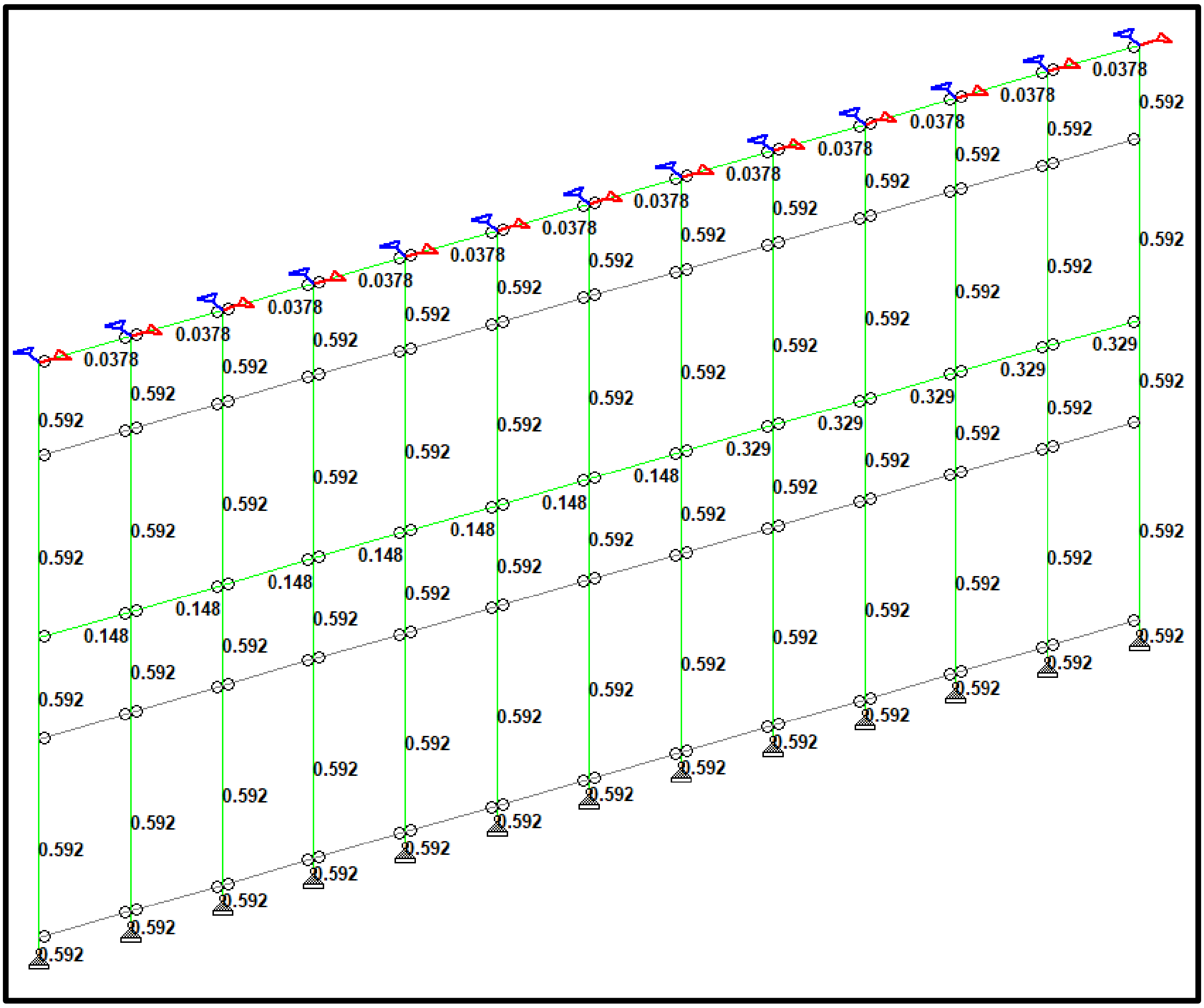
7.1 UTILITY CHECK

Below images shows value ranges for utility ratios & colored diagrams to understand utilization of structural members.



Utility ratio ranges for detailed diagram

Below image shows that failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors, if any. It can be seen from below image that all members are green. Hence, all members have passed in design.



Detailed colored diagram of utility ratios

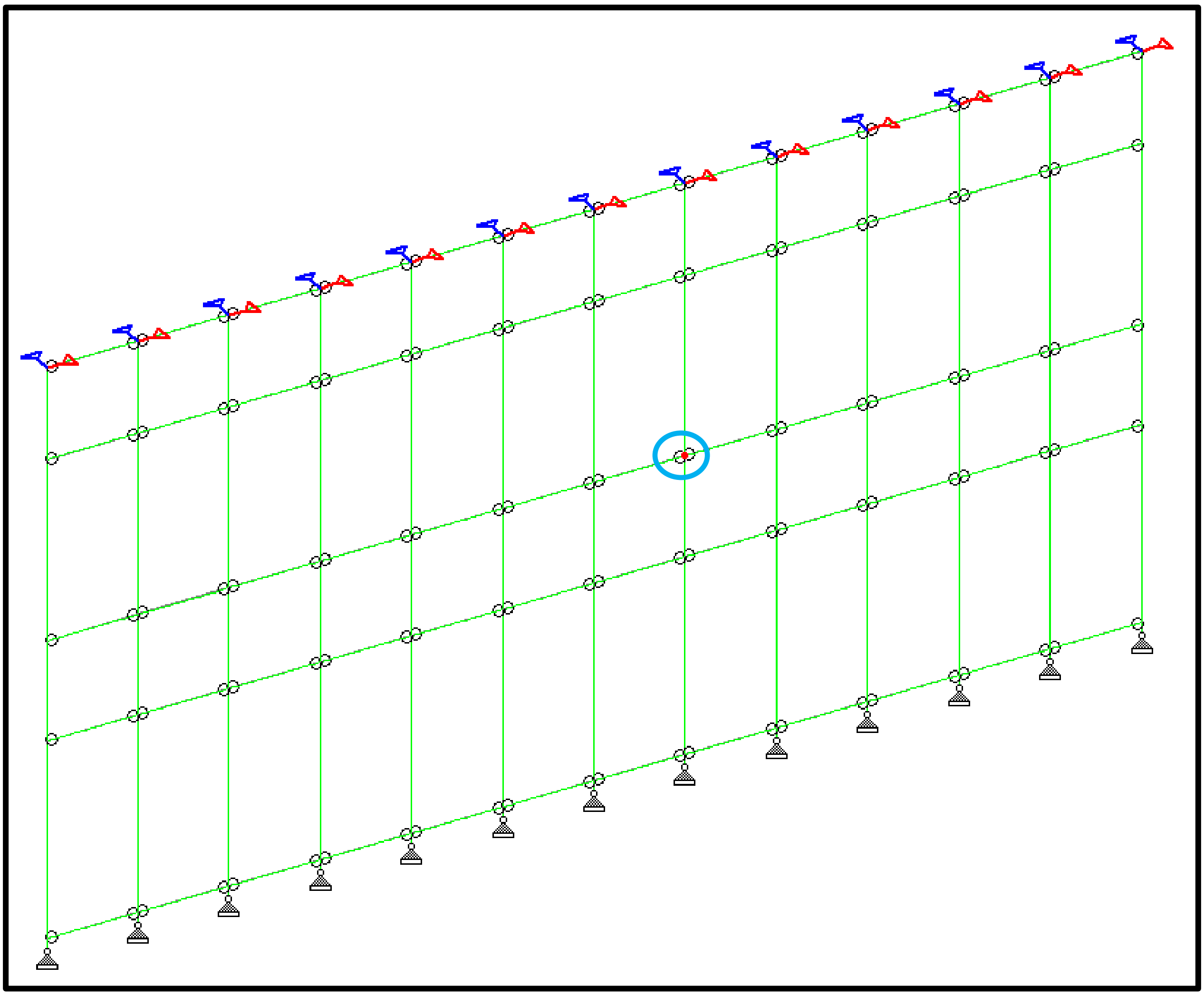
7.2 DEFLECTION CHECK

- **<u>Horizontal Deflection</u>**

Refer below table shows nodal displacement for serviceability load combinations.

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	46	207 1.0 G + E	**1.497**	-0.034	-0.751	1.675	0.000	0.000	0.000
Min X	4	208 1.0 G + E	**-1.017**	-0.019	0.000	1.017	-0.000	-0.000	0.000
Max Y	1	201 1.0 G	0.000	**0.000**	0.000	0.000	0.000	-0.000	-0.000
Min Y	12	205 1.0 G +	0.000	**-0.071**	0.000	0.071	0.004	0.000	0.000
Max Z	46	206 1.0 G +	0.241	-0.030	**17.030**	17.031	-0.001	-0.000	0.000
Min Z	46	203 1.0 G + 0.	0.241	-0.043	**-16.843**	16.845	0.001	0.000	0.000
Max rX	43	206 1.0 G +	0.000	0.000	0.000	0.000	**0.007**	-0.000	-0.000
Min rX	43	203 1.0 G + 0.	0.000	0.000	0.000	0.000	**-0.007**	0.000	-0.000
Max rY	73	206 1.0 G +	0.000	0.000	0.000	0.000	0.004	**0.002**	-0.000
Min rY	1	206 1.0 G +	0.000	0.000	0.000	0.000	0.004	**-0.002**	-0.000
Max rZ	48	207 1.0 G + E	0.000	-0.049	0.000	0.049	0.000	0.000	**0.001**
Min rZ	2	207 1.0 G + E	0.079	-0.001	0.000	0.079	0.000	-0.000	**-0.001**
Max Rst	46	206 1.0 G +	0.241	-0.030	17.030	**17.031**	-0.001	-0.000	0.000

Refer below image shows deflection diagram for serviceability load combinations.

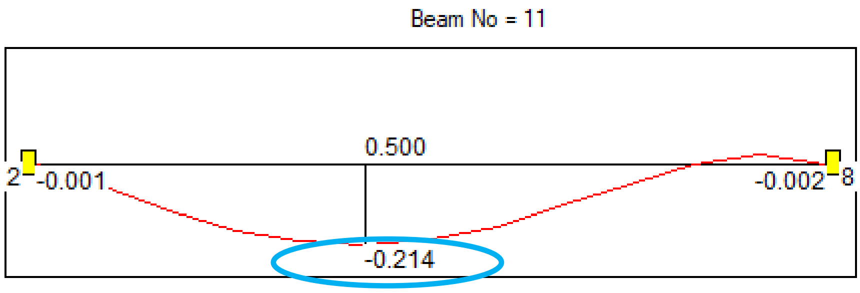


From above displacement summary,
Maximum horizontal displacement of structure in Z direction = 17.030 mm

Permissible Horizontal deflection = Height /250 = 7645 mm /250 = 30.58 mm
Actual maximum Horizontal deflection = 17.03 mm ≤ 30.58 mm ……..(Hence, OK)

- **Vertical Deflection**

Refer below image shows member deflection for serviceability load combinations.

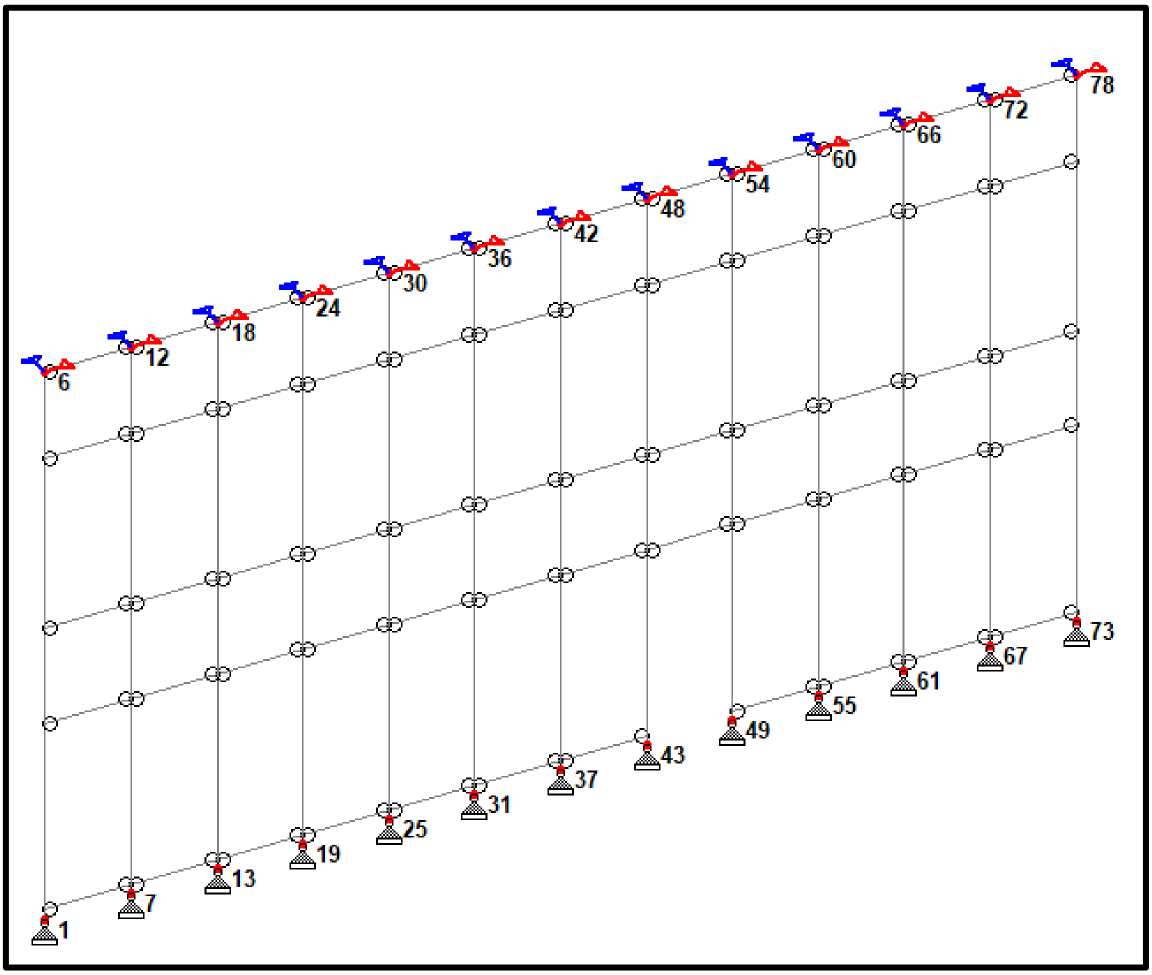


From above displacement diagram,
Maximum vertical displacement of structure in Y direction = 0.214 mm

Permissible Vertical deflection = Span /250 = 1200 mm /250 = 4.8 mm
Actual maximum Vertical deflection = 0.214 mm ≤ 4.8 mm …….(Hence, OK)

7.3 SUPPORT REACTION

Refer below image showing supports.



Supports with Node Numbers

SUPPORT REACTIONS FOR SERVICEABILITY COMBINATIONS:

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN
1	1	0.229	3.88	0
	2	0	0.158	1.579
	3	0	0.285	1.907
	4	0	-0.131	-2.633
	5	-0.074	-0.048	0
	6	0	0	-0.139
6	1	-0.036	0	0
	2	0	0	1.219
	3	-0.001	0	1.455
	4	0	0	-2.008
	5	-0.176	0	0
	6	0	0	-0.115
7	1	-0.172	6.848	0
	2	0	0.813	2.448
	3	0	1.474	3.026
	4	0	-0.68	-4.177
	5	-0.259	0.001	0
	6	0	0	-0.194
12	1	-0.016	0	0
	2	0	0	2.812
	3	-0.001	0	3.325
	4	0	0	-4.589
	5	-0.205	0	0
	6	0	0	-0.236
13	1	-0.075	6.656	0
	2	0	0.733	2.613
	3	0	1.33	3.209
	4	0	-0.613	-4.429
	5	-0.224	0	0
	6	0	0	-0.203
18	1	-0.015	0	0
	2	0	0	2.811
	3	0	0	3.323
	4	0	0	-4.587
	5	-0.206	0	0
	6	0	0	-0.227
19	1	-0.046	6.703	0
	2	0	0.756	2.652
	3	0	1.371	3.253
	4	0	-0.633	-4.49
	5	-0.213	0	0

	6	0	0	-0.204
24	1	-0.016	0	-0.002
	2	0	0	2.779
	3	0	0	3.285
	4	0	0	-4.538
	5	-0.206	0	0
	6	0	0	-0.226
25	1	-0.031	6.674	0.001
	2	0	0.74	2.66
	3	0	1.342	3.262
	4	0	-0.619	-4.5
	5	-0.21	0	0
	6	0	0	-0.204
30	1	-0.016	0	-0.005
	2	0	0	2.748
	3	0	0	3.249
	4	0	0	-4.493
	5	-0.207	0	0
	6	0	0	-0.224
31	1	-0.007	6.635	0.013
	2	0	0.715	2.676
	3	0	1.297	3.29
	4	0	-0.599	-4.52
	5	-0.209	0	0
	6	0	0	-0.204
36	1	-0.016	0	-0.023
	2	0	0	2.715
	3	0	0	3.197
	4	0	0	-4.453
	5	-0.207	0	0
	6	0	0	-0.221
37	1	0.066	6.597	0.053
	2	0	0.691	2.739
	3	0	1.253	3.393
	4	0	-0.578	-4.594
	5	-0.209	0	0
	6	0	0	-0.204
42	1	-0.017	0	0.063
	2	0	0	2.85
	3	0	0	3.418
	4	0	0	-4.612
	5	-0.207	0	0
	6	0	0	-0.219
43	1	0.292	6.613	-0.334
	2	0	0.695	3.858

	3	0	1.26	5.214
	4	0	-0.581	-5.915
	5	-0.209	0	0
	6	0	0	-0.204
48	1	-0.019	0	0.145
	2	0	0	2.974
	3	0	0	3.62
	4	0	0	-4.759
	5	-0.207	0	0
	6	0	0	-0.22
49	1	0.096	6.632	0.021
	2	0	0.713	2.847
	3	0	1.293	3.568
	4	0	-0.596	-4.722
	5	-0.21	0	0
	6	0	0	-0.204
54	1	-0.018	0	0.078
	2	0	0	2.878
	3	0	0	3.461
	4	0	0	-4.647
	5	-0.206	0	0
	6	0	0	-0.222
55	1	0.013	6.641	0.017
	2	0	0.724	2.68
	3	0	1.312	3.297
	4	0	-0.605	-4.522
	5	-0.213	0	0
	6	0	0	-0.204
60	1	-0.017	0	-0.017
	2	0	0	2.755
	3	0	0	3.247
	4	0	0	-4.51
	5	-0.206	0	0
	6	0	0	-0.223
61	1	0.008	6.622	0.003
	2	0	0.711	2.617
	3	0	1.289	3.216
	4	0	-0.595	-4.434
	5	-0.224	0	0
	6	0	0	-0.202
66	1	-0.017	0	-0.007
	2	0	0	2.8
	3	0	0	3.305
	4	0	0	-4.574
	5	-0.205	0	0

	6	0	0	-0.225
67	1	0.079	6.807	0
	2	0	0.768	2.448
	3	0	1.393	3.026
	4	0	-0.642	-4.177
	5	-0.26	-0.001	0
	6	0	0	-0.193
72	1	-0.016	0	-0.003
	2	0	0	2.808
	3	0.001	0	3.318
	4	0	0	-4.584
	5	-0.205	0	0
	6	0	0	-0.234
73	1	-0.247	3.929	0
	2	0	0.172	1.579
	3	0	0.312	1.907
	4	0	-0.144	-2.632
	5	-0.075	0.048	0
	6	0	0	-0.142
78	1	0.013	0	0
	2	0	0	1.219
	3	0.001	0	1.454
	4	0	0	-2.007
	5	-0.176	0	0
	6	0	0	-0.115

Date: 22nd April 2024
Revision – R6

DESIGN CALCULATION REPORT

FOR

GLAZED ROOF

AT

CHILD CARE CENTRE,

1458 PACIFIC HIGHWAY,

TURRAMURRA, NSW 2074

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Table of Contents

1. INTRODUCTION

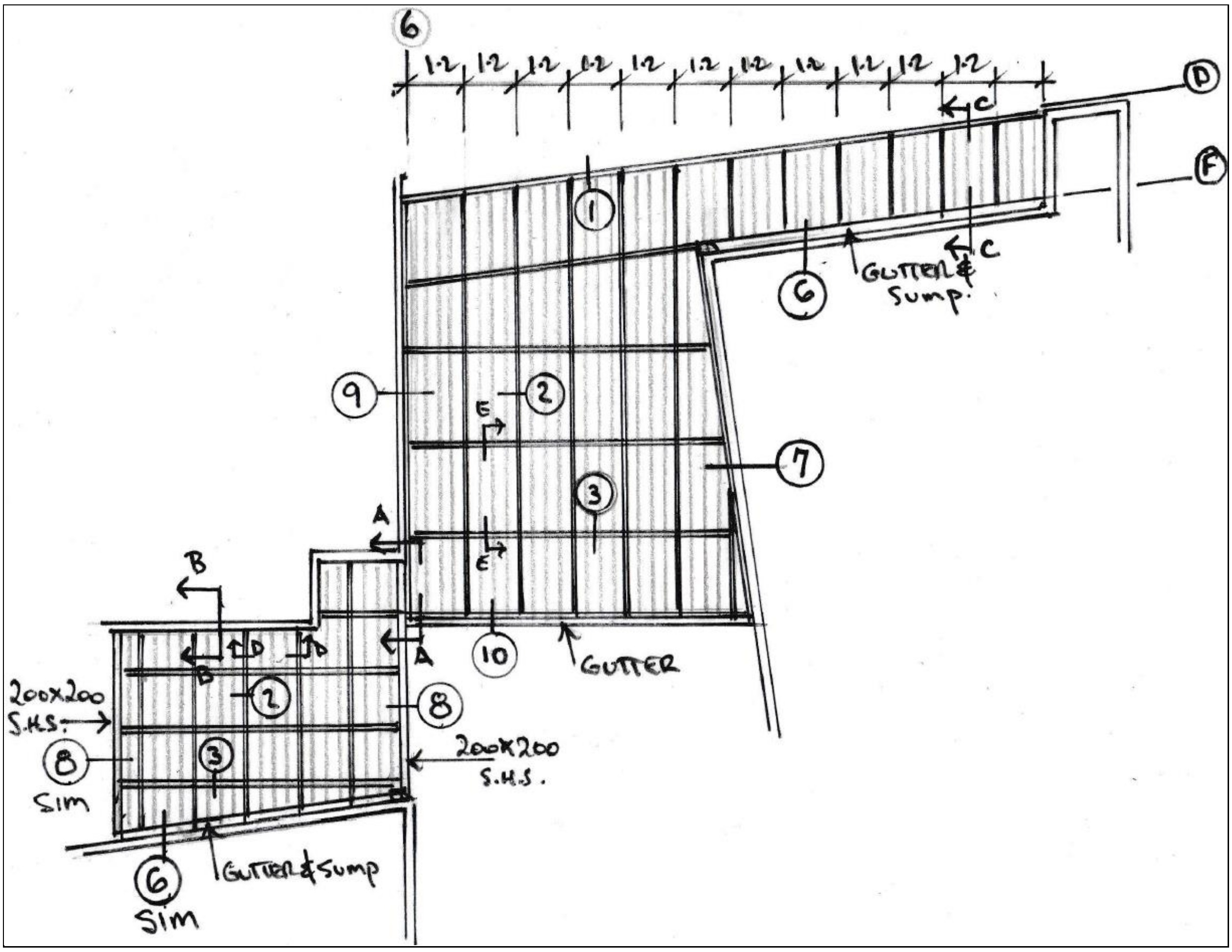
This design calculation is to justify the structural elements of Glazed Roof in the proposed Child Care Centre in Turramurra.

The facade system is designed to sustain the dead load, live load, earthquake load and wind load according to Structural design actions_ Wind actions as per AS/NZS 1170:2:2021.

The facade system will be fixed to parent concrete structure using post fixed anchors.

Load path for Glazed Roof

Load Path ⇨ Loading on Glass ⇨ Aluminum/Steel Frame ⇨ Fixings ⇨ Concrete Structure



Framing Plan for Glazed Roof

2. MATERIAL

Sr. No.	Member	Remarks	Grade (MPa)
1	SHS_200x200x5	Framing Main Member	Steel-350/450
2	RHS_200x100x5	Framing Main Frame	Steel-350/450
2	RHS_200x100x6	Framing Main Frame	Steel-350/450
3	RHS_100x50x3.0	Framing Secondary Member	Aluminium-110
4	RHS_100x50x1.8	Framing Secondary Member	Aluminium-110

3. CODES CONSIDERED

Following codes are referred for analysis and design of Glazed Roof structure.

- AS/NZS 1170.0. 2002 - Structural Design Actions Part 0: General principles
- AS/NZS 1170.1. 2002 - Structural Design Actions Part 1: Permanent, imposed, and other actions
- AS/NZS 1170.2.2021 - Structural Design Actions Part 2: Wind Actions
- AS/NZS 4100:1998 - Steel Structures
- AS/NZS 2047:1999 - Windows in Buildings Selection & Installation
- AS/NZS 1664:1997 - Aluminium Structures_Part-1
- AS/NZS 1170.4 - Structural Design Actions Part 4: Earthquake actions
- AS 1288 - Glass Buildings
- AS 5216 - Design of Post Installed & Cast-In Fastening in Concrete
- AS 1530.4 - Fire Resistance Tests for Elements of Construction
- AS1288 – 2006 -Glass-in-buildings-Selection-and-installation

4. DESIGN OF GLASS

Design Actions

Permanent Actions = Self-weight
Imposed Actions = 1.0 kPa
= 1.4kN vertical point load over 100x100mm
Wind Actions = +1.44 / -1.61 kPa

Summary of Items

Item	Details
12mm Laminated Glass. 6mm HS/1.52mm PVB/6mm HS.	1200mm x 2000mm
4-sided structural double-glazed glass panel.	Capral 150 St Kilda Glazing Adaptors with structural silicone.

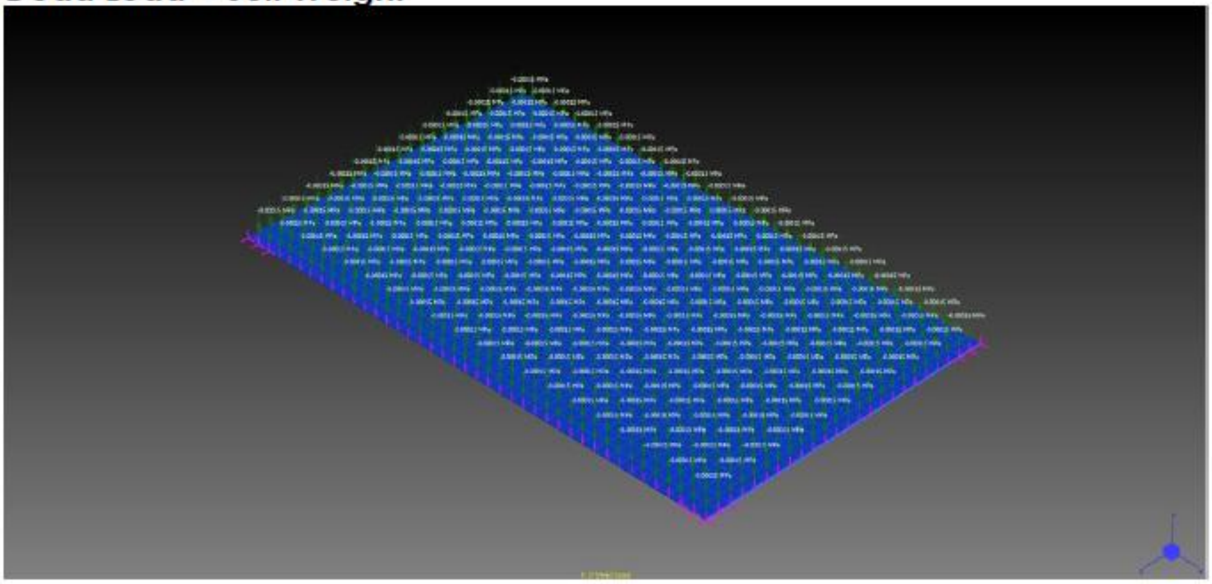
Glass Panel Design Stress

phi =	0.67		
c1 =	1.6	(1.0 ordinary annealed, 1.6 heat-strengthened, 2.5 toughened, 0.5 wired)	
c2 =	1	(1.0 untreated, 0.4 sand blasted/etched, 1.0 acid etched or patterned)	
c3 =	1		
t =	11.6	mm	
f't =	**47.2**	**MPa**	**away from edges**
f't =	**37.8**	**MPa**	**near edges**
phi. Ru =	**50.6**	**MPa**	**away from edges**
phi. Ru =	**40.5**	**MPa**	**near edges**

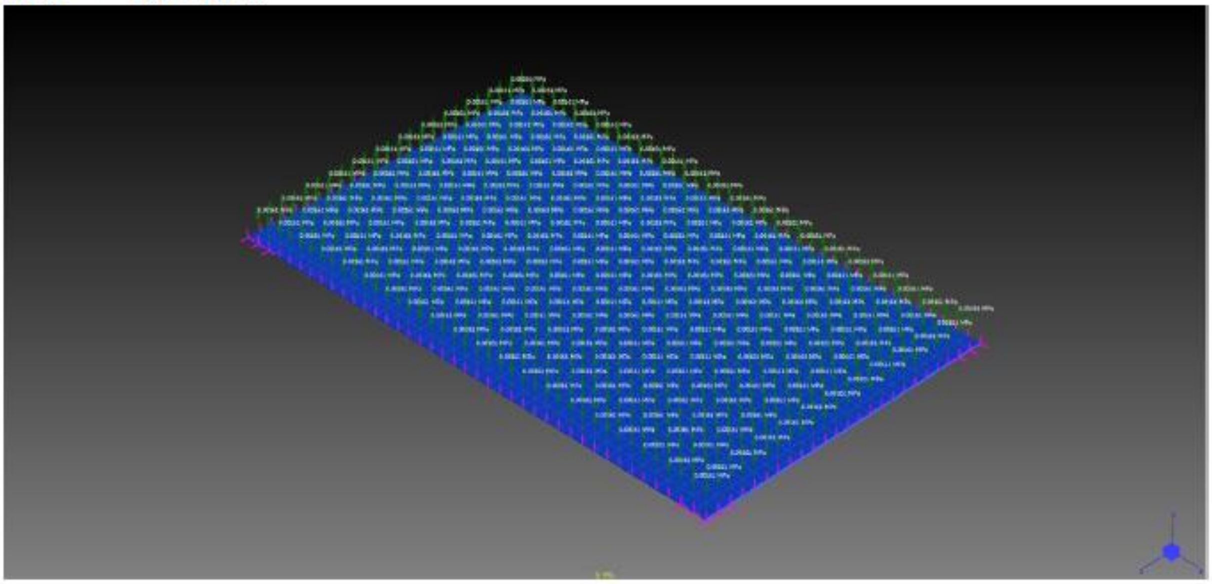
Adopt glass panel design stress phi. Ru = 40.5 MPa near edges.

Design Actions

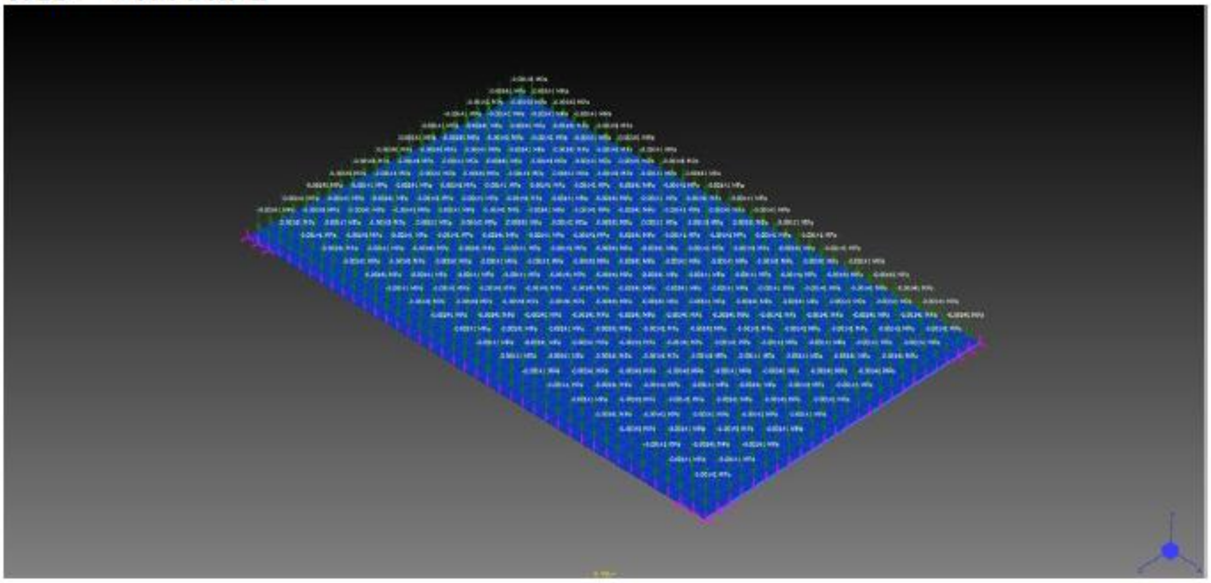
Dead Load + Self Weight



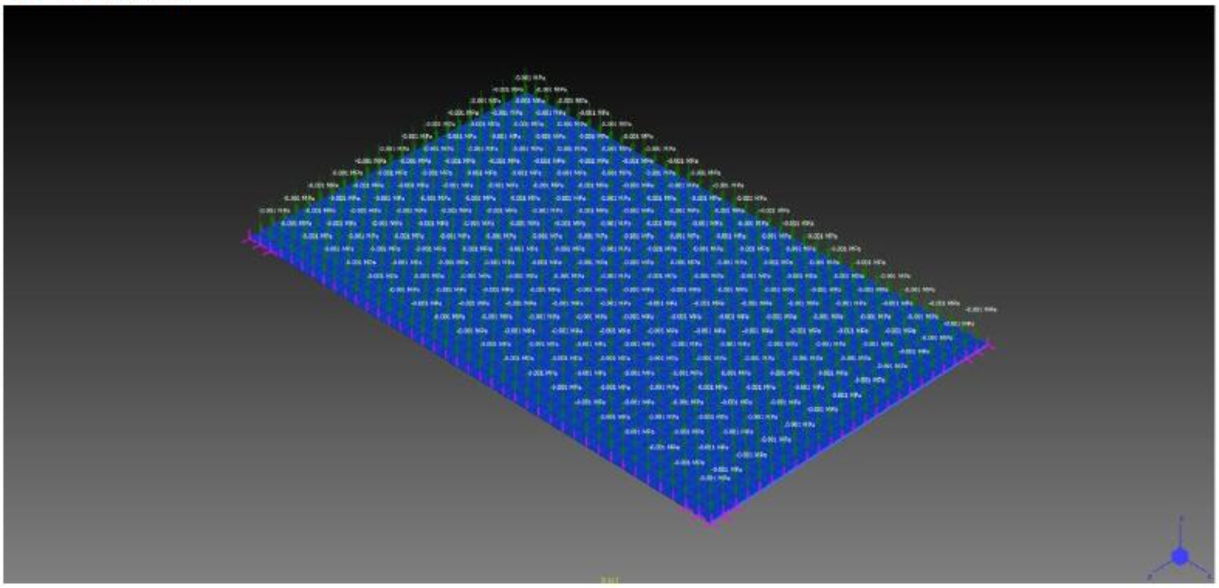
WLu = -1.61 kPa



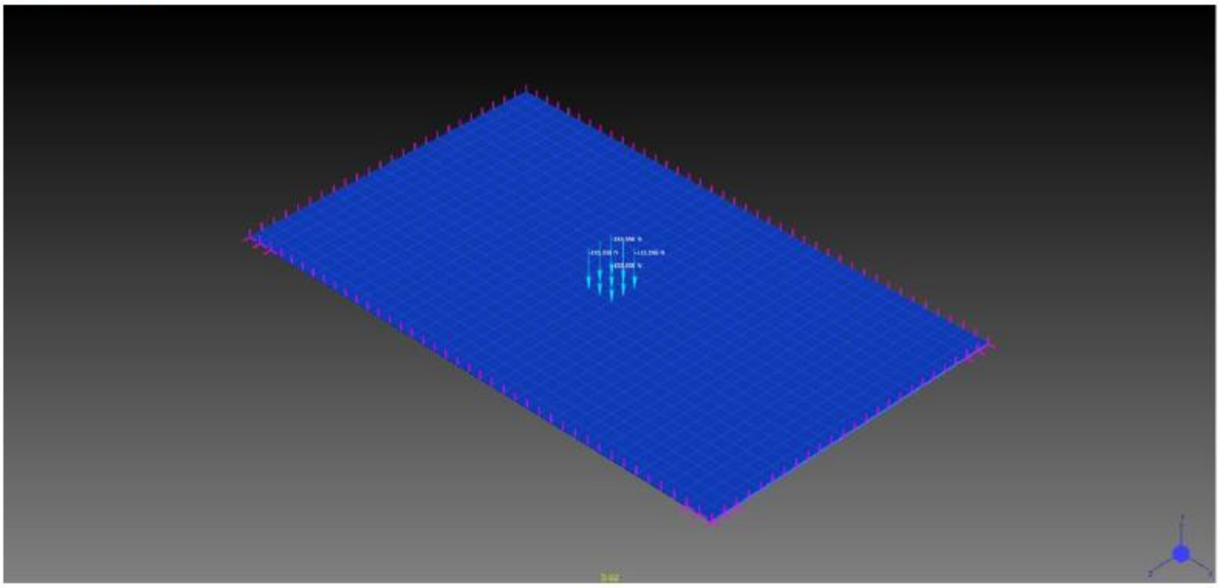
WLu = +1.44 kPa



LL1 = 1.0 kPa

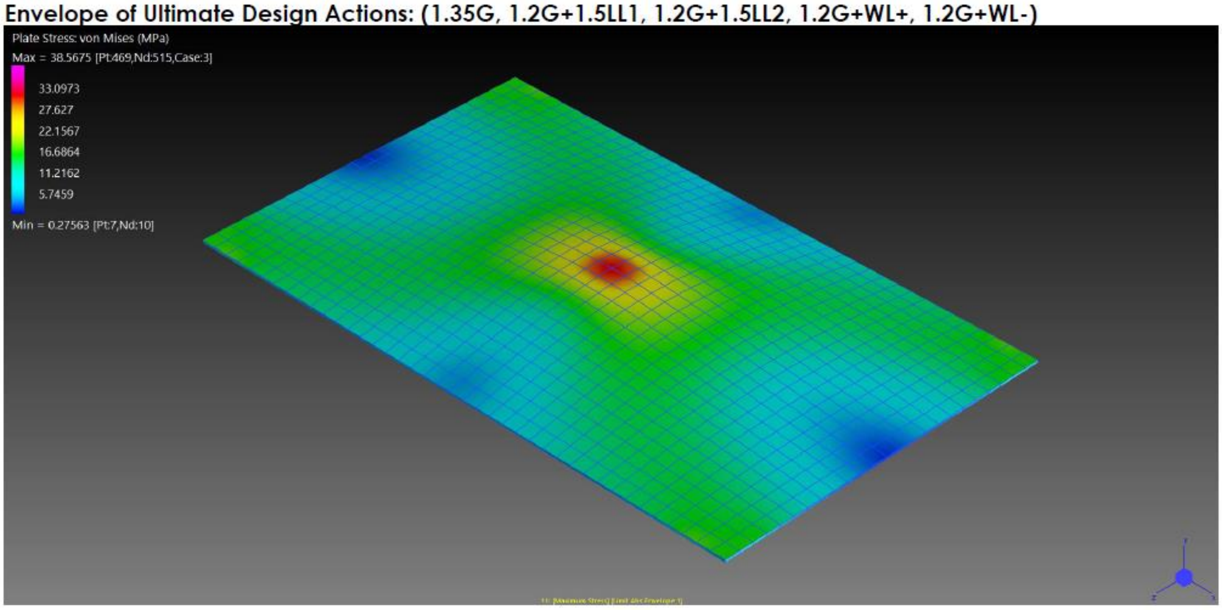


LL2 = 1.4kN

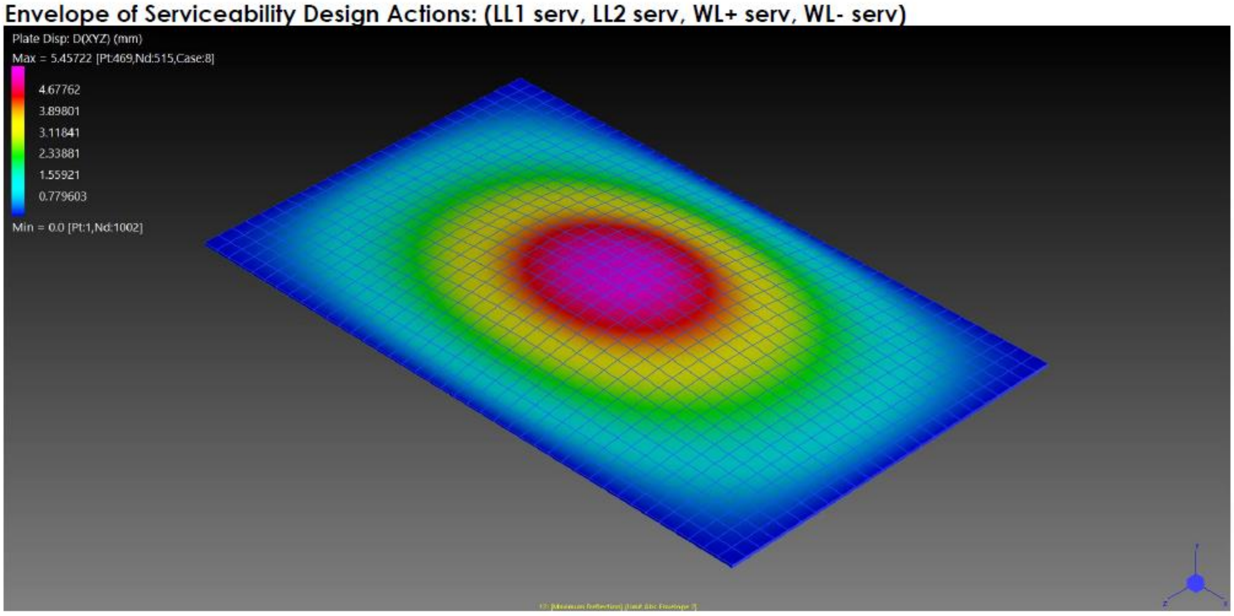


Combinations of Actions

Envelope of Ultimate Design Actions: (1.35G, 1.2G+1.5LL1, 1.2G+1.5LL2, 1.2G+WL+, 1.2G+WL-)

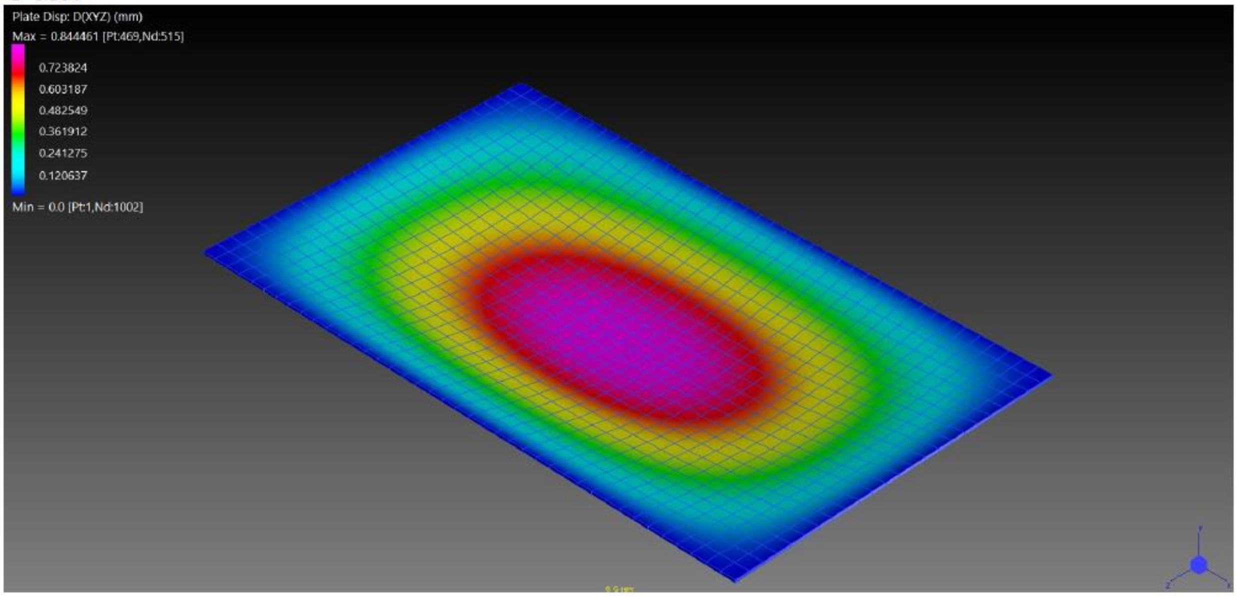


Envelope of Serviceability Design Actions: (LL1 serv, LL2 serv, WL+ serv, WL- serv)



Strength Check			
Maximum stress of glass	$\sigma_{allowable} = 40.5\ MPa$	$\sigma_{max} = 38.6\ MPa$	OK
Deflection Check			
Maximum deflection of glass	$\delta_{allowable} = \min(\frac{L}{60}, 20mm) = 20mm$	$\delta_{max} = 5.5mm$	OK

G serv

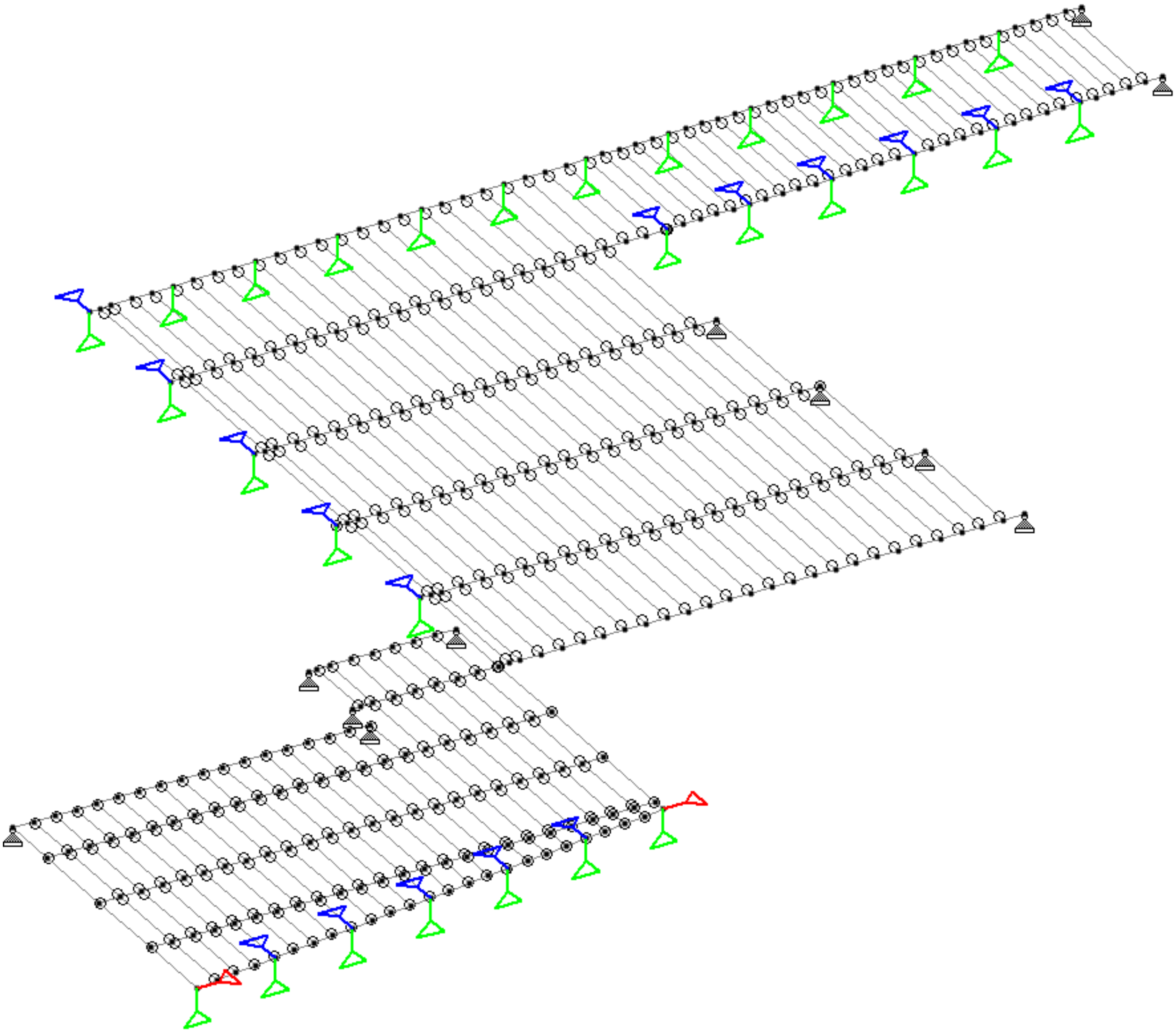


Deflection Check			
Maximum deflection of glass panel	$\delta_{max} < 1mm$	$\delta_{max} = 0.8\ mm$	OK

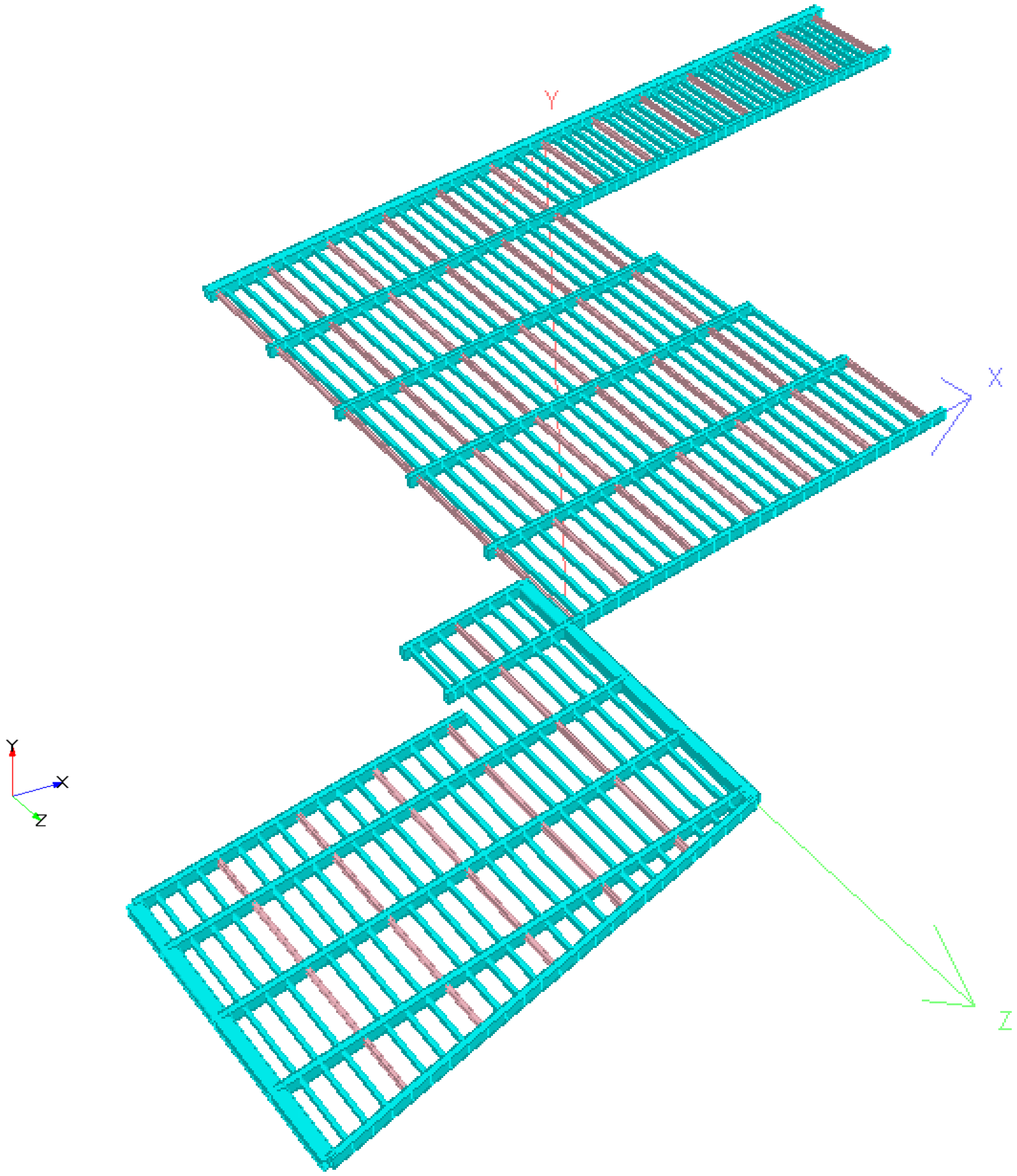
Glass design is safe in lower thickness. Hence, provided glass thickness DGU 13.52mm heat strengthened, 12 air argon and 11.52mm is safe.

5. STAAD MODELLING OF GLAZED ROOF

Refer below images showing normal & render 3D view of Glazed roof modeled in STAAD Pro software

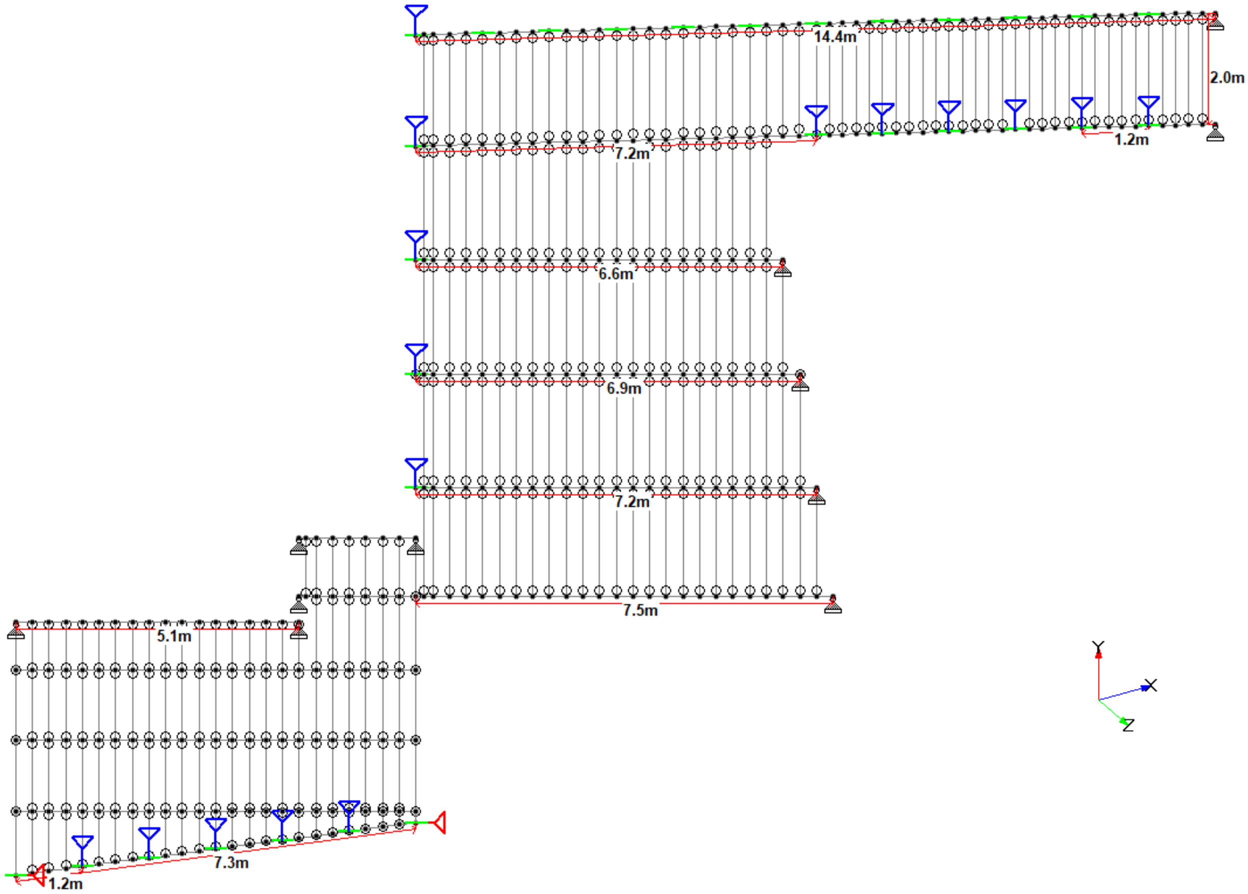


3D view of Glazed roof



3D render view of entire structure

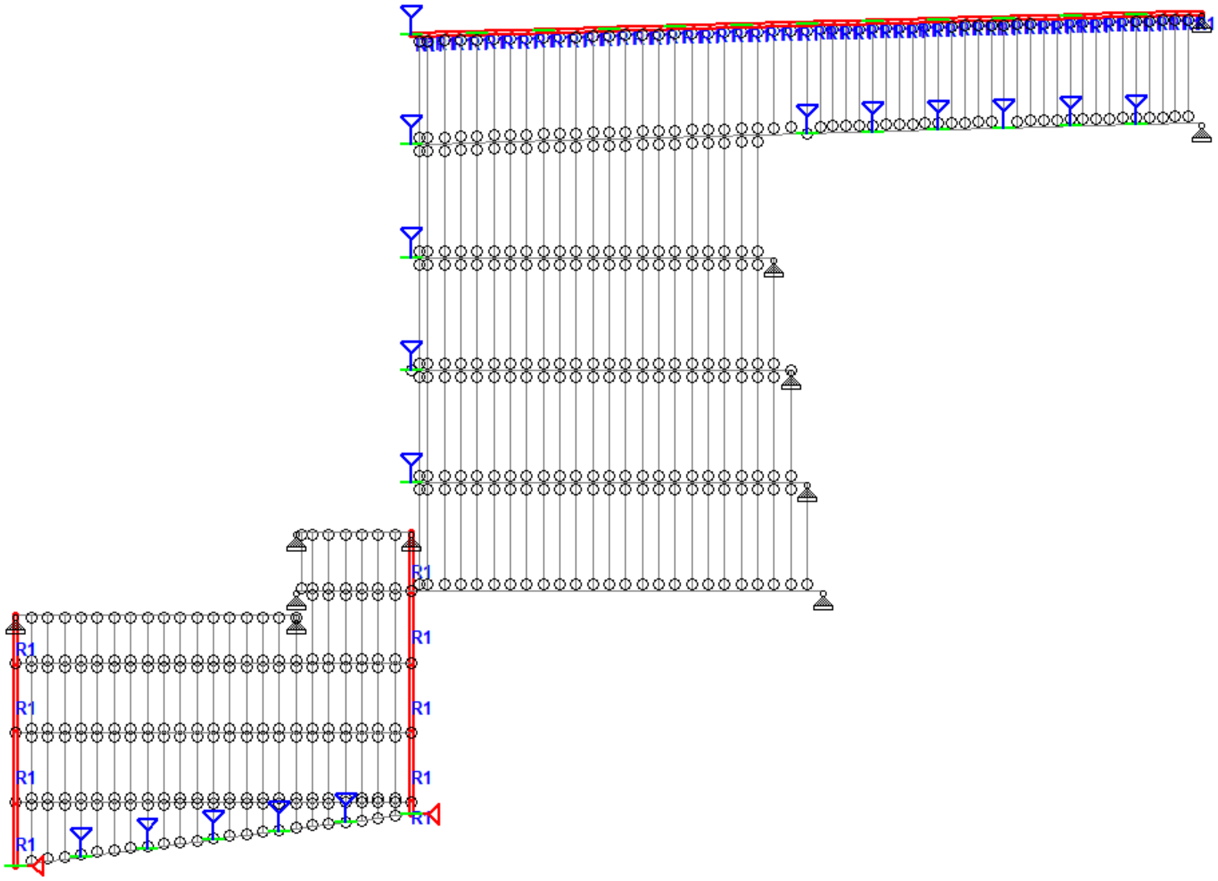
5.1. **GEOMETRY DATA**



Façade Geometry

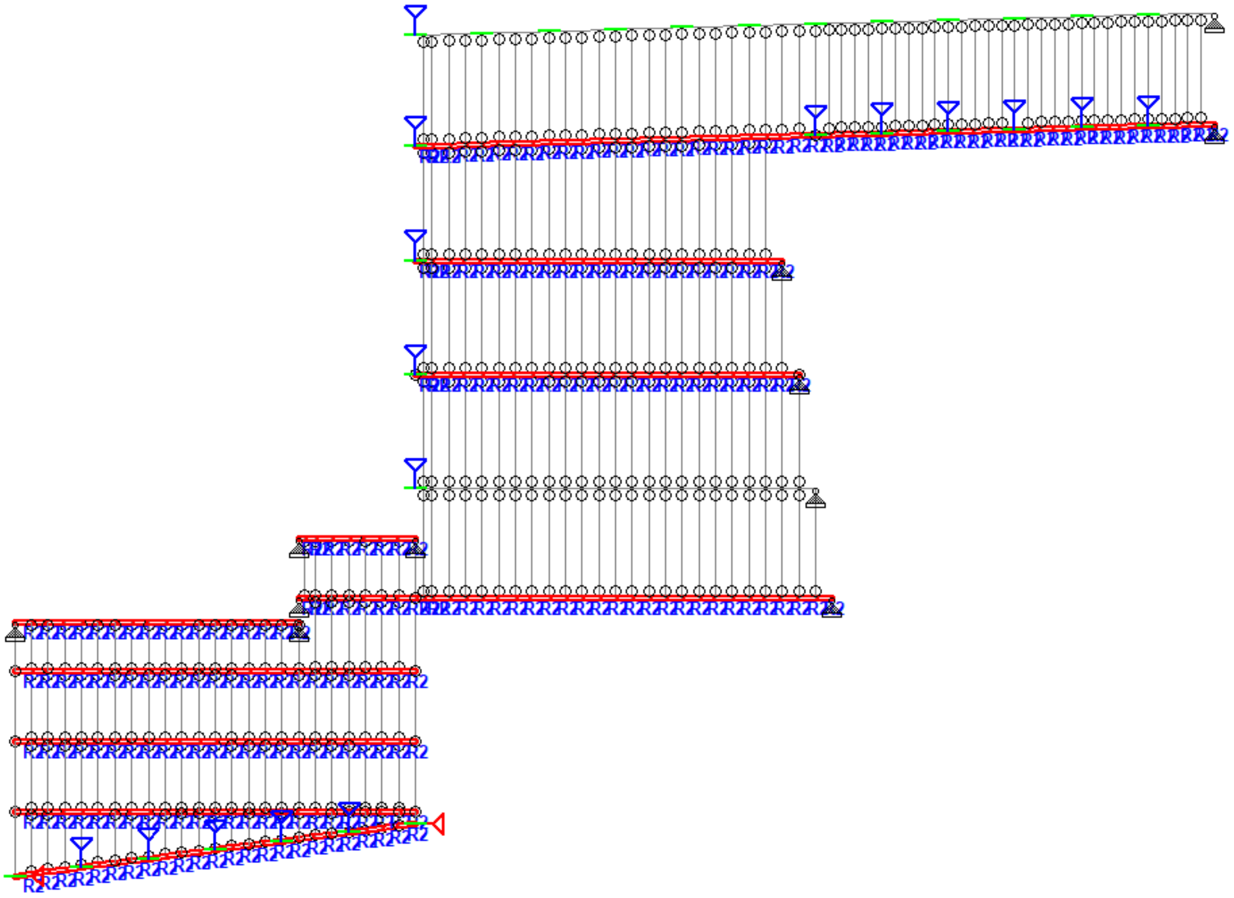
5.2. **MEMBER PROPERTIES**

1. 200 X 200 X 5.0 SHS – Steel Members:



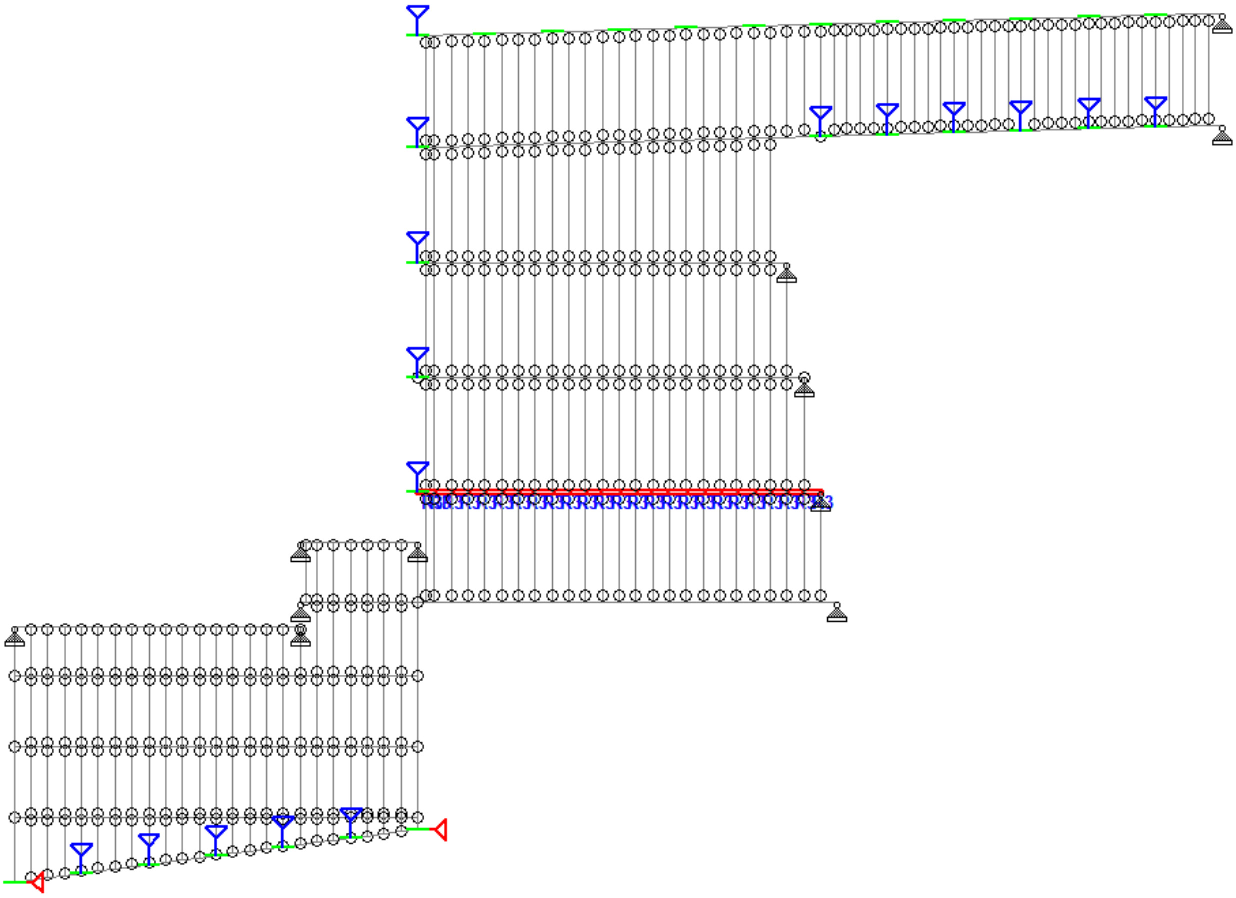
200 X 200 X 5.0 SHS

2. 200 X 100 X 5.0 RHS – Steel Members:



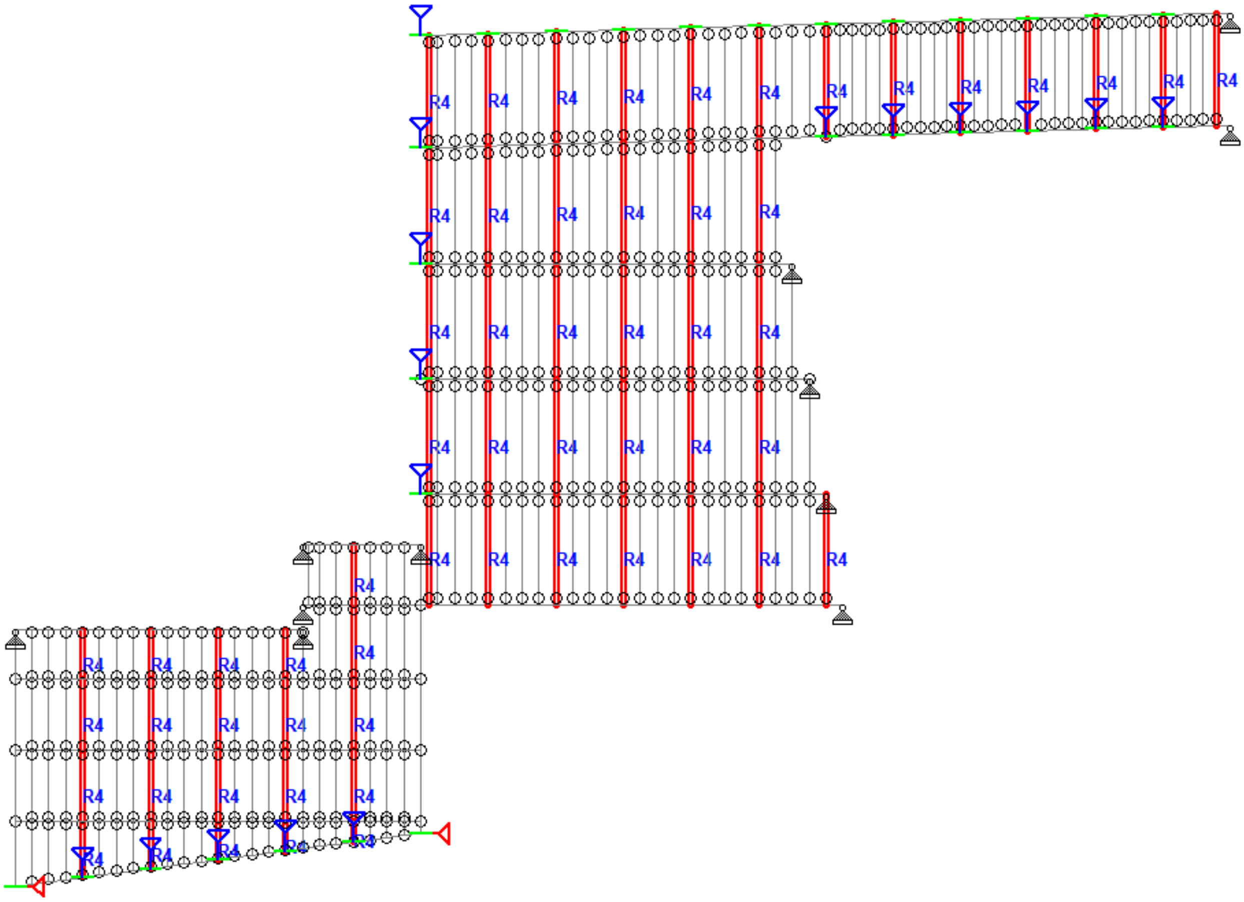
200 X 100 X 5.0 RHS

3. 200 X 100 X 6.0 RHS – Steel Members:



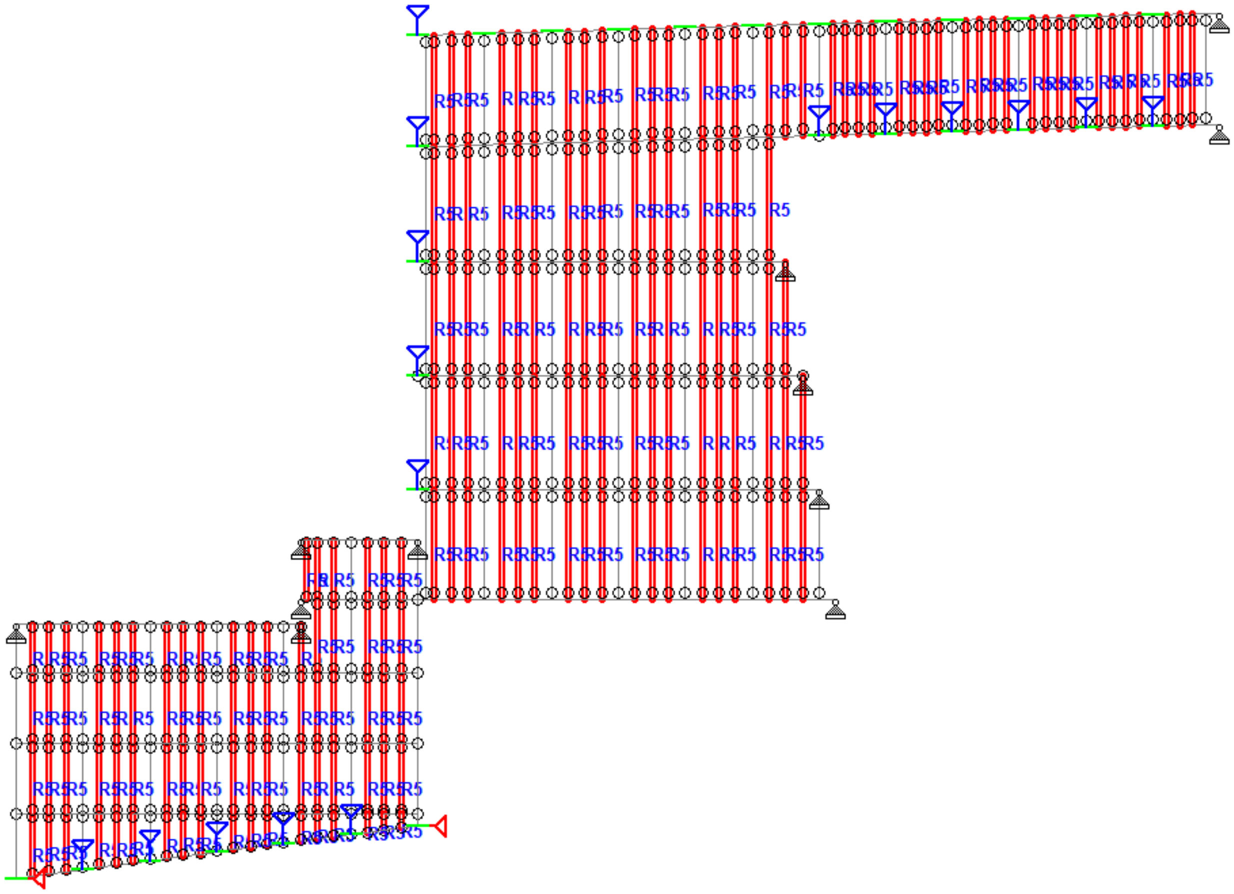
200 X 100 X 6.0 RHS

4. 100 X 50 X 3.0 RHS – Aluminium Members :



100 X 50 X 3.0 RHS - Aluminum Member Sections

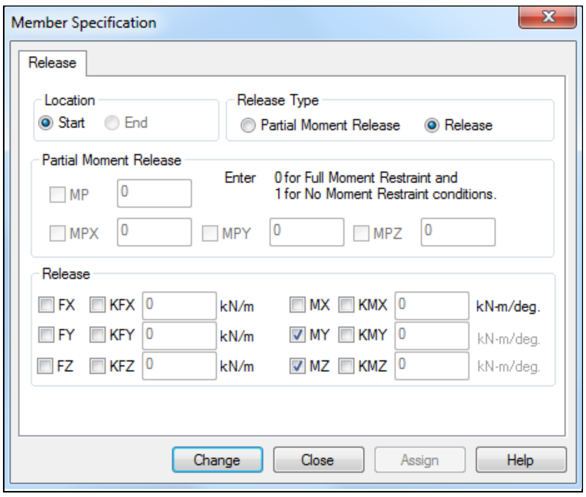
5. 100 X 50 X 1.8 RHS – Aluminium Members :

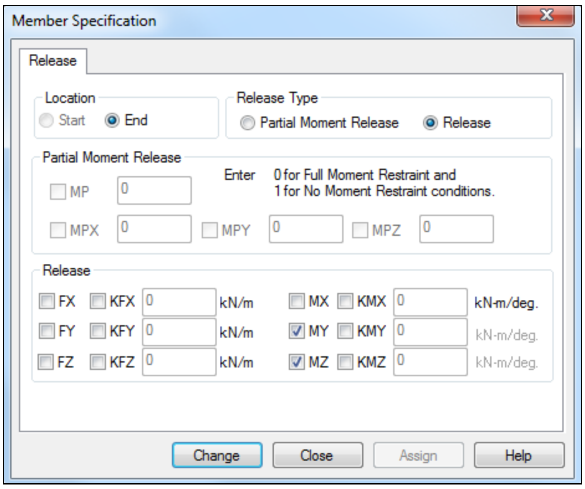


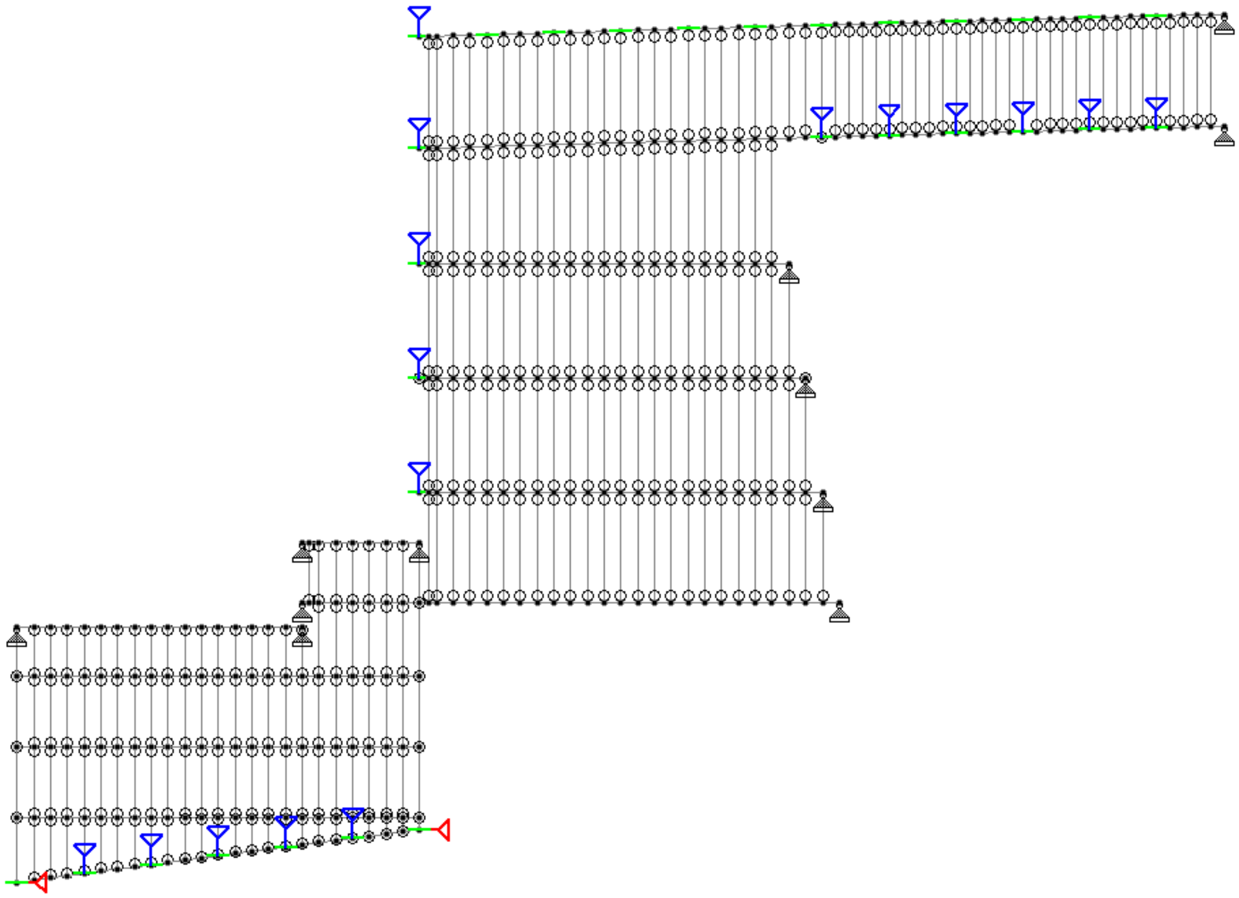
100 X 50 X 1.8 RHS - Aluminum Member Sections

5.3. <u>**MEMBER RELEASES**</u>

Refer below images shows member has been released at both ends.





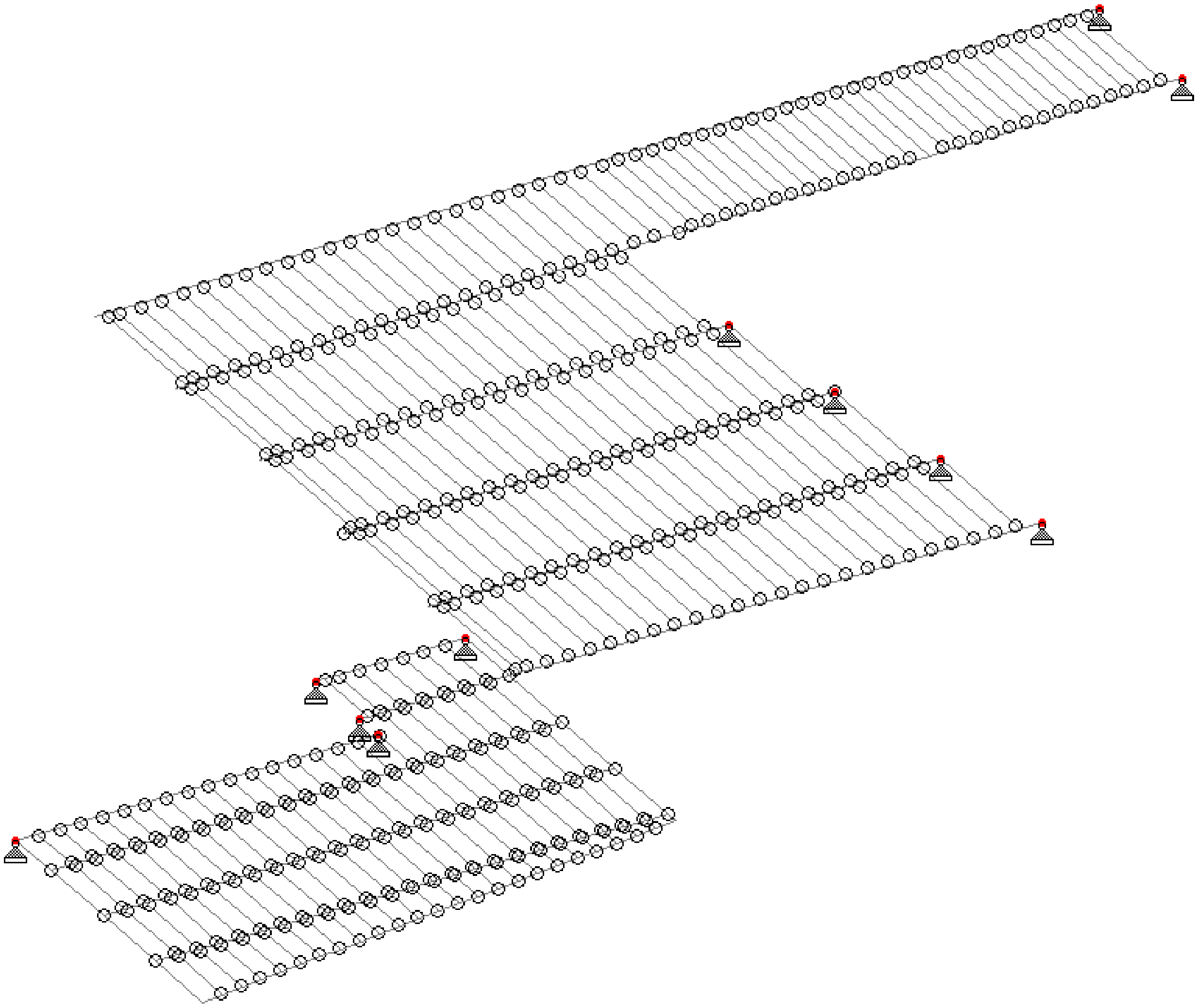


Moment Release

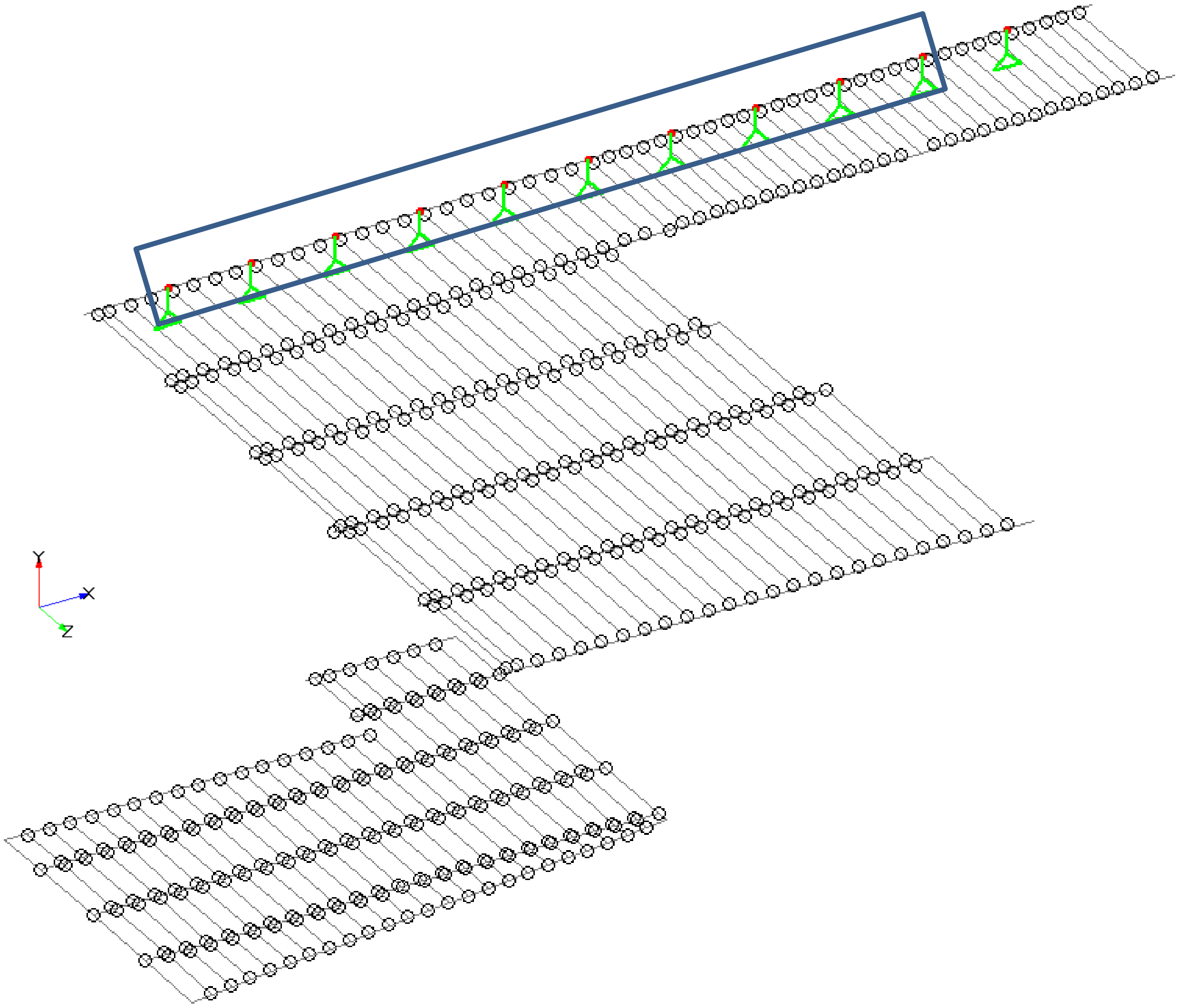
5.4. **SUPPORT CONDITION**

Pinned and fixed but vertical and horizontal supports (Glaze wall mullion support at 1.2m c/c) have been assigned in STAAD Pro Model.

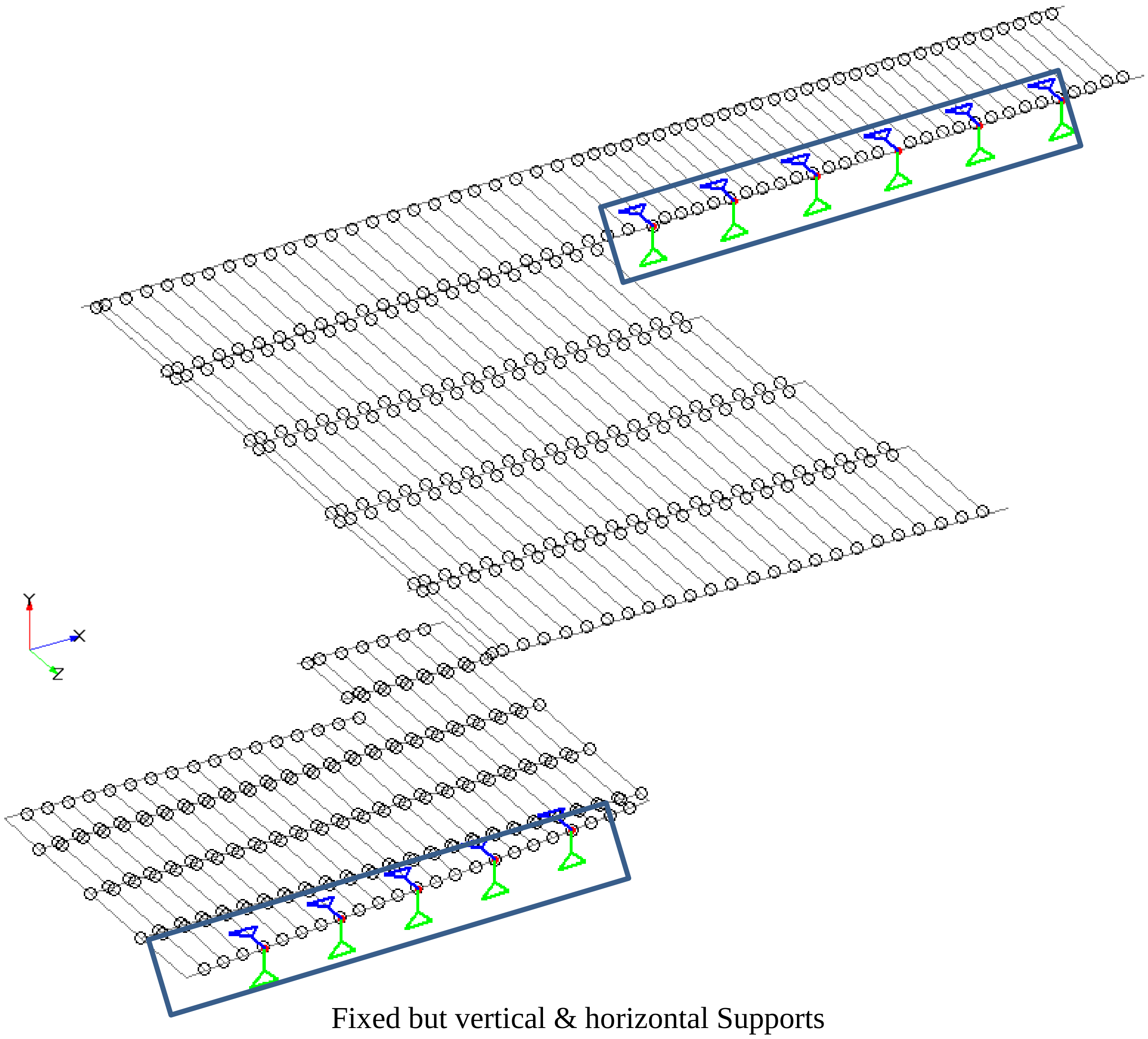
Refer below image showing location of these supports in STAAD model.



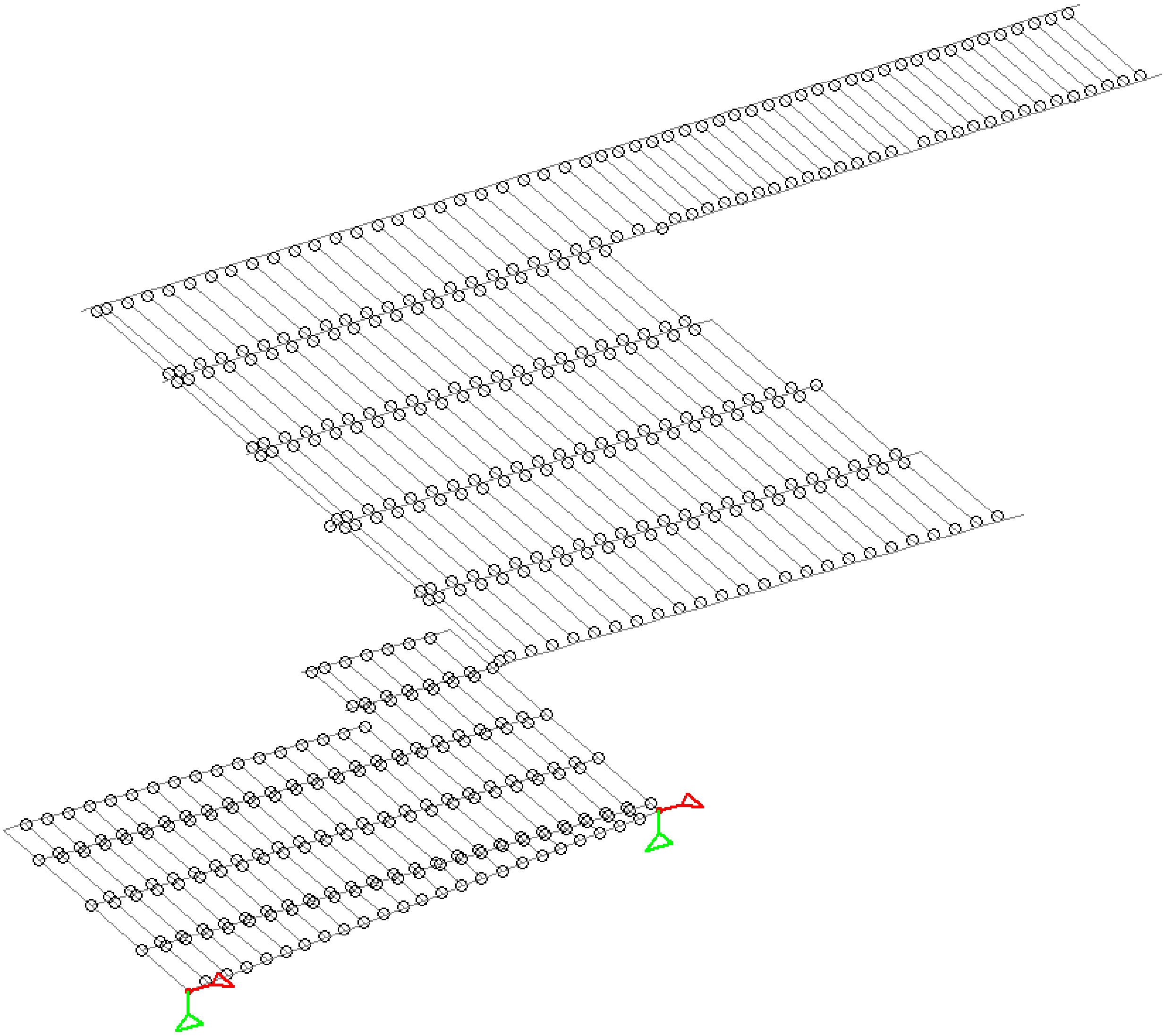
Pinned Supports



Fixed but vertical Supports



Fixed but vertical & horizontal Supports



Fixed but vertical & horizontal Supports

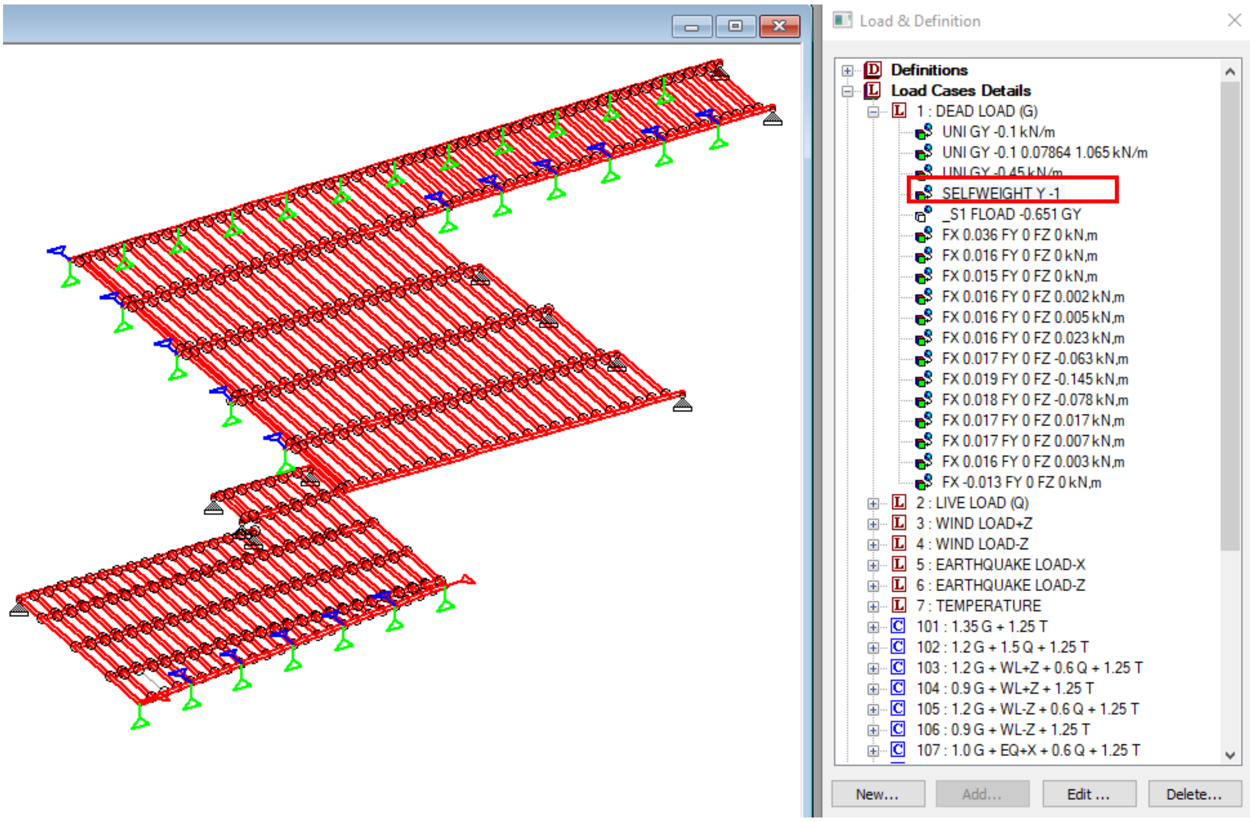
6. LOADING

Load cases:

1. DL: Dead Load
2. LL: Live load
3. WL: Wind Load (Pressure)
4. WL: Wind Load (Suction)
5. EL: Earthquake Load-X direction
6. EL: Earthquake Load-Z direction
7. TL: Temperature Load

6.1. <u>**DL: Dead Load**</u>

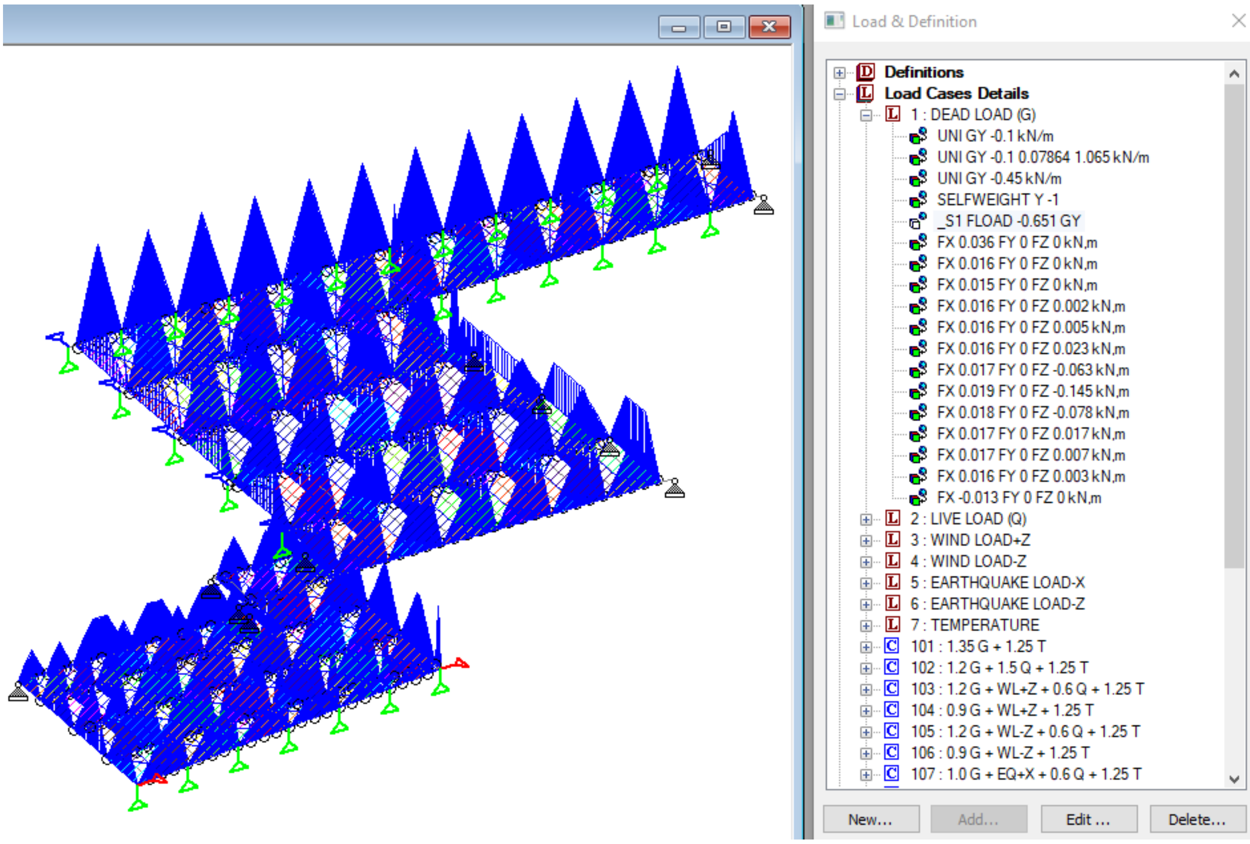
1. Self-weight of framing members



Self-weight of Members

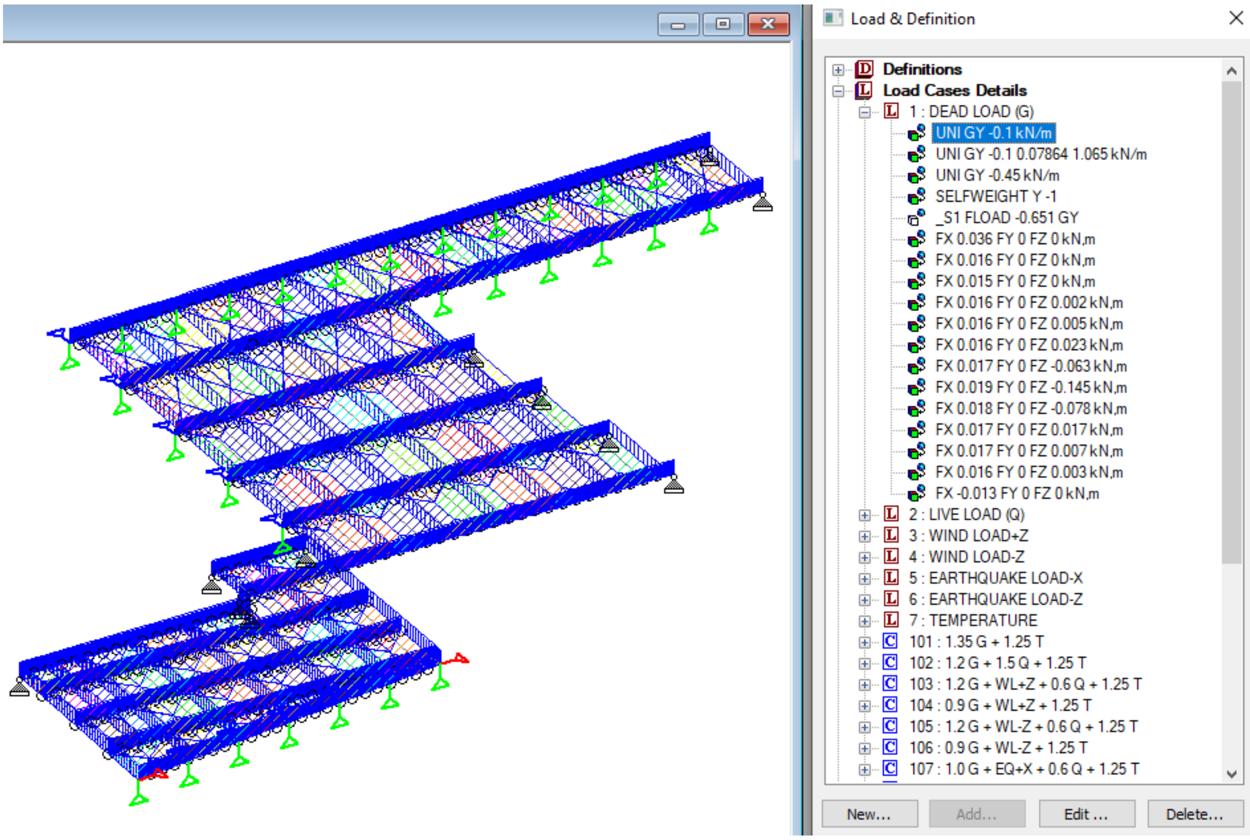
2. Glass Panel load for Glaze roof (13.52mm + 11.52mm) glass,

Glass panel load = 26 kN/m^3 x (13.52mm + 11.52mm) = 0.651 kN/m^2



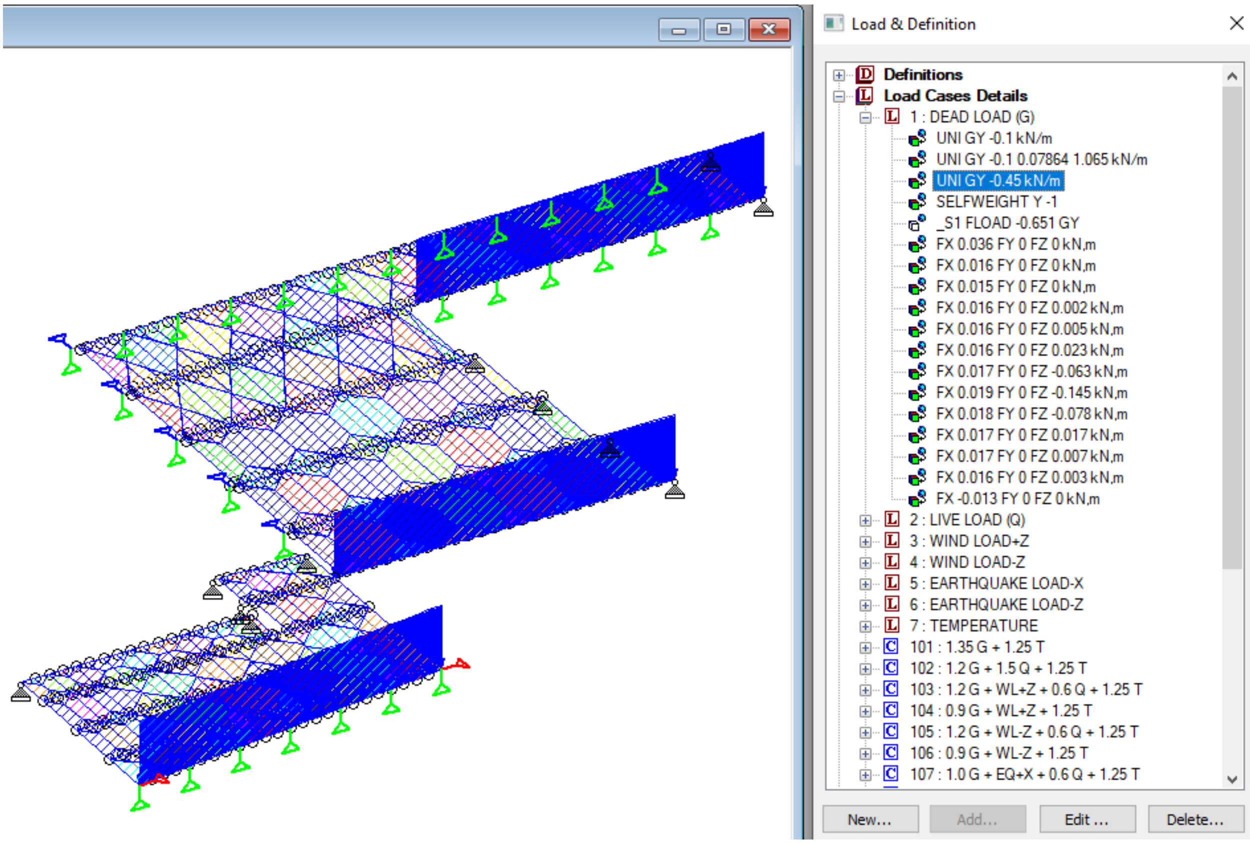
Glass Panel Load

3. Glass supported aluminum framing load = 0.1 kN/m

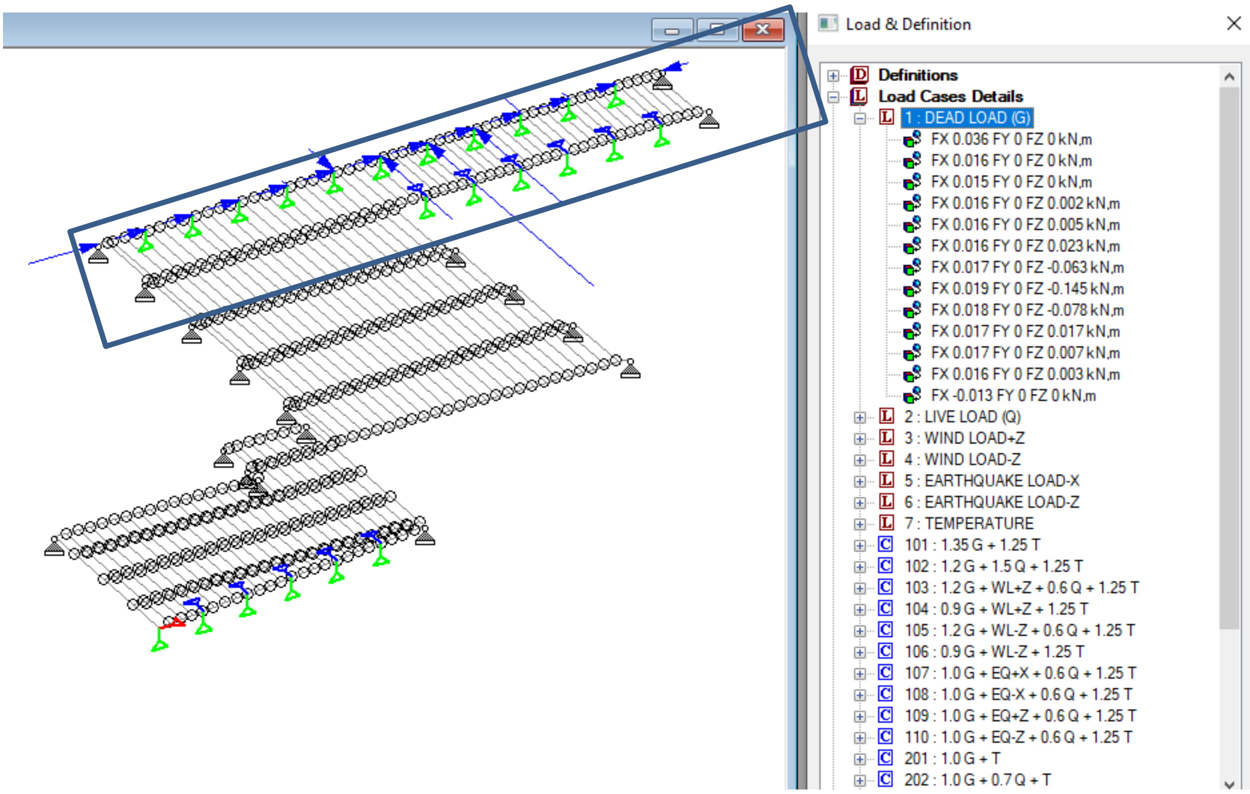


Aluminum framing load

4. Gutter load = 10 kN/m^3 x 0.15 x 0.3 = 0.45 kN/m



Gutter load



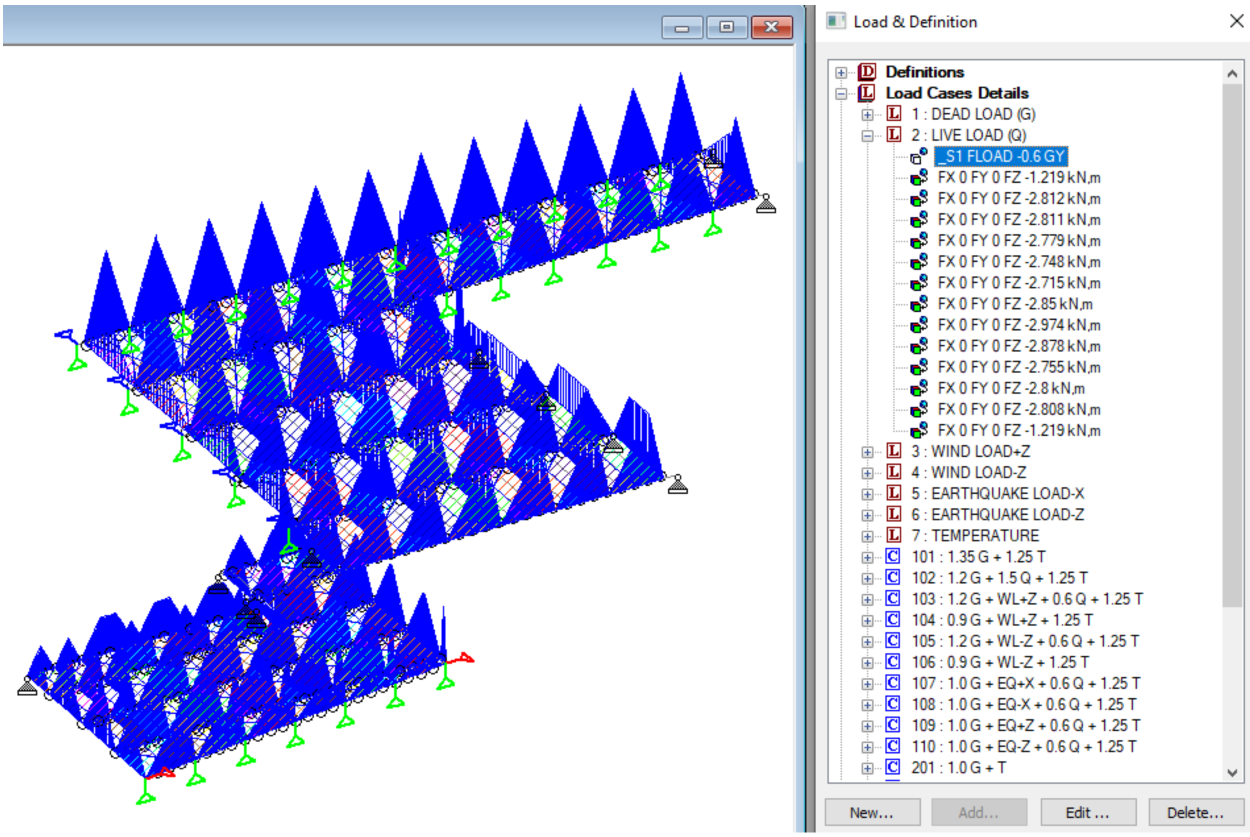
Load from glaze wall

6.2. **LL: Live Load**

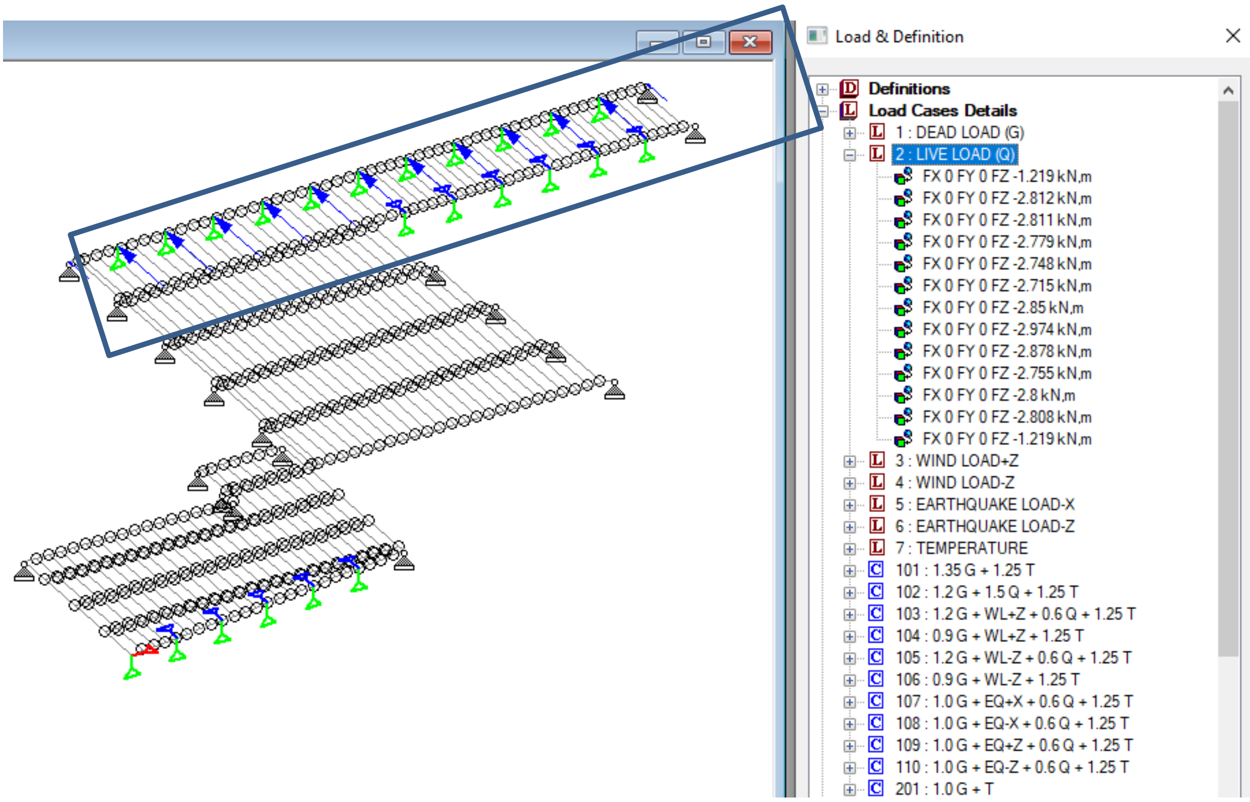
1. Live load on Facade

3.5.4 Loads from Maintenance and Building occupants	
Loads from Building Occupants	Vertical point load of 1kN applied anywhere or a uniformly distributed load of 0.6kN/m2 whichever is the most onerous to internal ledges, horizontal framing members and horizontal surfaces
Horizontal/near horizontal surfaces	Vertical uniformly distributed load of 0.6kPa, and a concentrated load of 1.1kN acting separately on a 150mm diameter contact area applied separately to any gutters, copings or flat and near flat surfaces.
Vertical/near vertical surfaces	500N applied horizontally through a 150mm diameter contact area on any vertical or near vertical surface which is accessible by building occupants or maintenance staff.

For live load application, 0.6 kN/m^2 applied as UDL.



Live load on Facade



Load from glaze wall

6.3. **WL: Wind Load**

AS/NZS_1170-2-2021
WIND LOAD CALCULATION

Wind Load as per AS 1170 Part 2 :-

Regional Wind Speed :

$V_{sit,\beta}$ = V_R X Md X (Mz,cat X Ms X Mt)

Where,

Regional Wind Speed :

V_R = Regional gust wind speed (m/s)

M_d = wind directional multipliers

$M_{z,cat}$ = terrain/height multiplier

Ms = shielding multiplier

Mt = topographic multiplier

As building is in Australia, Region of Wind is A2 (As per AS 1170.2:2021)

AS/NZS 1170.2:2021 28

Figure 3.1(A) — Wind regions — Australia

As mentioned earlier, The importance level of building considered 2.

for Importance level and annual probability of exceedence, Regional wind speed considered.

Table 3.1(A) — Regional wind speeds — Australia

Regional wind speed (m/s)	Region			
	Non-cyclonic		Cyclonic	
	A (0 to 5)	B1, B2	C (maximum)	D (maximum)
V_1	30	26	23	23
V_5	32	28	33	35
V_{10}	34	33	39	43
V_{20}	37	38	45	51
V_{25}	37	39	47	53
V_{50}	39	44	52	60
V_{100}	41	48	56	66
V_{200}	43	52	61	72
V_{250}	43	53	62	74
V_{500}	45	57	66	80
V_{1000}	46	60	70	85
V_{2000}	48	63	73	90
V_{2500}	48	64	74	91
V_{5000}	50	67	78	95
V_{10000}	51	69	81	99
V_R ($R \geq 5$ years)	$67-41R^{-0.1}$	$106-92R^{-0.1}$	$122-104R^{-0.1}$	$156-142R^{-0.1}$

NOTE 1 The peak gust has an equivalent moving average time of approximately 0.2 s (Holmes and Ginger, 2012).

NOTE 2 Values for V_1 have not been calculated by the formula for V_R in the Australian regions.

NOTE 3 For ultimate or serviceability limit states, refer to the National Construction Code (Australia) or AS/NZS 1170.0 for information on values of importance level and annual probability of exceedance appropriate for the design of structures. For buildings in townships in cyclonic regions, users should consider overall risk to a community when selecting importance levels.

NOTE 4 For Regions C and D, only the maximum values for the region are tabulated. Lower values of V_R may apply in those regions, depending on the distance of the site from the smooth coastline.

VR considered with, Importance level as 2 and Design workign life as 50 year.

V500 =	45	m/s	Table 3.1 AS1170 Part 2 (For Ultimat Limit State)
V25 =	37	m/s	Table 3.1 AS1170 Part 2 (For serviceability Limit State)
Md =	1	from Clause 3.3.2 AS1170 Part 2 by considering any direction.	

Table 3.2(A) — Wind direction multiplier (M_d) — Australia

Cardinal directions	Region A0	Region A1	Region A2	Region A3	Region A4	Region A5	Region B1	Regions B2, C, D
N	0.90	0.90	0.85	0.90	0.85	0.95	0.75	0.90
NE	0.85	0.85	0.75	0.75	0.75	0.80	0.75	0.90
E	0.85	0.85	0.85	0.75	0.75	0.80	0.85	0.90
SE	0.90	0.80	0.95	0.90	0.80	0.80	0.90	0.90
S	0.90	0.80	0.95	0.90	0.80	0.80	0.95	0.90
SW	0.95	0.95	0.95	0.95	0.90	0.95	0.95	0.90
W	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.90
NW	0.95	0.95	0.95	0.95	1.00	0.95	0.90	0.90

NOTE In Region A0 non-synoptic winds are dominant. In Regions A1 and A4, extra-tropical synoptic winds are dominant. Extreme winds in Regions A2, A3, A5 and B1 are caused by a mixture of synoptic (extra-tropical large-scale pressure systems, or tropical cyclones in the case of B1) and non-synoptic (thunderstorm) events. In Regions B2, C, and D, extreme winds from tropical cyclones are dominant.

Table 3.2 NZS1170 Part 2 (WIND DIRECTION MULTIPLIER Md) (For region A2)

wind direction in X+, Wind direction multiplier for NE =	0.75
wind direction in X-, Wind direction multiplier for SW =	0.95
wind direction in Y+, Wind direction multiplier for NW =	0.95
wind direction in Y-, Wind direction multiplier for SE =	0.95

wind direction in N, Wind direction multiplier =	0.85
wind direction in S, Wind direction multiplier =	0.95
wind direction in E, Wind direction multiplier =	0.85
wind direction in W, Wind direction multiplier =	1

Wind applied at 45° angle,	Vsit,β =	45°
	Cos45° =	0.71
	Sin45° =	0.71

Wind applied at 45° angle for NE = NE X Cos45°/Sin45°
= 0.5325

Wind applied at 45° angle for SW = SW X Cos45°/Sin45°
= 0.6745

Wind applied at 45° angle for NW = NW X Cos45°/Sin45°
= 0.6745

Wind applied at 45° angle for SE = SE X Cos45°/Sin45°
= 0.6745

Hence, Above all direction multiplier take a critical at E and W direction.

Wind direction multiplier Md considered with worst case with considering maximum cardinal direction within a sector 45 degree in both side.

Terrain Category (AS1170 Part 2)

Based upon the site condition, Terrain category considered = 3

Table 4.1 — Terrain/height multipliers for gust wind speeds in fully developed terrains — All regions except A0

Height (z) (m)	Terrain/height multiplier ($M_{z,cat}$)				
	Terrain Category 1	Terrain Category 2	Terrain Category 2.5	Terrain Category 3	Terrain Category 4
≤ 3	0.97	0.91	0.87	0.83	0.75
5	1.01	0.91	0.87	0.83	0.75
10	1.08	1.00	0.92	0.83	0.75
15	1.12	1.05	0.97	0.89	0.75
20	1.14	1.08	1.01	0.94	0.75
30	1.18	1.12	1.06	1.00	0.80
40	1.21	1.16	1.10	1.04	0.85
50	1.23	1.18	1.13	1.07	0.90
75	1.27	1.22	1.17	1.12	0.98
100	1.31	1.24	1.20	1.16	1.03
150	1.36	1.27	1.24	1.21	1.11
200	1.39	1.29	1.27	1.24	1.16

NOTE 1 In Region A0, use $M_{z,cat\ 2}$ for all $z \leq 100$ m in all terrains. For 100 m < $z \leq 200$ m, take $M_{z,cat}$ as 1.24 in all terrains.

NOTE 2 For all other regions, for intermediate terrains use linear interpolation.

NOTE 3 For intermediate values of height z, use linear interpolation.

By linear interpolation,

Height of building,h = 7.62 m

Determination of terrain/height multiplier (Mz,cat) = 0.83

Table 4.1 AS 1170 Part 2

Ms = Shielding multiplier = 1 AS1170 Part 2

Mt= Topographic multiplier = As per AS.1170.2:2021, Mlee can be taken as 1.0

Mt=Mh

Mt= Topographic multiplier = 1

Site Wind Speed

Vsit,β = Vr X Md X (Mz,cal X Ms X Mt)

V500 = 45 x 1 x 0.83 x 1 x 1

= 37.35

V25 = 37 x 1 x 0.83 x 1 x 1

= 30.71

Design wind pressure :

$$p = (0.5\ \rho_{air})\ [V_{des,\theta}]^2\ C_{fig}\ C_{dyn}$$

C_{dyn} = 1

$C_{fig,e}$ = $C_{p,e}\ K_a\ K_{c,e}\ K_\ell\ K_p$, for external pressures

$C_{fig,i}$ = $C_{p,i}\ K_{c,i}$, for internal pressures

ρ_{air} = density of air, which shall be taken as 1.2 kg/m^3

Cpe = external pressure coefficient		Table 5.2 (A)/(B)/(C) AS-1170 Part 2
Cpi = Internal pressure coefficient		Table 5.1 (A) AS-1170 Part 2
Ka =	0.9	Table 5.4 AS-1170 Part 2
Kce=	0.9	Table 5.5 AS-1170 Part 2
Kci=	1	Table 5.5 AS-1170 Part 2
Kl=	1.5	Table 5.6 AS-1170 Part 2
Kp =	0.9	Table 5.8 AS-1170 Part 2

Puls = 0.6 X 37.35^2 X (Cfig.e+Cfig,i)

Psls = 0.6 X 30.71^2 X (Cfig.e+Cfig,i)

External wind coeficients

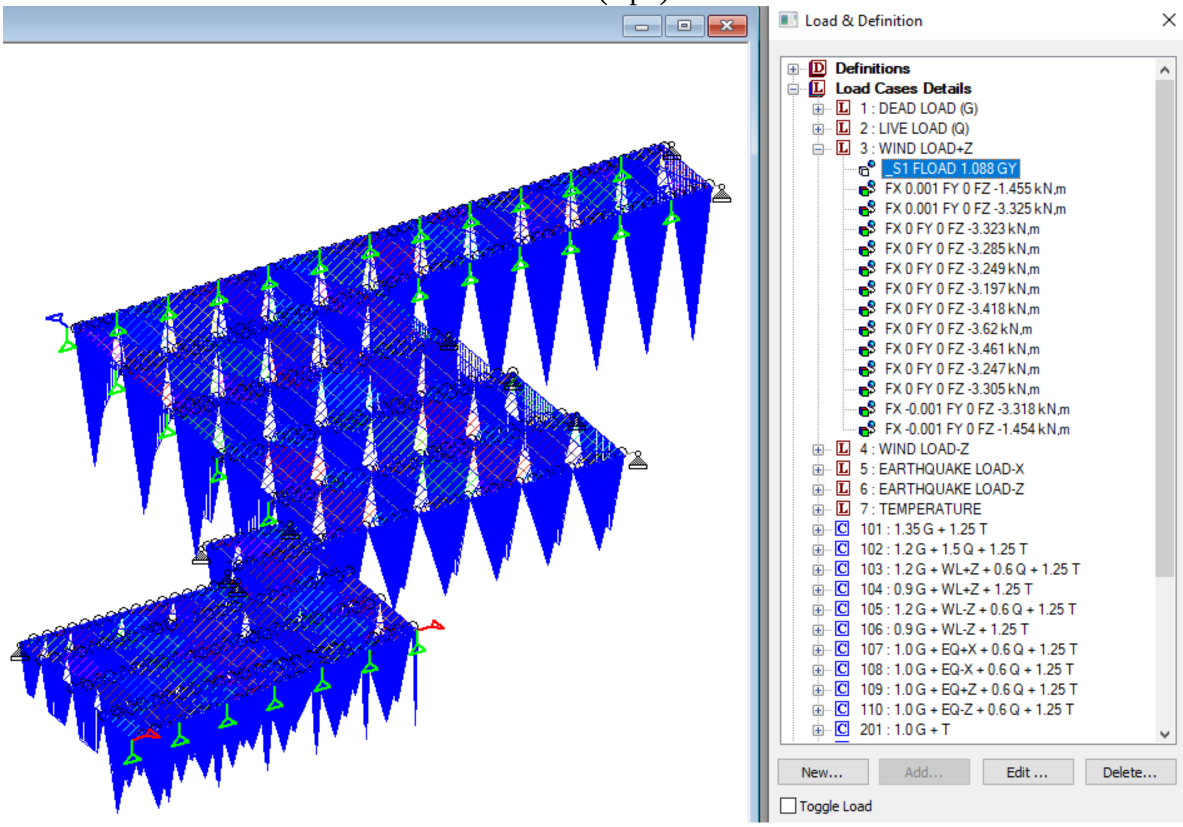
Windward	C_{pe} =	0.8	
Leeward	C_{pe} =	-0.5	
Side wall	C_{pe} =	-0.65	
Roof	C_{pe} =	-1.3	-0.6

Windward	$C_{fig\ e}$ =	0.87	
Leeward	$C_{fig\ e}$ =	-0.55	
Side wall	$C_{fig\ e}$ =	-0.71	
Roof	$C_{fig\ e}$ =	-1.42	-0.66

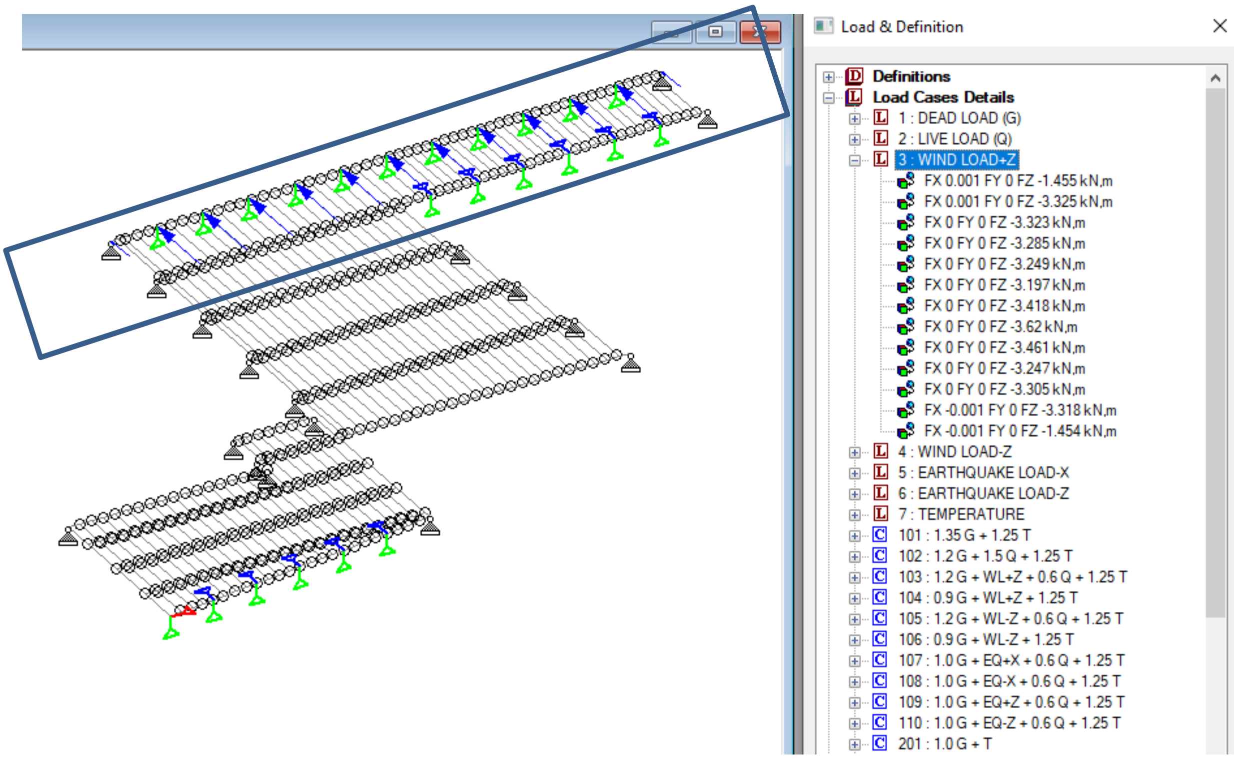
Wind pressure = =0.6 x 37.35^2/1000 = 0.837 kN/m^2

6.3.1. Wind Load on glass in +Z direction

Wind Load on Glass = 0.837 kN/m^2 x -1.3(Cpe) = -1.088 kN/m^2



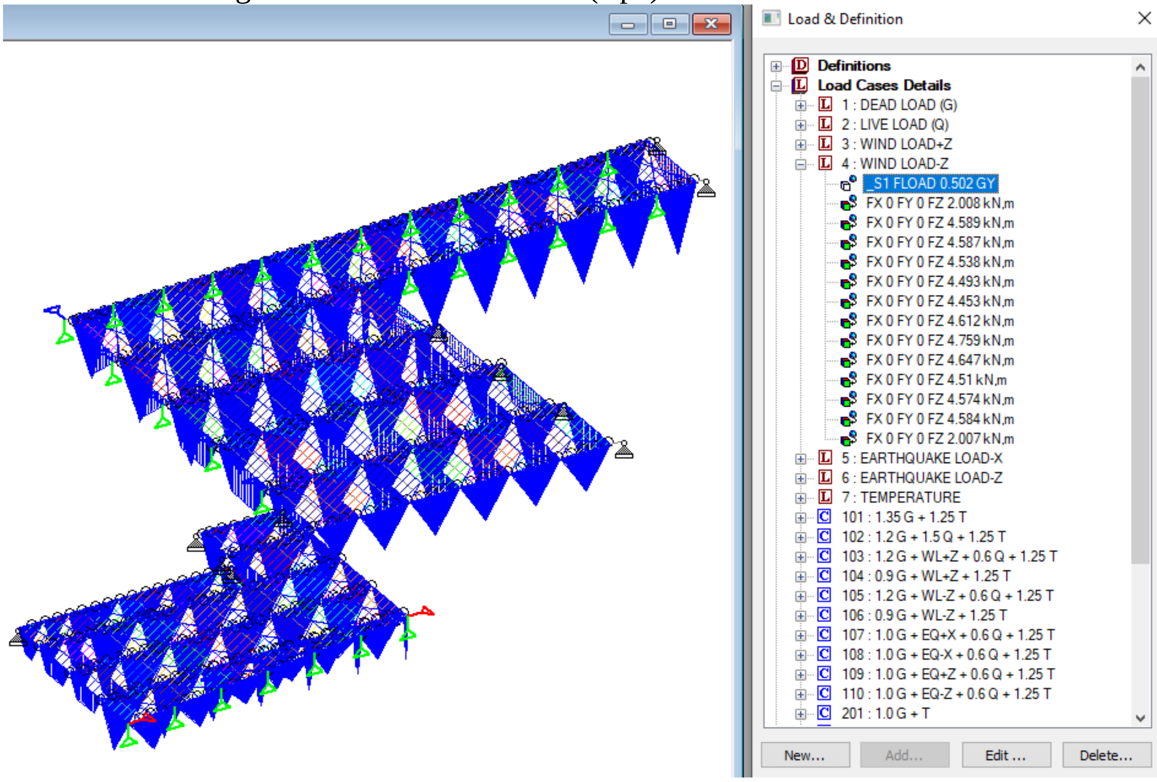
<u>Wind load on Façade</u>



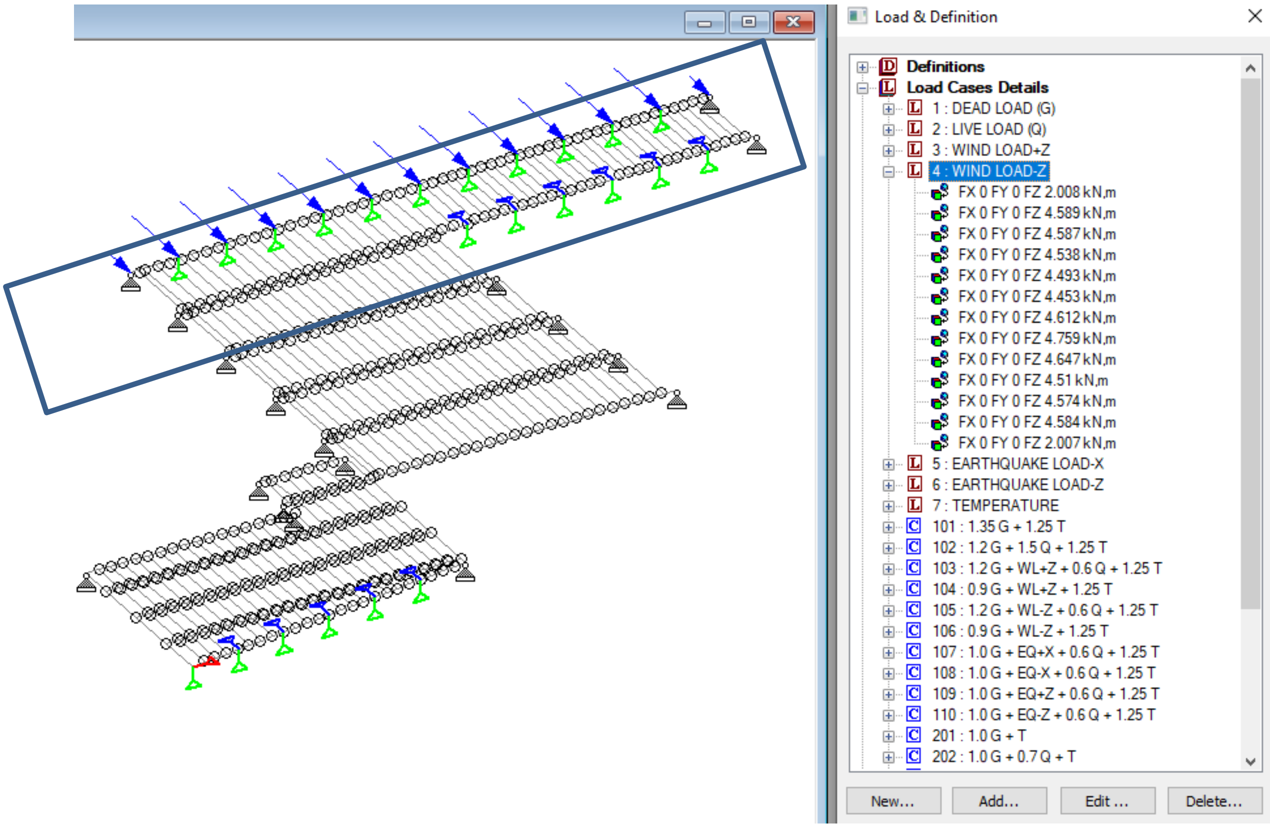
Load from glaze wall

6.3.2. Wind Load on Glass in –Z direction

Wind load on glass = 0.837 kN/m^2 x -0.6 (Cpe) = -0.502 kN/m^2



Wind load on Façade



Load from glaze wall

6.4. **EQ: Earthquake Load**

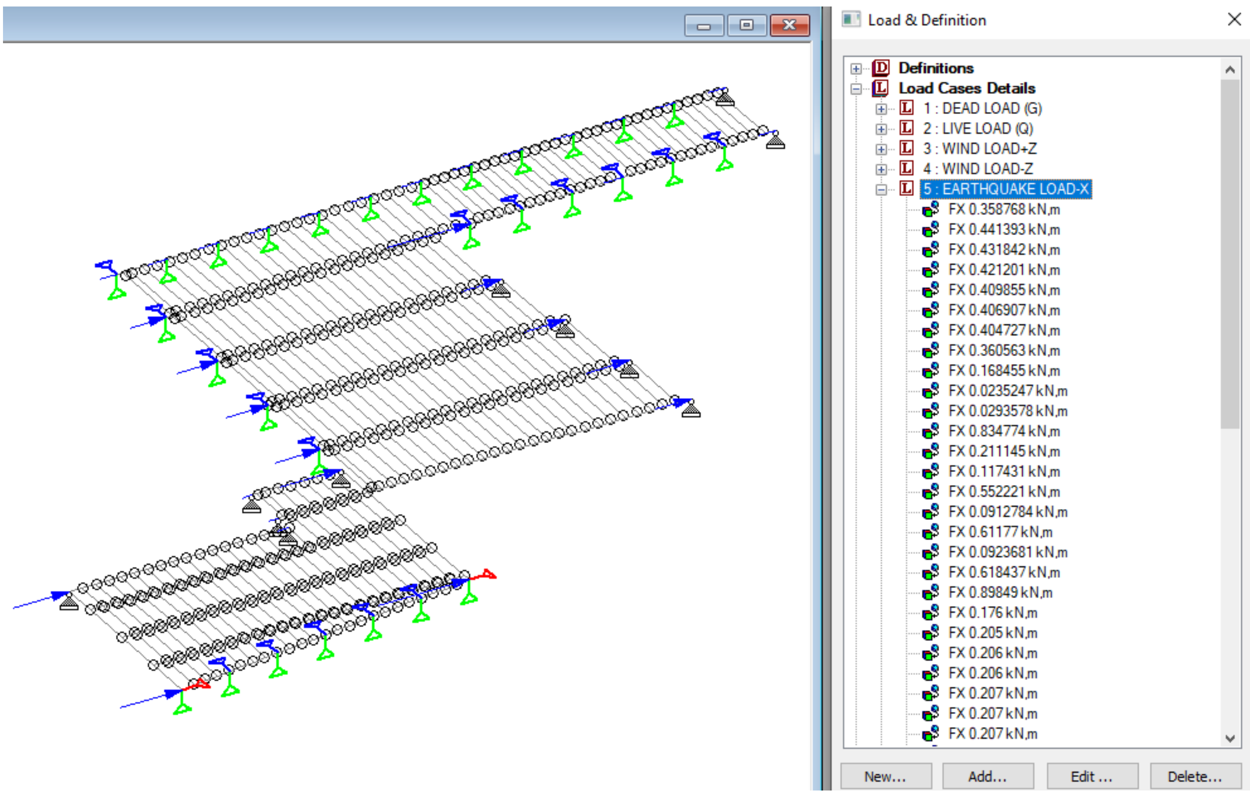
EQUIVALENT STATIC METHOD

The horizontal equivalent static shear force (V) acting at the base of the structure (base shear) in the direction being considered shall be calculated from the following equations:

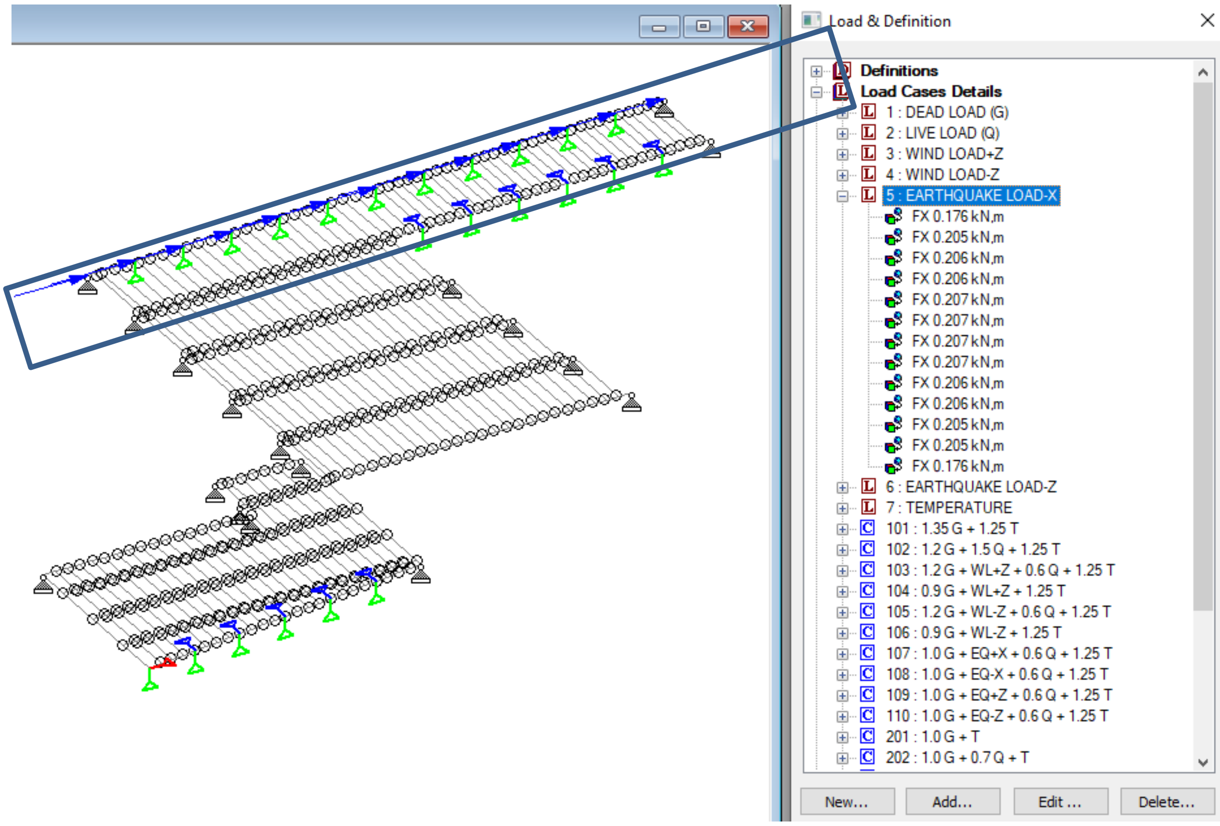
Earthquake base shear
$$
\begin{aligned}
V &= C_d(T_1)W_t \\
&= [C(T_1)S_p/\mu]W_t \\
&= [k_p Z C_h(T_1)S_p/\mu]W_t
\end{aligned}
$$

Factors	Abbrivi.	Value	Unit	Remarks
Probability Factor	K_p =	1	-	As per Annual probabolity P = 1/500
Hazard factor	Z =	0.08	-	As per Sydney location
Spectral time period for T_1	C_h(T1) =	2.08	sec	As per class C soil
Structural performance factor	S_p =	0.77	-	For Steel OMRF
Structural ductility factor	μ =	2	-	For Steel OMRF
Seismic weight of structure	W_t =	58	kN	Considering Dead load + 0.3x Imposed load
Horizontal design action coefficient	C_d(T1) =	0.0641	-	-
Horizontal equivelent static base shear	V =	3.7	kN	-

6.4.1. Earthquake Load in X direction

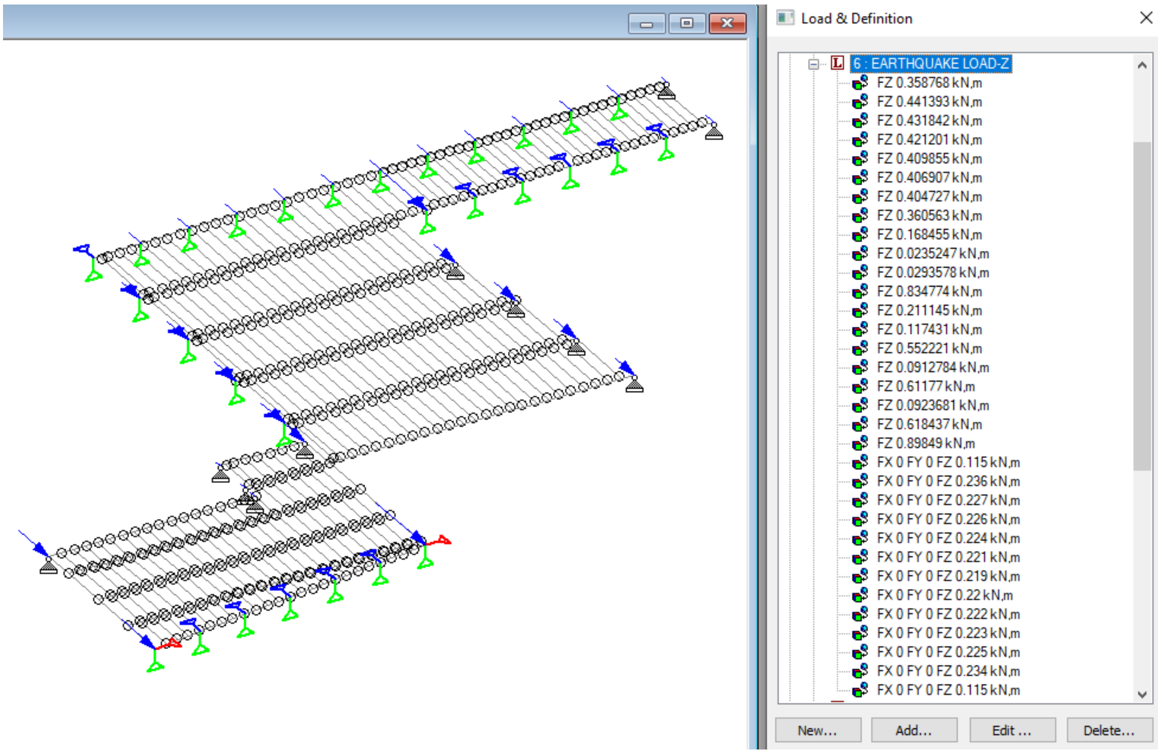


Earthquake Load in X direction

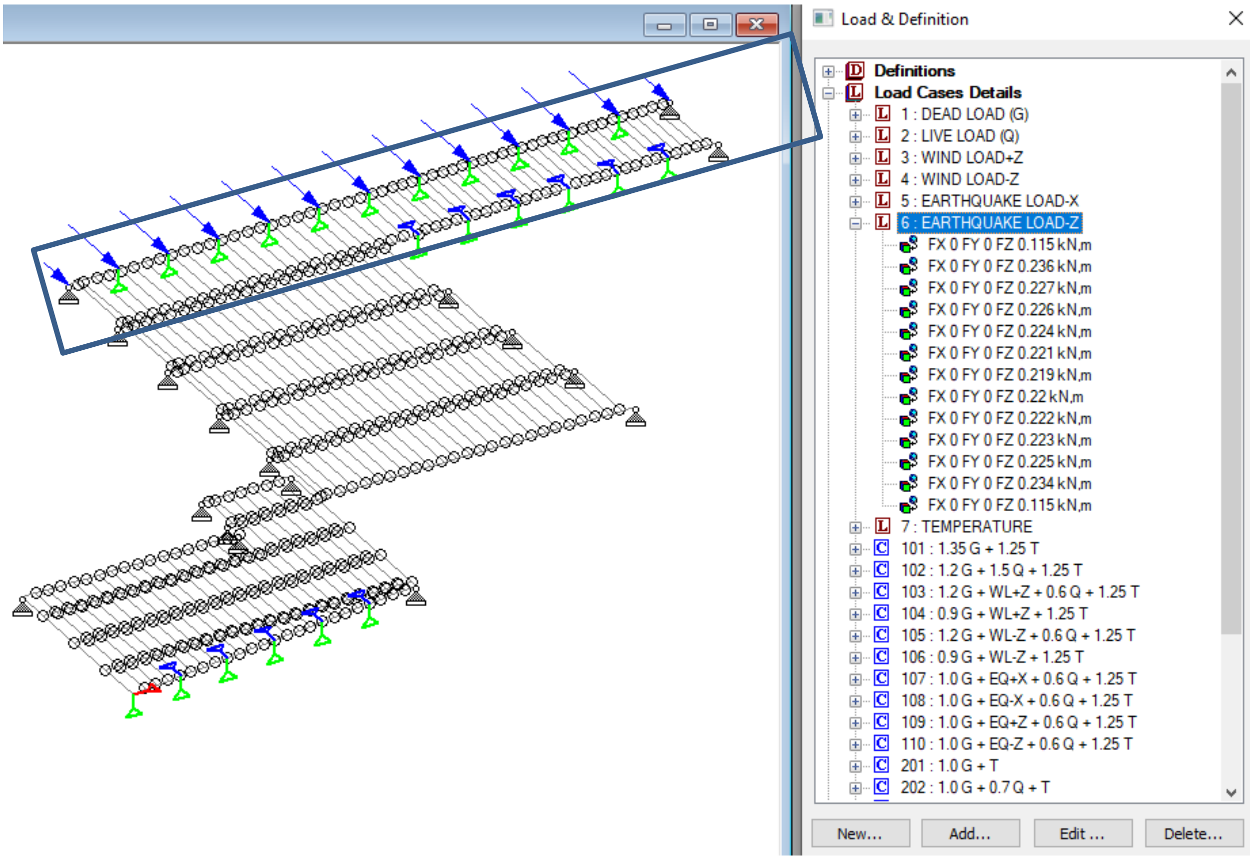


Load from glaze wall

6.4.2. Earthquake Load in Z direction

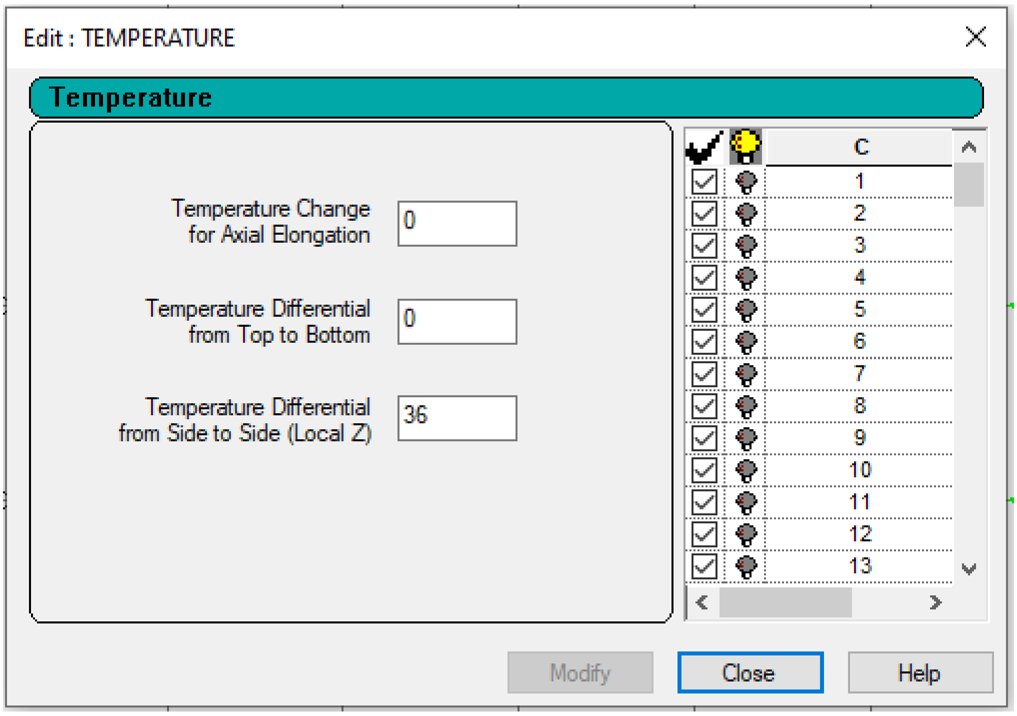


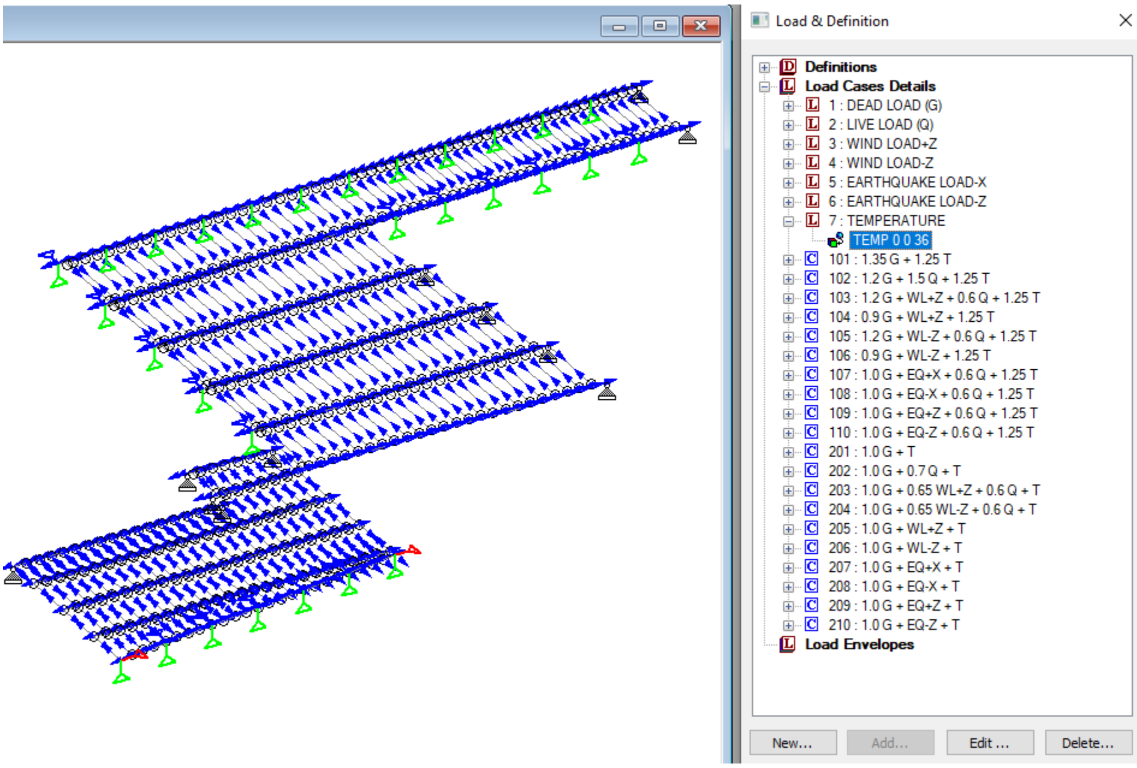
Earthquake Load in Z direction



Load from glaze wall

6.5. **TL: Temperature Load**





Temperature Load

7. LOAD COMBINATIONS

Load combinations as per AS/NZS 1170.0.2002 Structural Design Actions

<u>Design load combinations</u>

101. 1.35 G + 1.25 T
102. 1.2 G + 1.5 Q + 1.25 T
103. 1.2 G + WL-Z (PRESSURE) + 0.6Q + 1.25 T
104. 0.9 G + WL-Z (PRESSURE) + 1.25 T
105. 1.2 G + WL-Z (SUCTION) + 0.6Q + 1.25 T
106. 0.9 G + WL-Z (SUCTION) + 1.25 T
107. 1.0 G + EQX + 0.6 Q + 1.25 T
108. 1.0 G - EQX + 0.6 Q + 1.25 T
109. 1.0 G + EQZ + 0.6 Q + 1.25 T
110. 1.0 G - EQZ + 0.6 Q + 1.25 T

<u>Service load combinations</u>

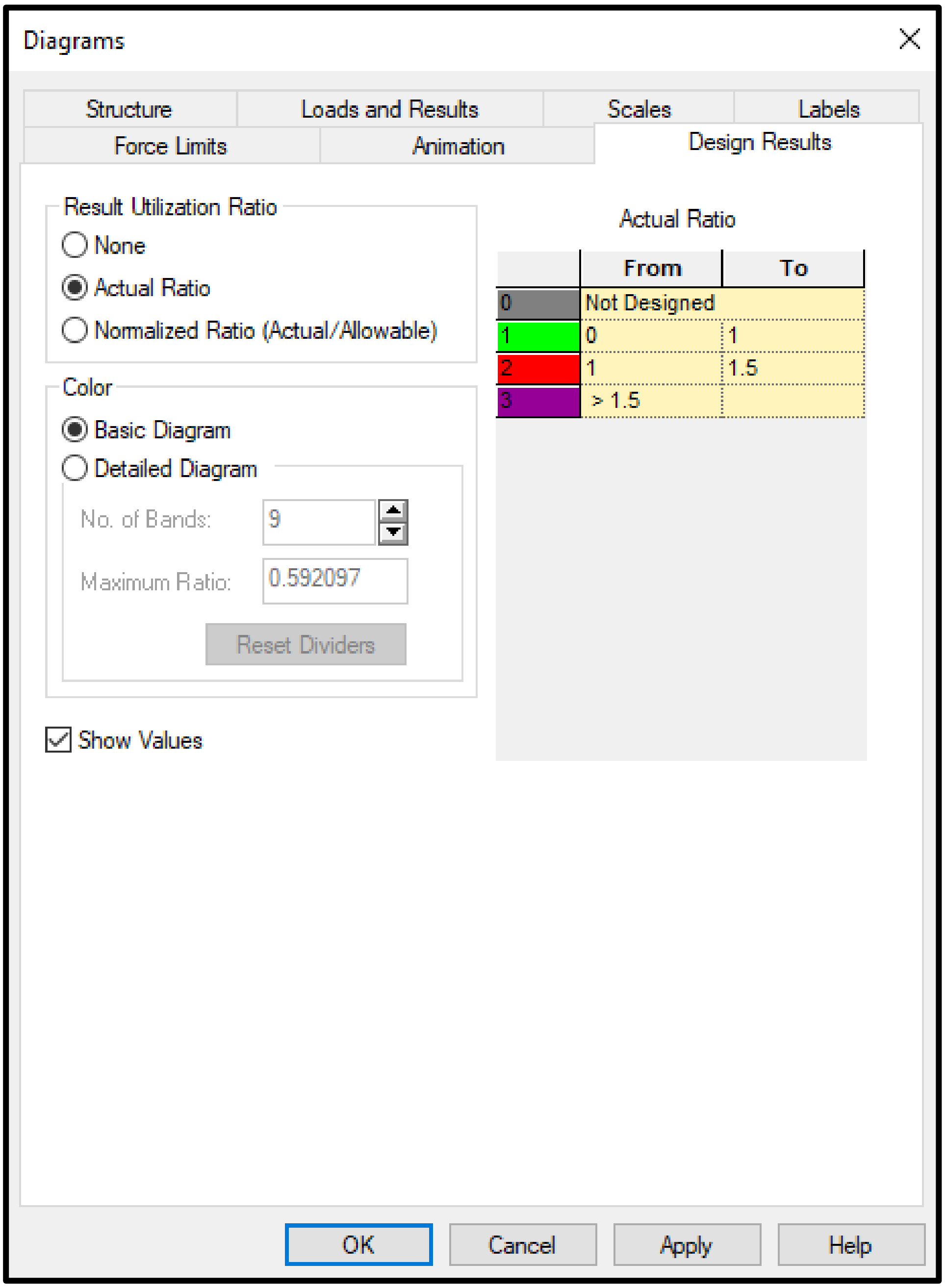
201. 1.0 G + 1.0 T
202. 1.0 G + 0.7 Q + 1.0 T
203. 1.0 G + 0.65WL-Z (PRESSURE) + 0.6 Q + 1.0 T
204. 1.0 G + 0.65WL-Z (SUCTION) + 0.6 Q + 1.0 T
205. 1.0 G + WL-Z (PRESSURE) + 1.0 T
206. 1.0 G + WL-Z (SUCTION) + 1.0 T

207 1.0 G + EQX + 1.0 T
208 1.0 G – EQX + 1.0 T
209 1.0 G + EQZ + 1.0 T
210 1.0 G – EQZ + 1.0 T

8. ANALYSIS & DESIGN RESULTS

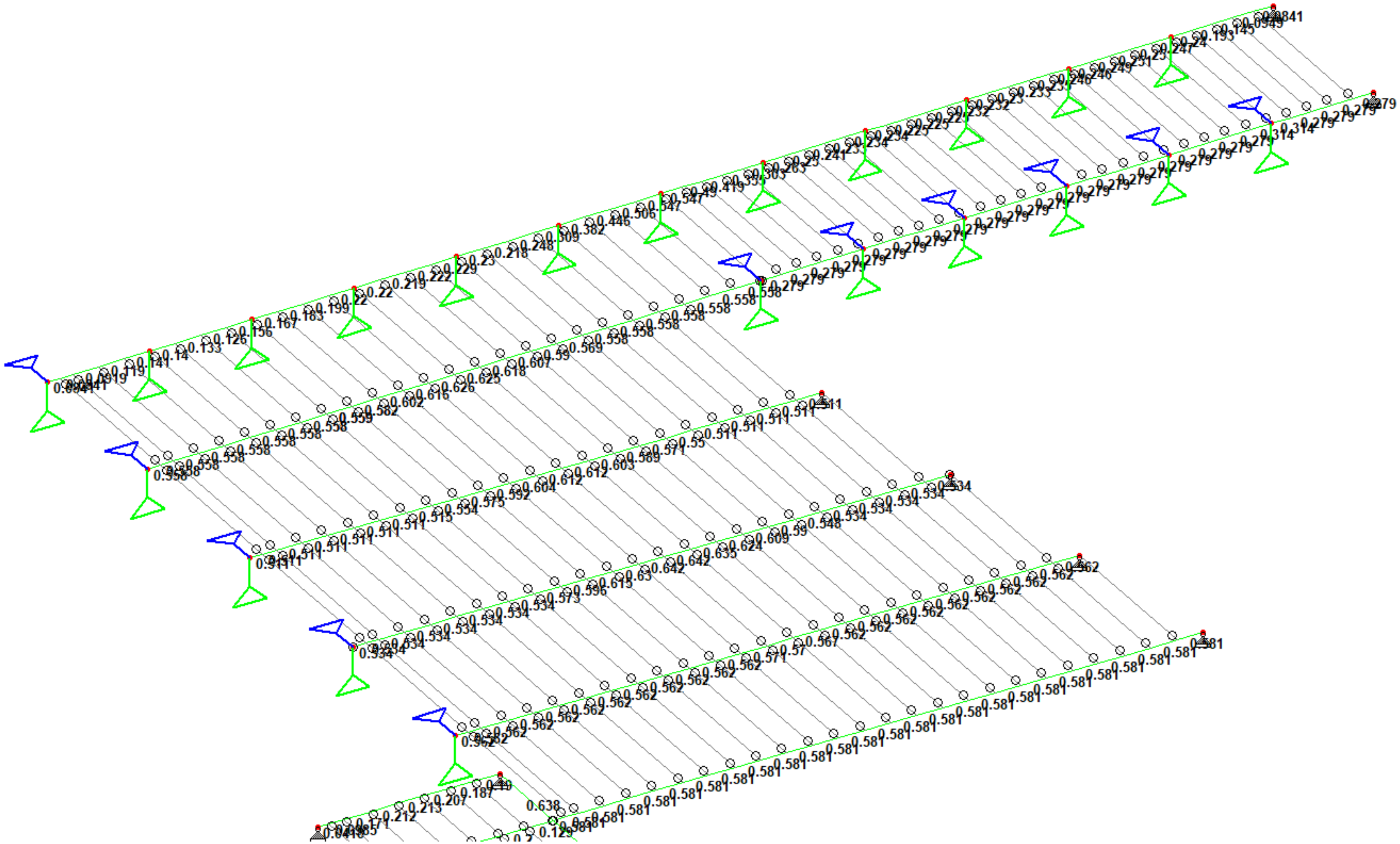
8.1. UTILITY CHECK

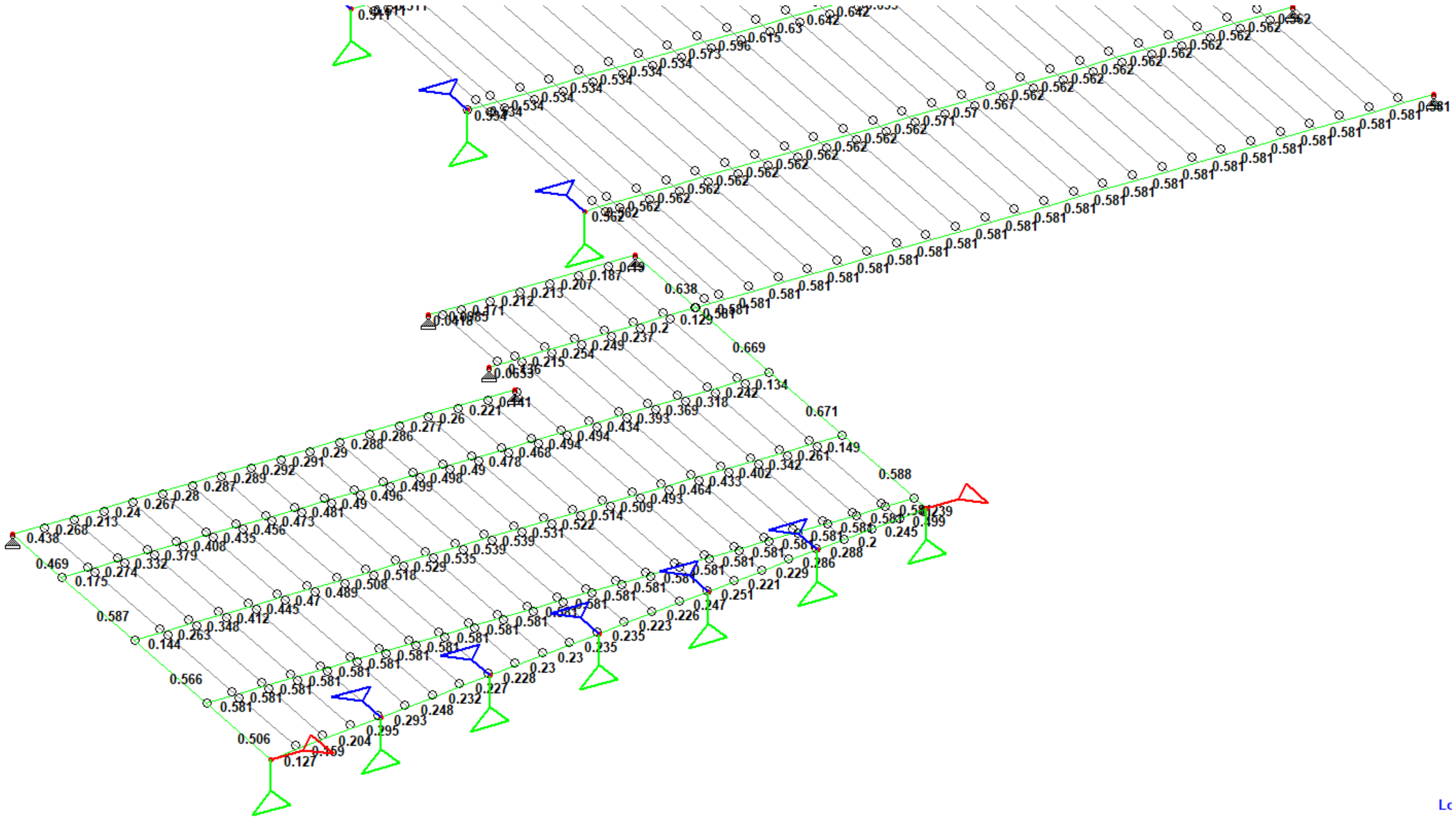
Below images shows value ranges for utility ratios & colored diagrams to understand utilization of structural members.



Utility ratio ranges for detailed diagram

Below image shows that failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors, if any. It can be seen from below image that all members are green. Hence, all members have passed in design.

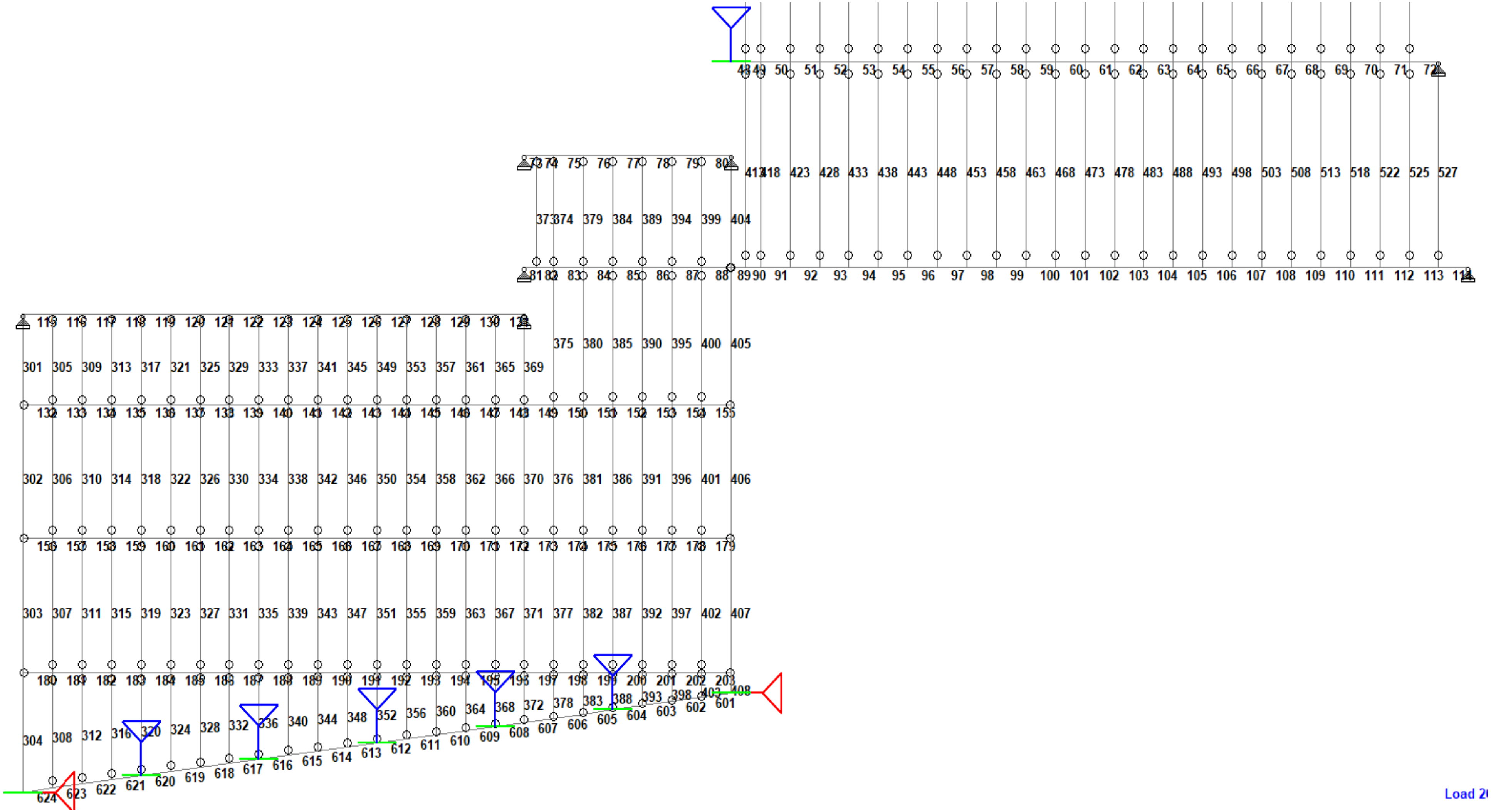




Detailed colored diagram of utility ratios

8.2. **BEAM NUMBER**

Beam Numbers

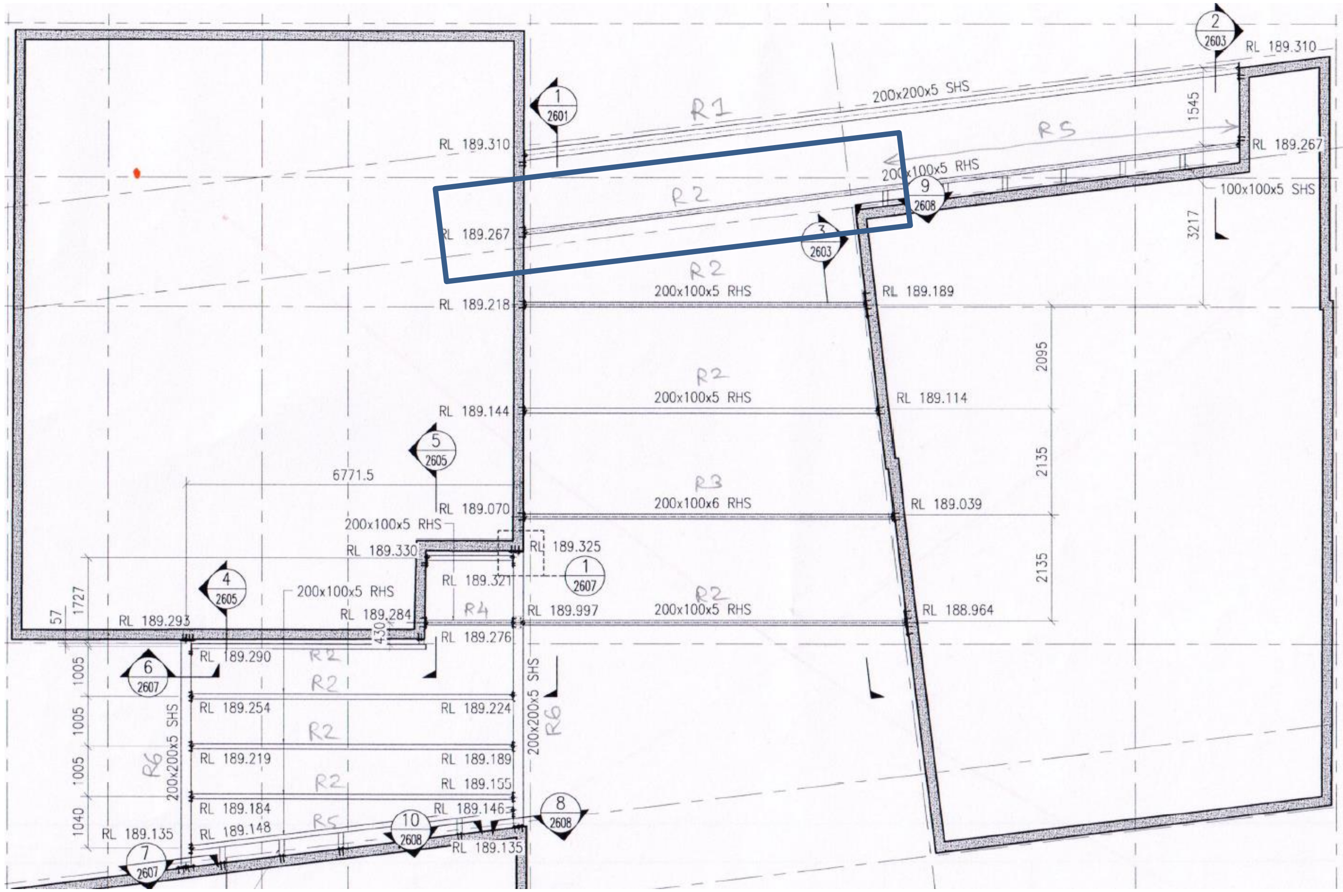


Beam Numbers

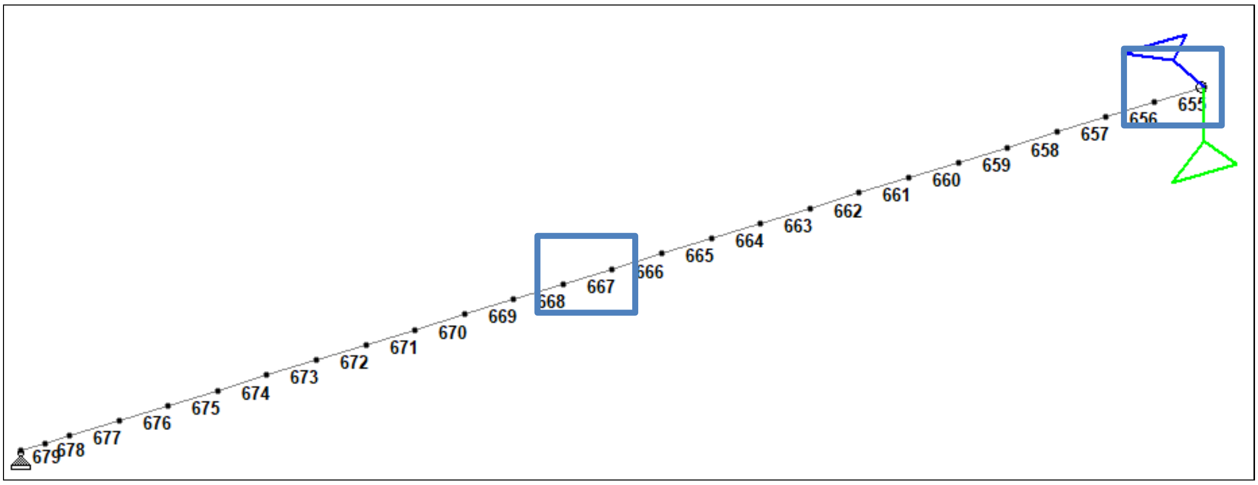
8.3. **MEMBER DESIGN**

For three different type of member property governing member design calculation is given as below:

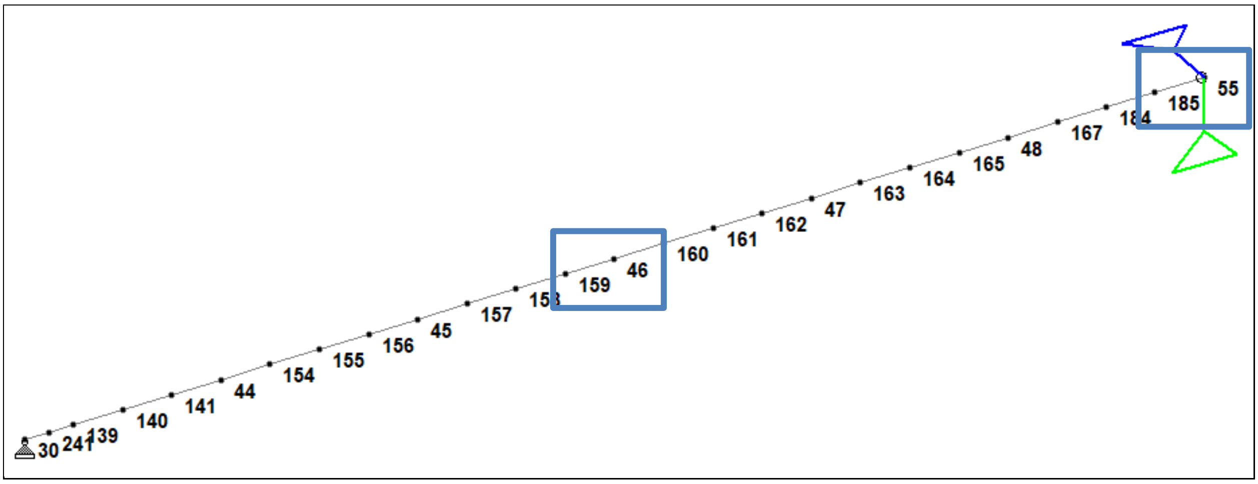
8.3.1. R2 - RHS 200x100x5:



Geometry data of Rafter R2:

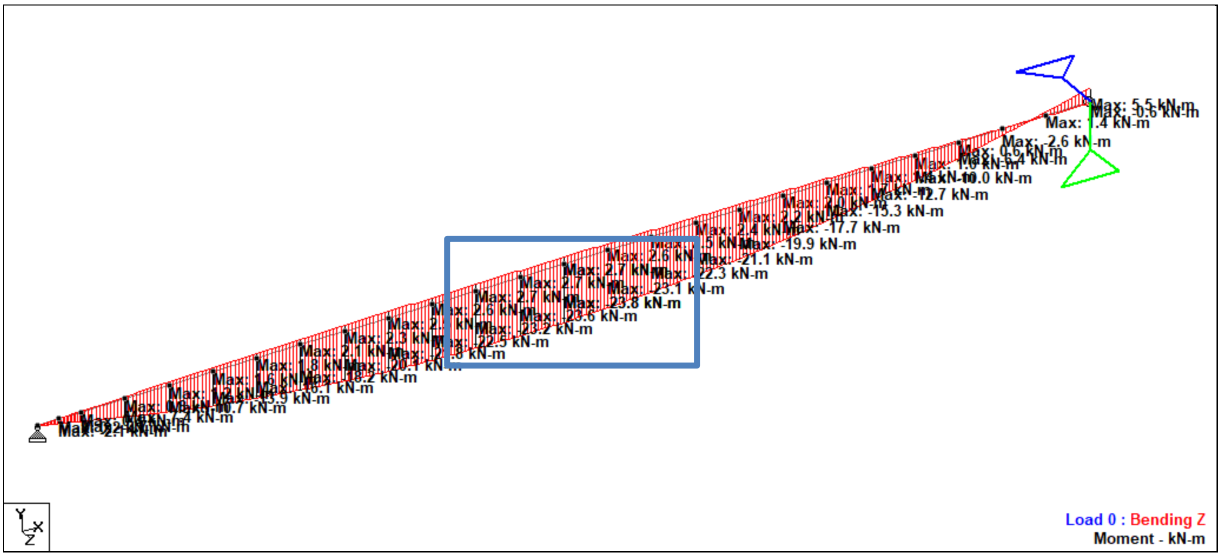


Beam number

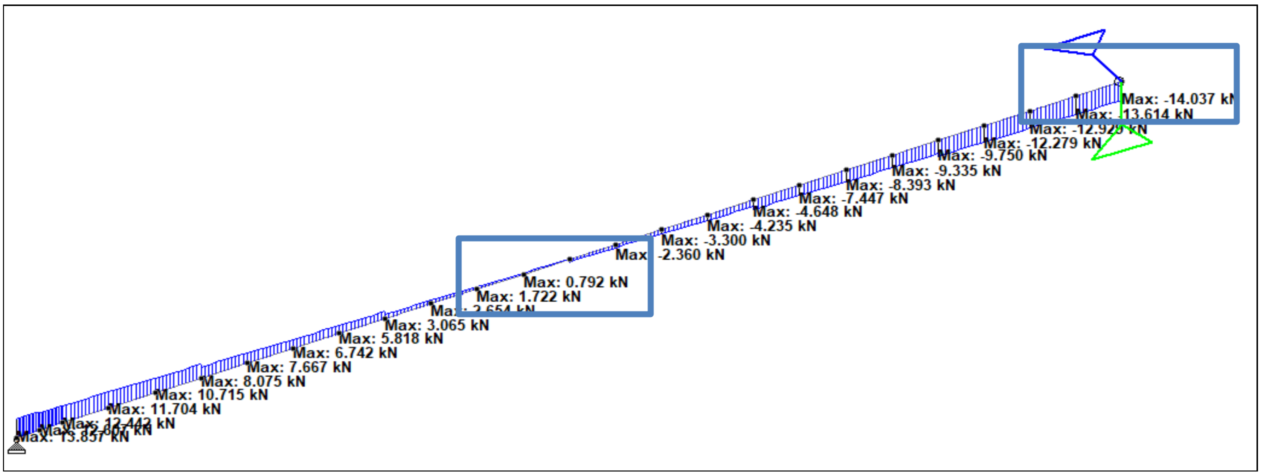


Node number

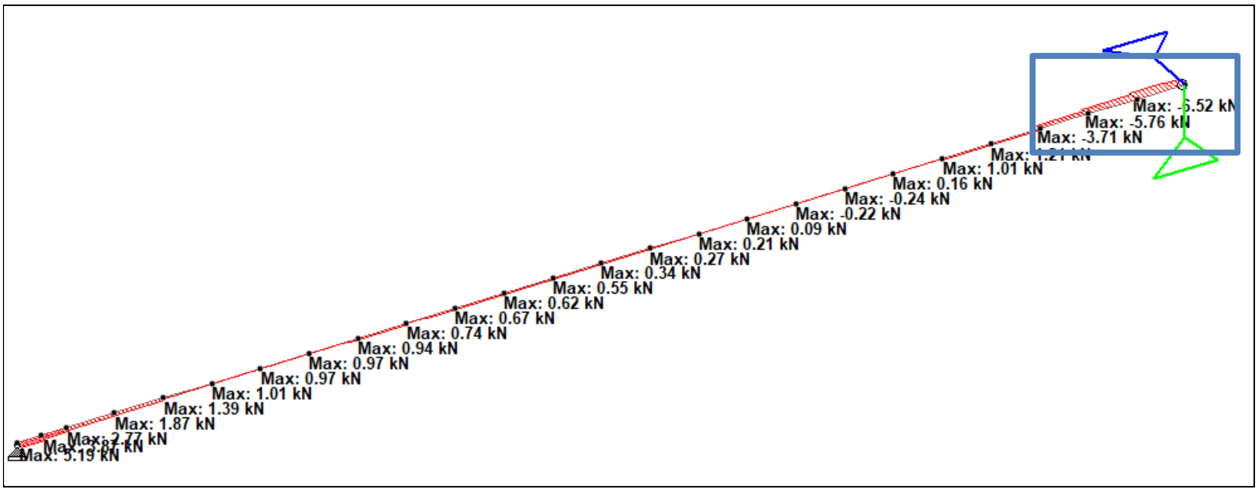
- Analysis results for R2 rafter



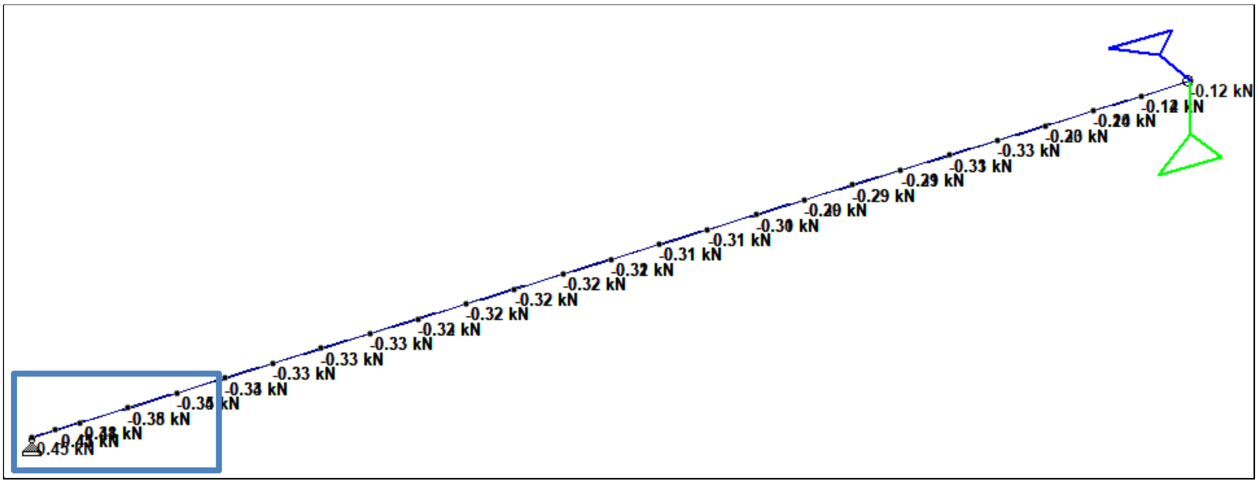
Governing Bending Moment Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fy) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fz) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Axial Force(Fx) Diagram Diagram – 102-1.2G+1.5Q+1.25T

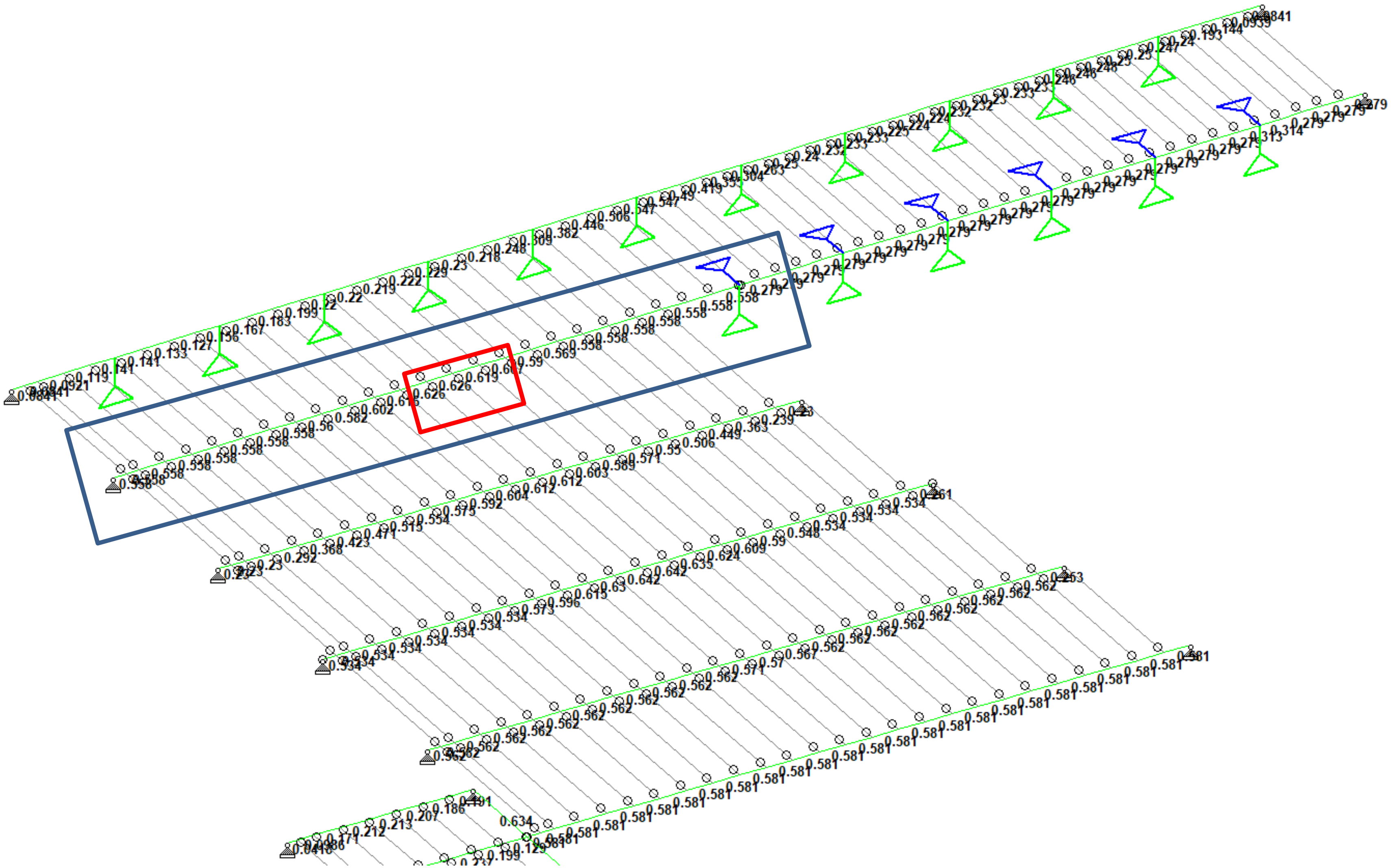
Member forces table for Rafter R2:

Facade Roof_10-04-2024 - R9_Roller support & member added - Beam End Forces: R2

All | Summary | Envelope

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kN-m	My kN-m	Mz kN-m
Max Fx	656	108 1.0 G + EQ-X + 0.6 Q + 1.25 T	184	**0.345**	-8.320	-6.542	-0.0	5.1	-1.7
Min Fx	679	107 1.0 G + EQ+X + 0.6 Q + 1.25 T	30	**-0.908**	8.941	4.701	0.0	-0.0	0.0
Max Fy	679	102 1.2 G + 1.5 Q + 1.25 T	30	-0.445	**13.857**	5.192	-0.0	-0.0	0.0
Min Fy	655	102 1.2 G + 1.5 Q + 1.25 T	55	-0.119	**-14.037**	-6.523	-0.0	1.2	5.5
Max Fz	679	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	30	-0.448	2.975	**5.346**	-0.0	-0.0	0.0
Min Fz	657	106 0.9 G + WL-Z + 1.25 T	167	-0.067	-2.144	**-9.234**	-0.0	8.3	-1.2
Max Mx	665	104 0.9 G + WL+Z + 1.25 T	160	-0.314	0.269	0.370	**0.0**	5.0	2.6
Min Mx	665	102 1.2 G + 1.5 Q + 1.25 T	160	-0.309	-2.396	0.272	**-0.0**	5.2	-23.1
Max My	657	106 0.9 G + WL-Z + 1.25 T	167	-0.067	-2.144	-9.234	-0.0	**8.3**	-1.2
Min My	679	101 1.35 G + 1.25 T	30	-0.434	8.822	4.375	0.0	**-0.0**	0.0
Max Mz	655	102 1.2 G + 1.5 Q + 1.25 T	55	-0.119	-14.037	-6.523	-0.0	1.2	**5.5**
Min Mz	667	102 1.2 G + 1.5 Q + 1.25 T	46	-0.316	0.417	0.551	-0.0	5.1	**-23.8**

8.3.2. Design calculation of – R2 - RHS 200x100x5:



Max. Utilization ratio for beam number 667

- Beam no.: 667 - Design calculation for Maximum bending moment

```
|-----------------------------------------------------------------------------|
|                                                   Y        PROPERTIES       |
|*************                                      |        IN   CM  UNIT    |
|            *  |=============================|             ------------     |
|MEMBER 667  *  |                             |             AX=0.2836E+2     |
|            *  | ST  200X100X5.0RHS          |        --Z  AY=0.1900E+2     |
|DESIGN CODE *  |                             |             AZ=0.1000E+2     |
|AS4100 1998 *  ===============================             PY=0.1121E+3     |
|            *                                              PZ=0.1814E+3     |
|            *  |<---LENGTH (ME=     0.30 --->|             RY=0.4186E+1     |
|*************                                              RZ=0.7173E+1     |
|                                                           Iw=0.0000E+0     |
|                 23.8( KN-METR)                                             |
|PARAMETER       |                               ?102FORCE/MOMENT           |
|IN  NEWTON MM   |                       ?102?102    IN  KN METRE           |
|--------------- +               ?102?102            ------------           |
| KL/R-Y=    7.2 |                                   PNC=0.4042E+3          |
| KL/R-Z=  100.4 +       ?102?102                    PNT=0.8933E+3          |
| UNL    =  300.1 |                                  pn =-.3163E+0          |
| MAIN   =  180.0 +     ?102                         MNZ=0.5713E+2          |
| PHI    =   0.90 |   ?102                           mnz=-.2376E+2          |
| FULT   =  430.0 + ?102                             MNY=0.2437E+2          |
| FYLD   =  350.0 |?102                              mny=0.5111E+1          |
| NSF    =   1.00 +---+---+---+---+---+---+---+---+---+---|  VZ =0.1890E+3   |
| SKT    =   1.00  23.6                              vz =0.5506E+0          |
| SKL    =   1.00           ABSOLUTE MZ ENVELOPE     VY =0.3591E+3          |
| SKR    =   1.00             (WITH LOAD NO.)        vy =0.4170E+0          |
|                                                                           |
| Section Type:  Compact - about Z axis;     Slender - about Y axis         |
|                                                                           |
|                    Parameters used to calculate RATIO                     |
|                    ----------------------------------                     |
| Ns=0.8453E+3  Msz=0.6348E+2  Msy=0.2708E+2  Mbz=0.6348E+2                 |
|               Mrz=0.6348E+2  Mry=0.2707E+2  Moz=0.6348E+2                 |
|              Nciz=0.4491E+3 Nciy=0.8453E+3  Ncz=0.4491E+3  Ncy=0.8453E+3  |
|              Vvmy=0.3990E+3 Vvmz=0.2100E+3                                |
| ALPHA,M= 0.984  ALPHA,B=-0.500  ALPHA,SZ= 1.036                           |
|                                                                           |
|                    MAX FORCE/ MOMENT SUMMARY ( KN-METR)                   |
|                    -------------------------                              |
|                AXIAL      SHEAR-Y    SHEAR-Z    MOMENT-Y    MOMENT-Z      |
|      VALUE      -0.8         0.8        0.9         5.4        23.8        |
|   LOCATION       0.3         0.0        0.0         0.3         0.3        |
|    LOADING       107         102        106         103         102        |
|***************************************************************************|
|*                        DESIGN SUMMARY ( KN-METR)                        *|
|*                        --------------                                   *|
|*       RESULT/     CRITICAL COND/     RATIO/        LOADING/             *|
|*         FX            MY             MZ           LOCATION              *|
|*        ======================================================           *|
|*         PASS        AS-8.3.4         0.626            102               *|
|*         0.32 T         5.1          -23.8             0.30              *|
|***************************************************************************|
```

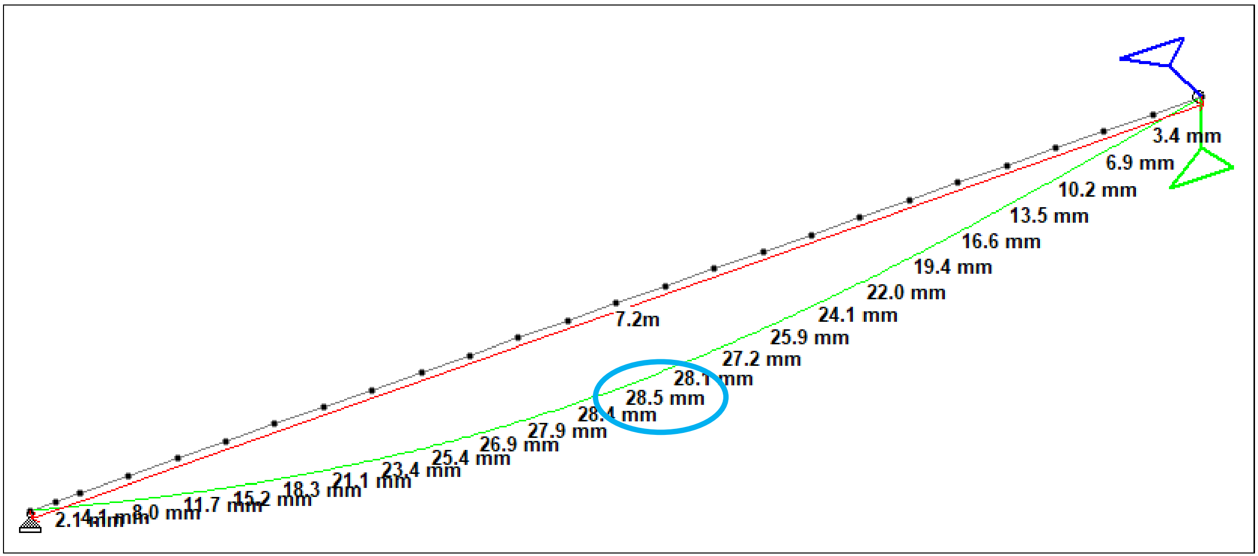
- Beam no.: 655 - Design calculation for Maximum Shear Force

```
|-------------------------------------------------------------------------|
|                                                    Y        PROPERTIES  |
|*************                                       |        IN  CM  UNIT|
|            *  |=============================|               ------------|
|MEMBER 655  *  |                             |               AX=0.2836E+2|
|            *  | ST  200X100X5.0RHS          |         --Z   AY=0.1900E+2|
|DESIGN CODE *  |                             |               AZ=0.1000E+2|
|AS4100 1998 *  ===============================               PY=0.1121E+3|
|            *                                                PZ=0.1814E+3|
|            *  |<---LENGTH (ME=     0.30 --->|               RY=0.4186E+1|
|*************                                                RZ=0.7173E+1|
|                                                             Iw=0.0000E+0|
|                  5.5( KN-METR)                                          |
|PARAMETER        |                                     ?102FORCE/MOMENT |
|IN  NEWTON MM    |                                 ?102    IN  KN METRE |
|--------------   +                         ?102?102        -------------|
| KL/R-Y=    7.1  |                     ?102                PNC=0.4042E+3|
| KL/R-Z=  100.4  +                                         PNT=0.8933E+3|
| UNL   =  298.7  |               ?102?102                  pn =0.3291E+0|
| MAIN  =  180.0  +                                         MNZ=0.5713E+2|
| PHI   =   0.90  |           ?102                          mnz=0.8877E+0|
| FULT  =  430.0  +   ?102?102                              MNY=0.2437E+2|
| FYLD  =  350.0  |?102                                     mny=0.3102E+1|
| NSF   =   1.00  +---+---+---+---+---+---+---+---+---+---|  VZ =0.1890E+3|
| SKT   =   1.00    1.2                                     vz =-.5988E+1|
| SKL   =   1.00              ABSOLUTE MZ ENVELOPE          VY =0.3591E+3|
| SKR   =   1.00                 (WITH LOAD NO.)            vy =-.8800E+1|
|                                                                         |
| Section Type:  Compact - about Z axis;     Slender - about Y axis      |
|                                                                         |
|                   Parameters used to calculate RATIO                   |
|                   ----------------------------------                   |
| Ns=0.8453E+3  Msz=0.6348E+2  Msy=0.2708E+2  Mbz=0.6348E+2              |
|               Mrz=0.6348E+2  Mry=0.2707E+2  Moz=0.1130E-4              |
|              Nciz=0.4491E+3 Nciy=0.8453E+3  Ncz=0.4491E+3  Ncy=0.8453E+3 |
|              Vvmy=0.3990E+3 Vvmz=0.2100E+3                              |
| ALPHA,M= 1.533  ALPHA,B=-0.500  ALPHA,SZ= 1.036                        |
|                                                                         |
|                     MAX FORCE/ MOMENT SUMMARY ( KN-METR)                |
|                     -------------------------                           |
|                AXIAL      SHEAR-Y     SHEAR-Z     MOMENT-Y    MOMENT-Z  |
|      VALUE      -0.6        14.0         6.7         3.1         5.5    |
|    LOCATION      0.3         0.3         0.0         0.0         0.3    |
|     LOADING      107         102         103         103         102    |
|*************************************************************************|
|*                       DESIGN SUMMARY ( KN-METR)                       *|
|*                       --------------                                  *|
|*       RESULT/      CRITICAL COND/     RATIO/         LOADING/         *|
|*         FX              MY              MZ           LOCATION         *|
|*        ======================================================         *|
|*          PASS        SLENDERNESS       0.558            108           *|
|*          0.33 T          3.1            0.9             0.00          *|
|*************************************************************************|
|-------------------------------------------------------------------------|
```

8.3.3. DEFLECTION CHECK

- **<u>Deflection</u>**

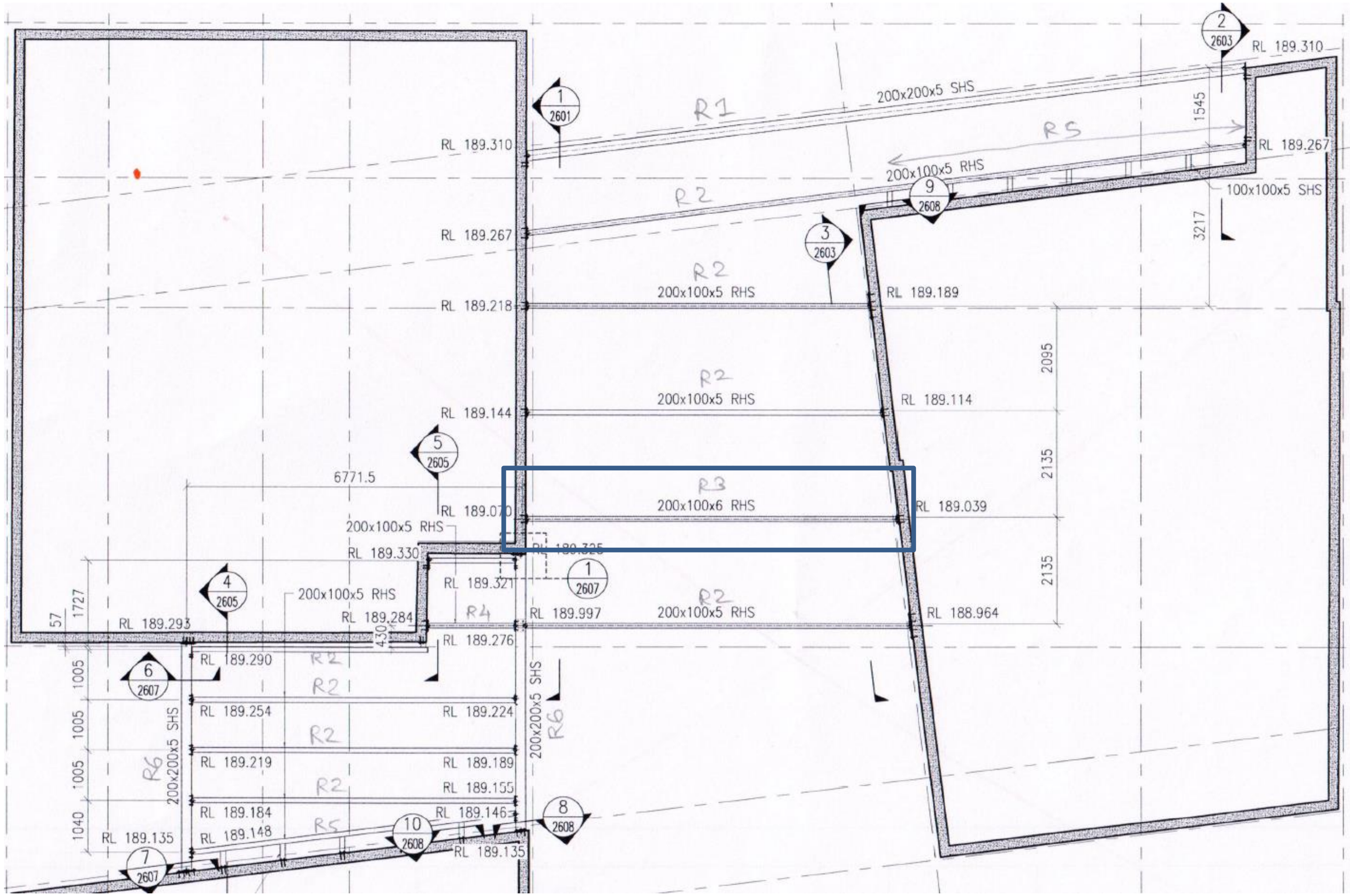
Refer below image shows deflection diagram for governing serviceability load combination **202. 1.0 G + 0.7 Q + 1.0 T**.



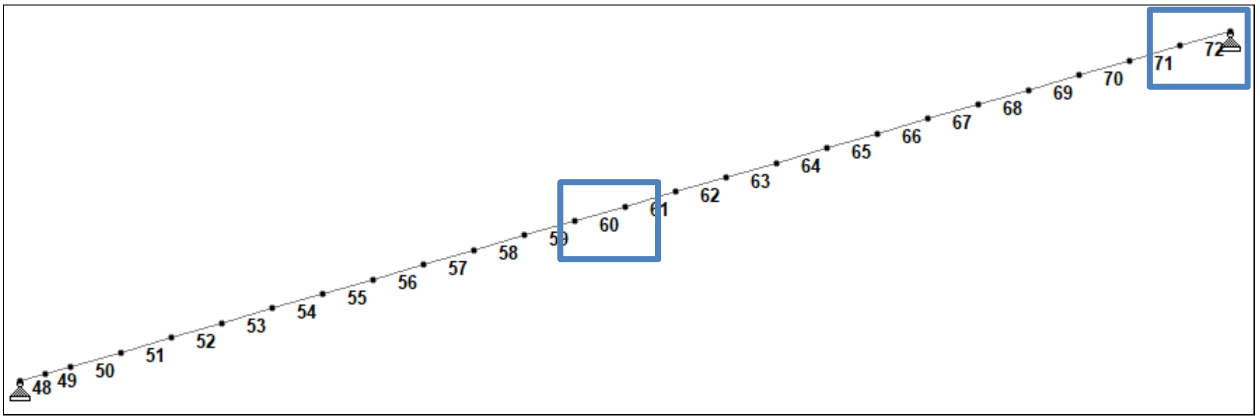
From above displacement diagram,
Maximum net vertical deflection of member in Y direction = 28.5 mm

Permissible Vertical deflection	= Span /250 = 7200 mm / 250 = 28.8 mm
Actual maximum Vertical deflection	= 28.5 mm ≤ 28.8 mm ……. (Hence, OK)

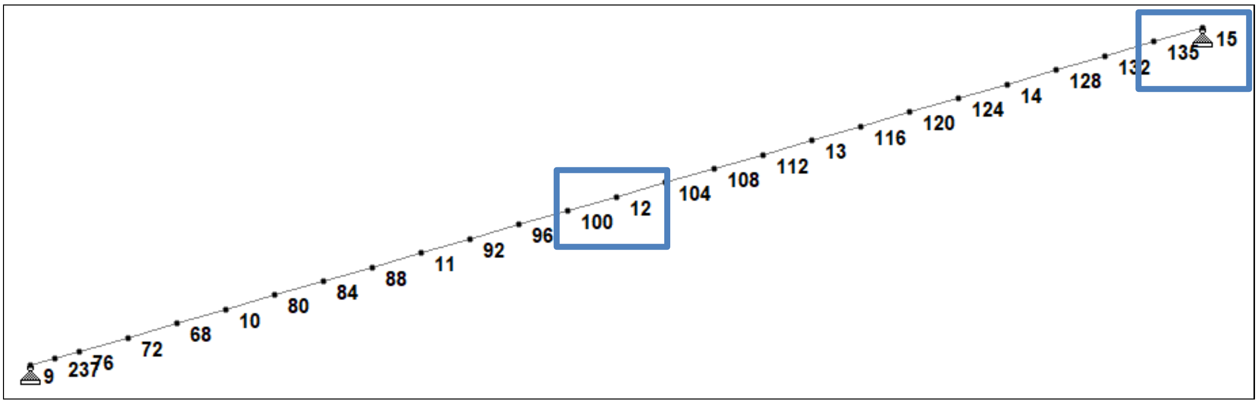
8.3.4. R3 - RHS 200x100x6:



Geometry data of Rafter R3:

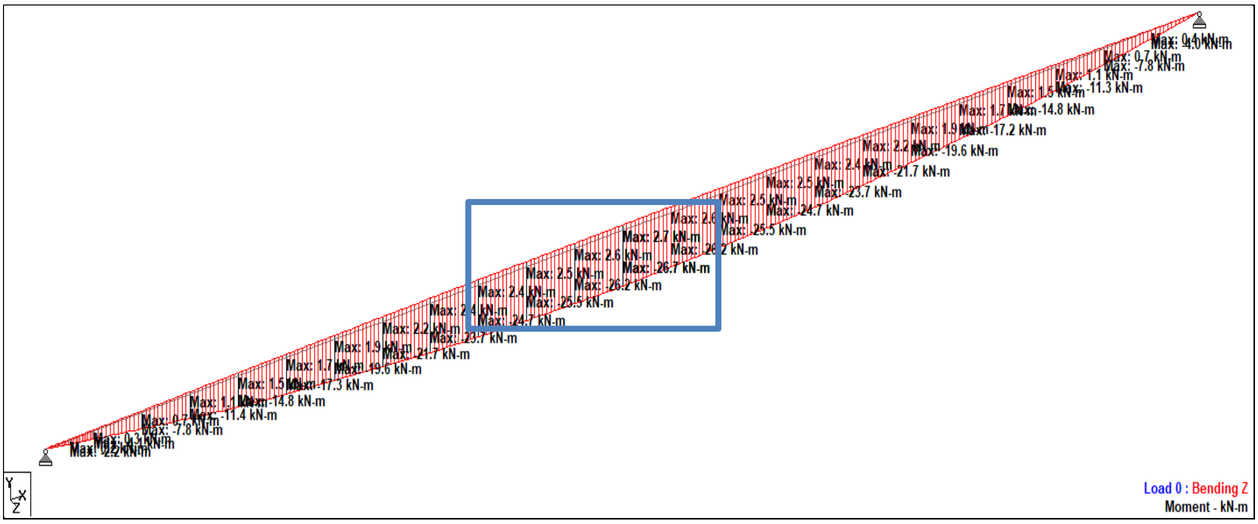


Beam number

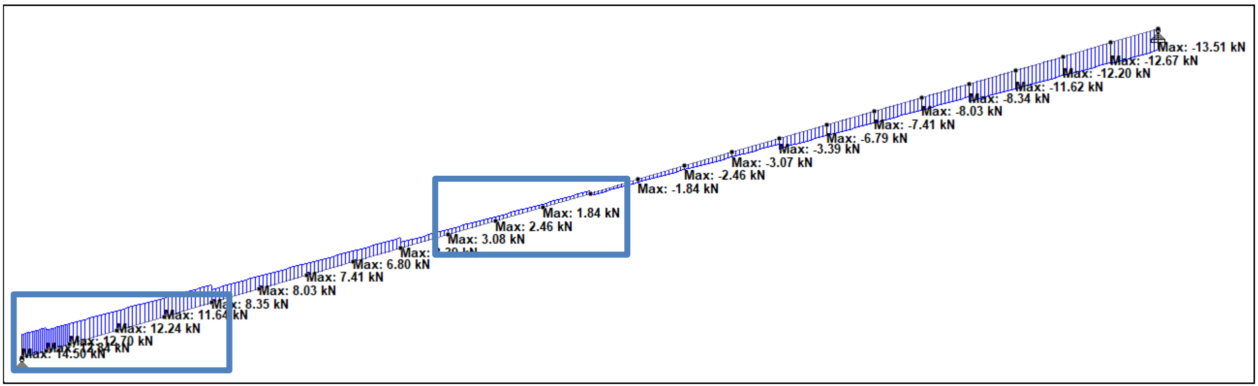


Node number

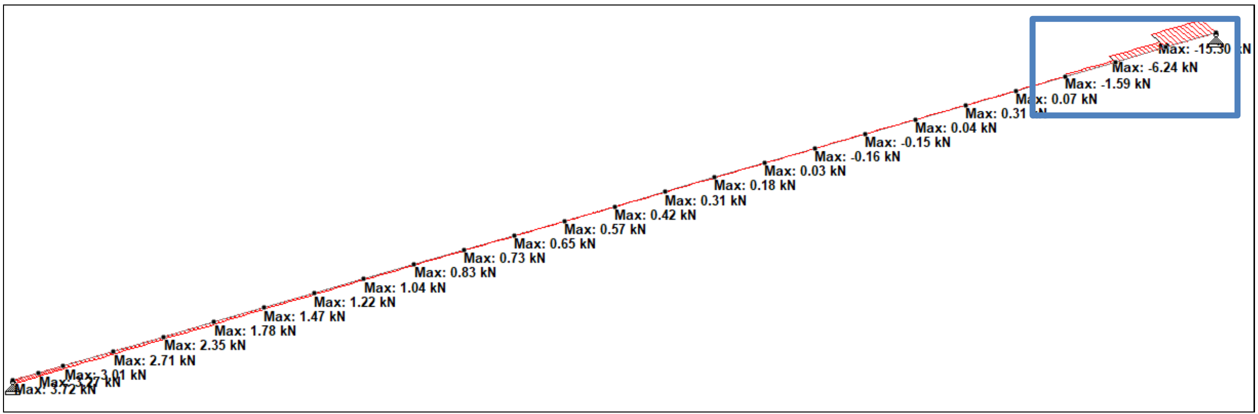
- Analysis results for R3 rafter



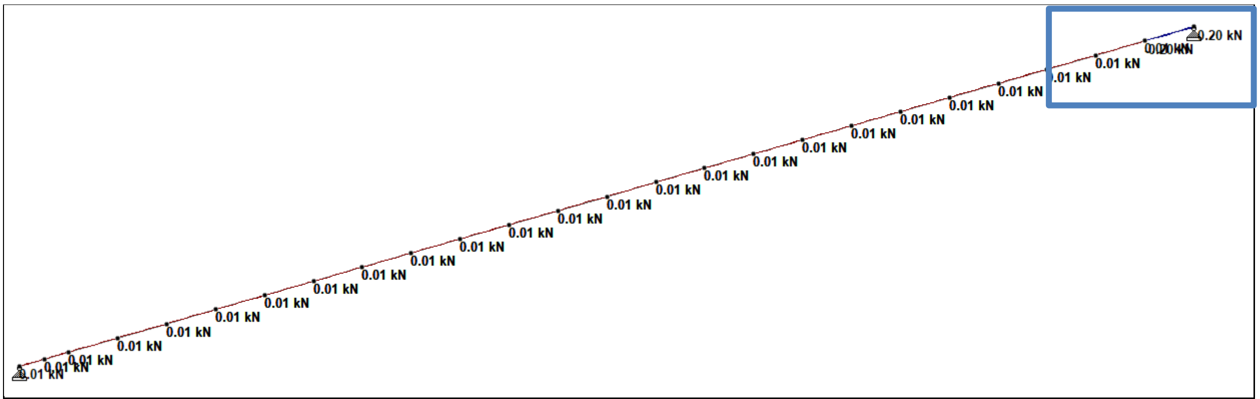
Governing Bending Moment Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fy) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fz) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Axial Force(Fx) Diagram Diagram – 102-1.2G+1.5Q+1.25T

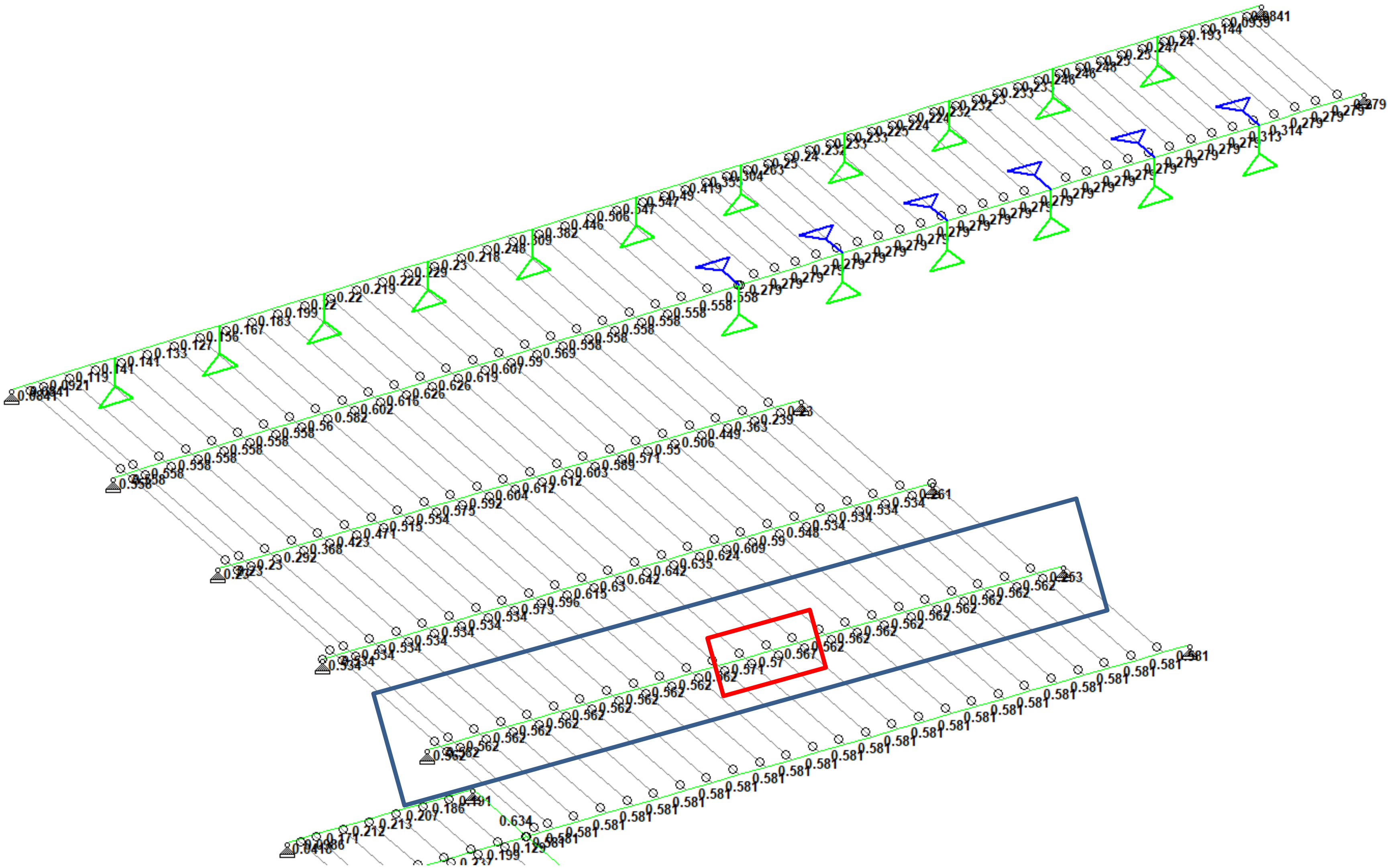
Member forces table for Rafter R3:

Facade Roof_10-04-2024 - R9_Roller support & member added - Beam End Forces: R3

All | Summary | Envelope

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kN-m	My kN-m	Mz kN-m
Max Fx	71	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	132	**0.01**	-2.62	-6.08	0.0	5.5	-1.7
Min Fx	72	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	135	**-0.20**	-2.81	-14.40	0.0	3.6	-0.9
Max Fy	48	102 1.2 G + 1.5 Q + 1.25 T	9	0.01	**14.50**	3.72	0.0	-0.0	0.0
Min Fy	72	102 1.2 G + 1.5 Q + 1.25 T	15	-0.20	**-13.51**	-15.30	0.0	-0.7	0.0
Max Fz	69	106 0.9 G + WL-Z + 1.25 T	14	0.01	-1.95	**4.77**	0.0	7.5	-2.7
Min Fz	72	106 0.9 G + WL-Z + 1.25 T	135	-0.16	-2.39	**-25.24**	0.0	7.0	-0.8
Max Mx	62	102 1.2 G + 1.5 Q + 1.25 T	104	0.01	-1.87	0.31	**0.0**	6.1	-26.2
Min Mx	60	104 0.9 G + WL+Z + 1.25 T	100	0.01	-0.26	0.61	**-0.0**	5.5	2.6
Max My	70	106 0.9 G + WL-Z + 1.25 T	132	0.01	-2.20	1.40	0.0	**9.4**	-1.4
Min My	72	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	15	-0.20	-2.94	-14.40	0.0	**-0.7**	-0.0
Max Mz	60	104 0.9 G + WL+Z + 1.25 T	12	0.01	-0.32	0.61	-0.0	5.6	**2.7**
Min Mz	60	102 1.2 G + 1.5 Q + 1.25 T	12	0.01	1.56	0.57	0.0	6.0	**-26.7**

8.3.5. Design calculation of – R3 - RHS 200x100x6:



Max. Utilization ratio for beam number 60

- Beam no.: 60 - Design calculation for Maximum bending moment

```
|--------------------------------------------------------------------------|
|                                                    Y        PROPERTIES   |
|*************                                       |        IN   CM  UNIT |
|           *   |=============================|                ------------ |
|MEMBER  60 *   |                             |                AX=0.3363E+2 |
|           *   | ST  200X100X6.0RHS          |           --Z  AY=0.2256E+2 |
|DESIGN CODE *  |                             |                AZ=0.1200E+2 |
|AS4100 1998 *  ===============================                PY=0.1315E+3 |
|           *                                                  PZ=0.2133E+3 |
|           *   |<---LENGTH (ME=     0.30 --->|                RY=0.4142E+1 |
|*************                                                 RZ=0.7117E+1 |
|                                                              Iw=0.0000E+0 |
|                 26.7( KN-METR)                                            |
|PARAMETER        |                                     ?102FORCE/MOMENT   |
|IN  NEWTON MM    |                              ?102?102    IN  KN METRE  |
|--------------   +                          ?102            ------------  |
| KL/R-Y=    7.2  |                      ?102                PNC=0.4886E+3|
| KL/R-Z=  101.2  +                  ?102                    PNT=0.1059E+4|
| UNL   =  300.0  |              ?102                        pn =0.8614E-2|
| MAIN  =  180.0  +                                          MNZ=0.6718E+2|
| PHI   =   0.90  |      ?102?102                            mnz=-.2667E+2|
| FULT  =  430.0  +   ?102                                   MNY=0.3431E+2|
| FYLD  =  350.0  |?102                                      mny=0.5958E+1|
| NSF   =   1.00  +---+---+---+---+---+---+---+---+---+---|  VZ =0.2268E+3|
| SKT   =   1.00   26.1                                      vz =0.5667E+0|
| SKL   =   1.00              ABSOLUTE MZ ENVELOPE            VY =0.4264E+3|
| SKR   =   1.00                 (WITH LOAD NO.)              vy =0.1564E+1|
|                                                                          |
| Section Type:  Compact - about Z axis;     Slender - about Y axis        |
|                                                                          |
|                    Parameters used to calculate RATIO                    |
|                    ----------------------------------                    |
| Ns=0.1133E+4  Msz=0.7464E+2  Msy=0.3812E+2  Mbz=0.7464E+2                |
|               Miz=0.7464E+2  Miy=0.3812E+2  Moz=0.7464E+2                 |
|              Nciz=0.5429E+3 Nciy=0.1133E+4  Ncz=0.5429E+3  Ncy=0.1133E+4  |
|              Vvmy=0.4738E+3 Vvmz=0.2520E+3                                |
| ALPHA,M= 0.991  ALPHA,B=-0.500  ALPHA,SZ= 1.036                          |
|                                                                          |
|                      MAX FORCE/ MOMENT SUMMARY ( KN-METR)                |
|                      -------------------------                           |
|                 AXIAL      SHEAR-Y     SHEAR-Z    MOMENT-Y    MOMENT-Z    |
|      VALUE        0.0         1.8         1.0         6.2        26.7     |
|   LOCATION        0.0         0.0         0.0         0.3         0.3     |
|    LOADING        102         102         106         103         102     |
|**************************************************************************|
|*                         DESIGN SUMMARY ( KN-METR)                       *|
|*                         --------------                                  *|
|*       RESULT/     CRITICAL COND/     RATIO/        LOADING/             *|
|*          FX           MY              MZ           LOCATION             *|
|*        ======================================================           *|
|*          PASS        AS-8.3.4         0.571           102               *|
|*          0.01 C           6.0       -26.7            0.30               *|
|**************************************************************************|
|--------------------------------------------------------------------------|
```

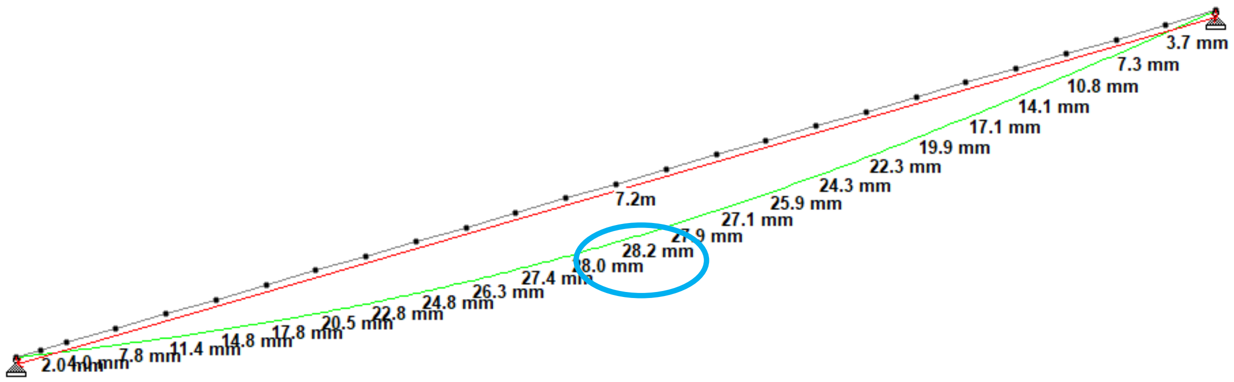
- Beam no.: 72 - Design calculation for Maximum Shear Force

```
|--------------------------------------------------------------------------|
|                                                    Y        PROPERTIES   |
|*************                                      |        IN  CM  UNIT  |
|            *  |=============================|              -----------  |
|MEMBER  72  *  |                             |              AX=0.3363E+2  |
|            *  | ST  200X100X6.0RHS          |          --Z AY=0.2256E+2  |
|DESIGN CODE *  |                             |              AZ=0.1200E+2  |
|AS4100 1998 *  ===============================              PY=0.1315E+3  |
|            *                                               PZ=0.2133E+3  |
|            *  |<---LENGTH (ME=     0.30 --->|               RY=0.4142E+1  |
|*************                                               RZ=0.7117E+1  |
|                                                            Iw=0.0000E+0  |
|                 4.0( KN-METR)                                            |
|PARAMETER      |?102                                        FORCE/MOMENT  |
|IN  NEWTON MM  |   ?102                                     IN  KN METRE  |
|---------------+       ?102?102                             -------------|
| KL/R-Y=    7.2 |              ?102                         PNC=0.4886E+3|
| KL/R-Z=  101.2 +                                           PNT=0.1059E+4|
| UNL    =  300.0 |                  ?102?102                pn =-.1777E+0|
| MAIN   =  180.0 +                                          MNZ=0.6718E+2|
| PHI    =   0.90 |                          ?102            mnz=-.2578E+1|
| FULT   =  430.0 +                              ?102?102    MNY=0.3431E+2|
| FYLD   =  350.0 |                                     ?205mny=0.5389E+1|
| NSF    =   1.00 +---+---+---+---+---+---+---+---+---|      VZ =0.2268E+3|
| SKT    =   1.00  -0.2                                      vz =-.2006E+2|
| SKL    =   1.00             ABSOLUTE MZ ENVELOPE           VY =0.4264E+3|
| SKR    =   1.00                (WITH LOAD NO.)             vy =-.8440E+1|
|                                                                          |
| Section Type:  Compact - about Z axis;     Slender - about Y axis        |
|                                                                          |
|                    Parameters used to calculate RATIO                    |
|                    ----------------------------------                    |
| Ns=0.1133E+4  Msz=0.7464E+2  Msy=0.3812E+2  Mbz=0.7464E+2                |
|               Mrz=0.7464E+2  Mry=0.3812E+2  Moz=0.7464E+2                |
|              Nciz=0.5429E+3 Nciy=0.1133E+4  Ncz=0.5429E+3  Ncy=0.1133E+4 |
|              Vvmy=0.4738E+3 Vvmz=0.2520E+3                               |
| ALPHA,M= 1.802  ALPHA,B=-0.500  ALPHA,SZ= 1.036                          |
|                                                                          |
|                      MAX FORCE/ MOMENT SUMMARY ( KN-METR)                |
|                      -------------------------                           |
|                AXIAL      SHEAR-Y     SHEAR-Z    MOMENT-Y    MOMENT-Z    |
|      VALUE      -0.2        13.5        25.2        7.0         4.0      |
|   LOCATION       0.3         0.3         0.0        0.0         0.0      |
|    LOADING       103         102         106        106         102      |
|**************************************************************************|
|*                          DESIGN SUMMARY ( KN-METR)                     *|
|*                          --------------                                *|
|*       RESULT/     CRITICAL COND/     RATIO/         LOADING/           *|
|*         FX             MY             MZ           LOCATION            *|
|*       ======================================================           *|
|*         PASS        SLENDERNESS      0.253            101              *|
|*         0.18 T          5.4          -2.6             0.00             *|
|**************************************************************************|
```

8.3.6. DEFLECTION CHECK

- **<u>Deflection</u>**

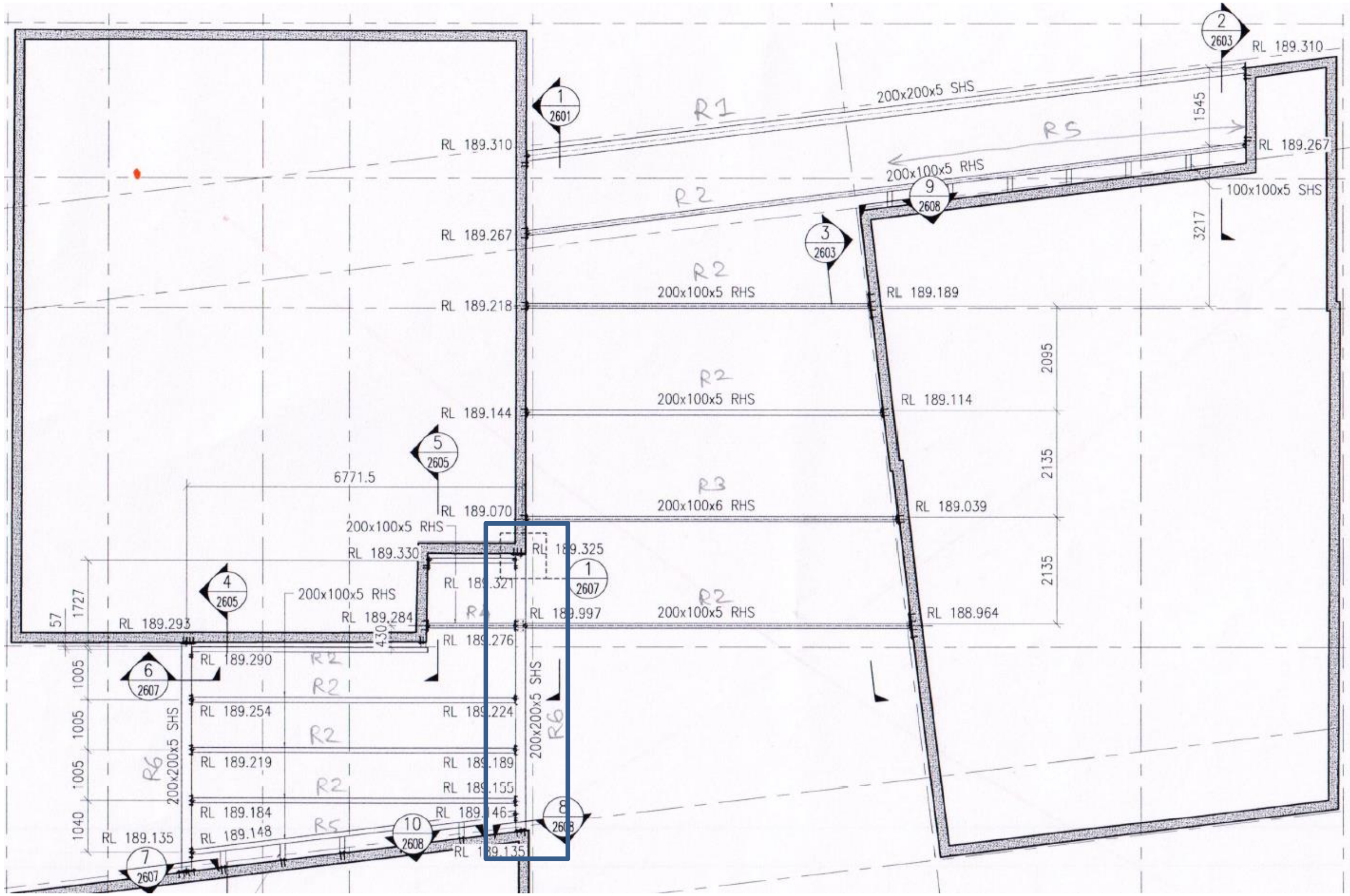
Refer below image shows deflection diagram for governing serviceability load combination **202. 1.0 G + 0.7 Q + 1.0 T**.



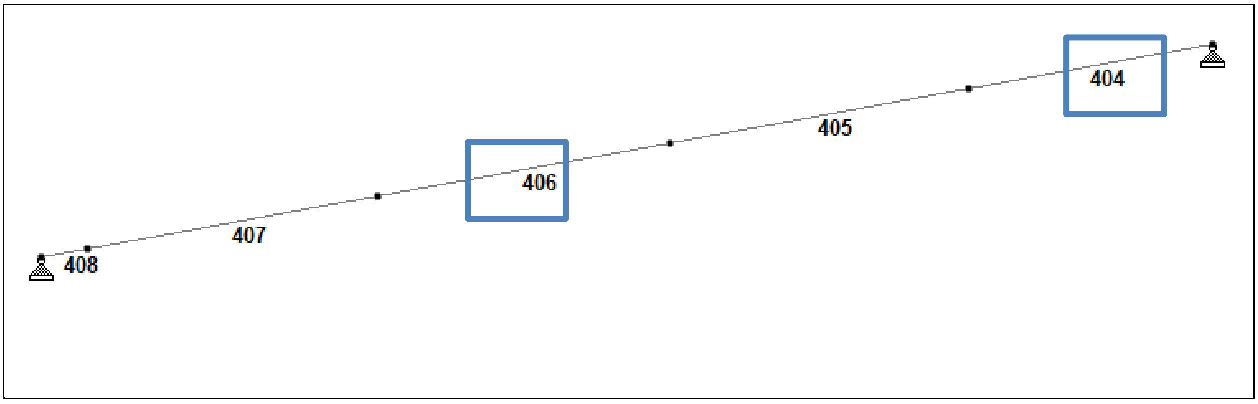
From above displacement diagram,
Maximum net vertical deflection of member in Y direction = 28.2 mm

Permissible Vertical deflection	= Span /250 = 7200 mm / 250 = 28.8 mm
Actual maximum Vertical deflection	= 28.2 mm ≤ 28.8 mm ……. (Hence, OK)

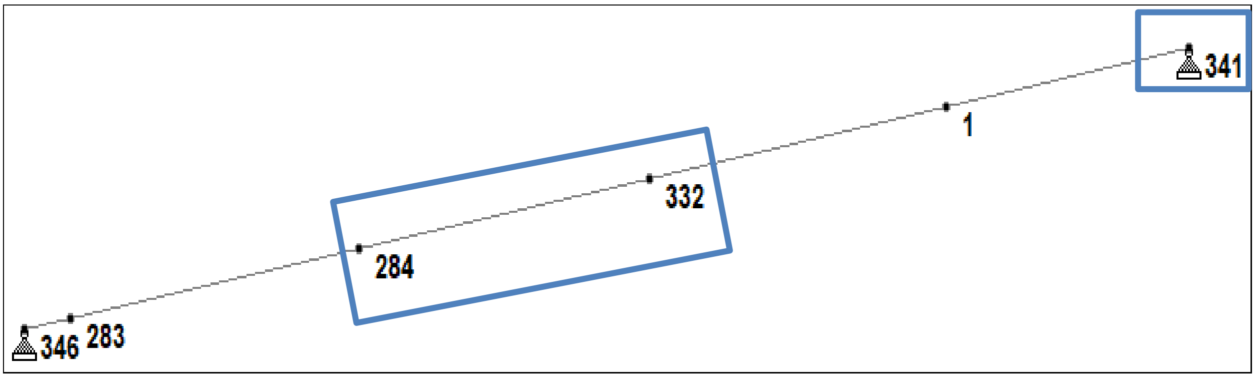
8.3.7. R6 - RHS 200x200x5:



Geometry data of Rafter R6:

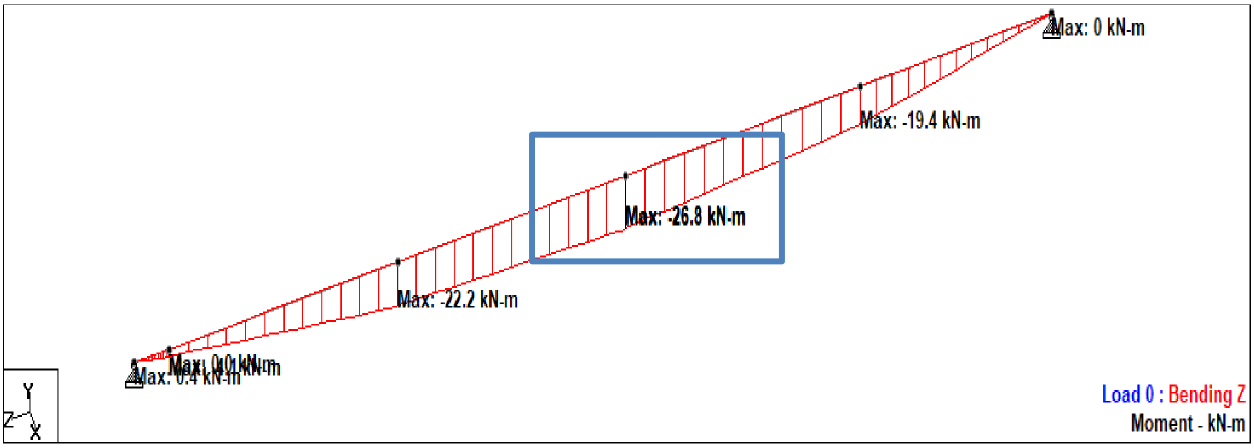


Beam number

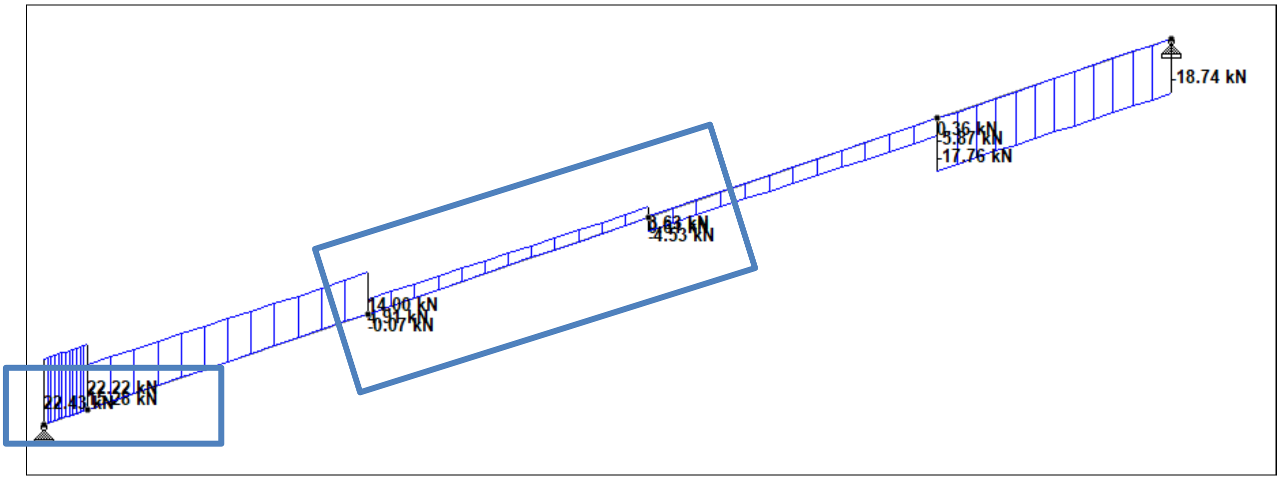


Node number

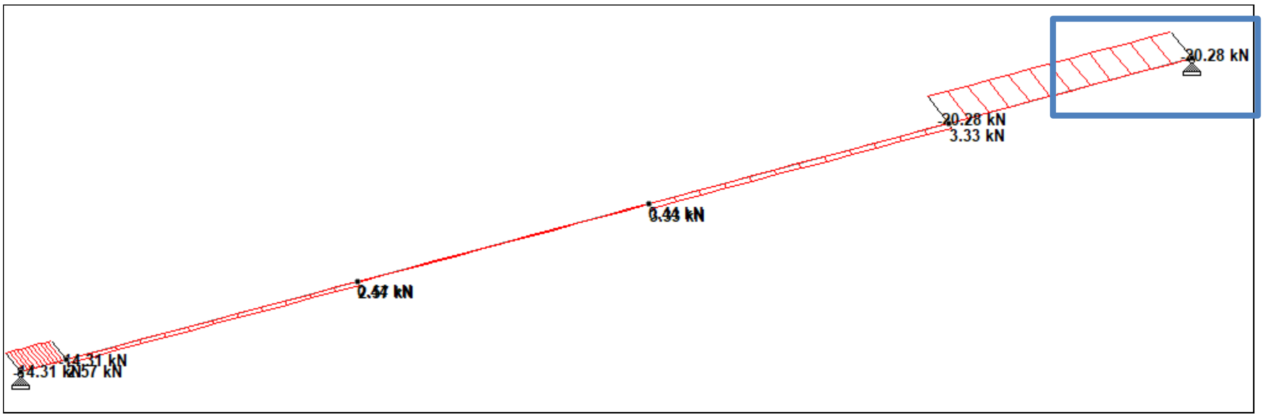
- Analysis results for R6 rafter



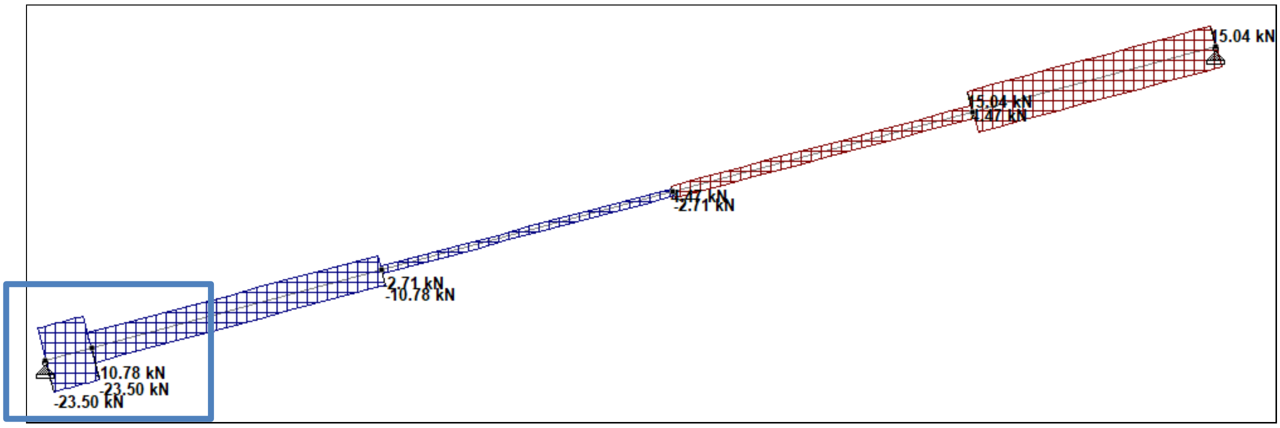
Governing Bending Moment Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fy) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Shear Force(Fz) Diagram Diagram – 102-1.2G+1.5Q+1.25T



Axial Force(Fx) Diagram Diagram – 102-1.2G+1.5Q+1.25T

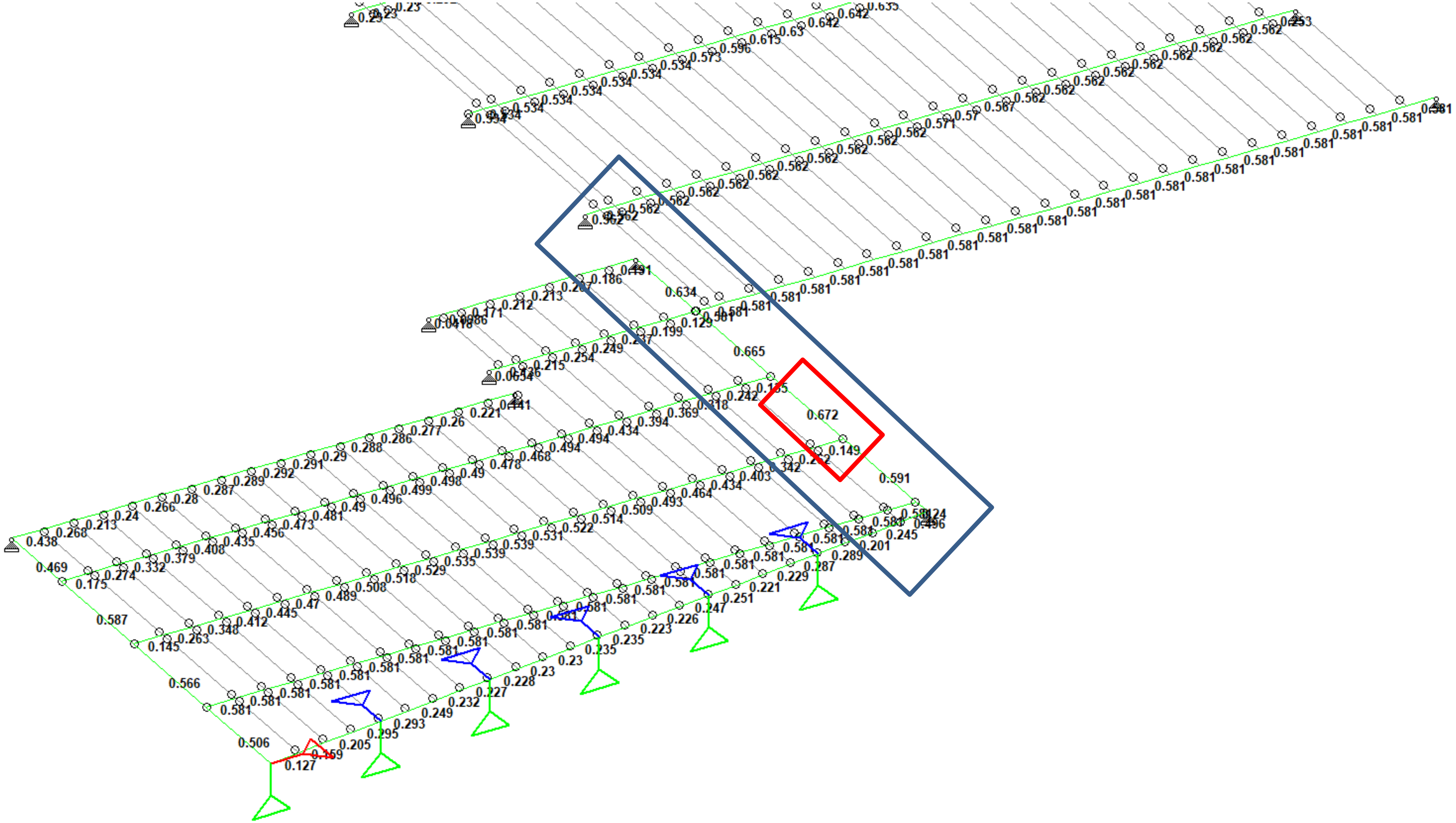
Member forces table for Rafter R6:

Facade Roof_10-04-2024 - R9_Roller support & member added - Beam End Forces: R6

All | Summary | Envelope

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kN-m	My kN-m	Mz kN-m
Max Fx	404	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	1	**15.16**	-6.33	-20.26	-0.2	17.1	-7.0
Min Fx	408	103 1.2 G + WL+Z + 0.6 Q + 1.25 T	346	**-23.53**	7.00	-14.29	-0.2	11.8	0.1
Max Fy	408	102 1.2 G + 1.5 Q + 1.25 T	346	-23.50	**22.43**	-14.29	-0.5	11.8	0.4
Min Fy	404	102 1.2 G + 1.5 Q + 1.25 T	341	15.04	**-18.74**	-20.26	-0.5	-4.5	-0.0
Max Fz	405	110 1.0 G + EQ-Z + 0.6 Q + 1.25 T	332	4.56	-2.89	**3.33**	-0.3	12.8	-17.6
Min Fz	404	110 1.0 G + EQ-Z + 0.6 Q + 1.25 T	1	14.70	-12.06	**-20.28**	-0.3	17.1	-13.2
Max Mx	404	104 0.9 G + WL+Z + 1.25 T	1	14.90	-0.94	-20.26	**-0.0**	17.1	-1.1
Min Mx	404	102 1.2 G + 1.5 Q + 1.25 T	1	15.04	-17.76	-20.26	**-0.5**	17.1	-19.4
Max My	404	110 1.0 G + EQ-Z + 0.6 Q + 1.25 T	1	14.70	-12.06	-20.28	-0.3	**17.1**	-13.2
Min My	404	106 0.9 G + WL-Z + 1.25 T	341	13.73	-5.46	-20.27	-0.1	**-4.5**	0.0
Max Mz	408	102 1.2 G + 1.5 Q + 1.25 T	346	-23.50	22.43	-14.29	-0.5	11.8	**0.4**
Min Mz	406	102 1.2 G + 1.5 Q + 1.25 T	332	-2.71	3.63	0.43	-0.5	12.8	**-26.8**

8.3.8. Design calculation of R6 - RHS 200x200x5:



Max. Utilization ratio for beam number 406

- Beam no.: 406 - Design calculation for Maximum bending moment

```
|-----------------------------------------------------------------------------|
|                                                      Y        PROPERTIES    |
|*************                                         |        IN  CM  UNIT  |
|            *  |=============================|                 ------------  |
|MEMBER 406  *  |                             |                 AX=0.3836E+2  |
|            *  | ST  200X200X5.0SHS          |             --Z AY=0.1900E+2  |
|DESIGN CODE *  |                             |                 AZ=0.2000E+2  |
|AS4100 1998 *  ===============================                 PY=0.2789E+3  |
|            *                                                  PZ=0.2789E+3  |
|            *  |<---LENGTH (ME=     1.28 --->|                 RY=0.7926E+1  |
|*************                                                  RZ=0.7926E+1  |
|                                                               Iw=0.0000E+0  |
|                 26.8( KN-METR)                                               |
|PARAMETER        |                                   ?102FORCE/MOMENT        |
|IN  NEWTON MM    |                           ?102?102    IN  KN METRE        |
|---------------  +                       ?102            -------------       |
| KL/R-Y=   16.1  |                   ?102                PNC=0.9380E+3|
| KL/R-Z=   16.1  +               ?102                    PNT=0.1208E+4|
| UNL   = 1275.0  |           ?102                        pn =-.2713E+1|
| MAIN  =  180.0  +       ?102                            MNZ=0.5910E+2|
| PHI   =   0.90  |   ?102                                mnz=-.2682E+2|
| FULT  =  430.0  + ?102                                  MNY=0.5910E+2|
| FYLD  =  350.0  |?102                                   mny=0.1277E+2|
| NSF   =   1.00  +---+---+---+---+---+---+---+---+---+---|  VZ =0.3780E+3|
| SKT   =   1.00   21.1                                   vz =0.4272E+0|
| SKL   =   1.00              ABSOLUTE MZ ENVELOPE        VY =0.3591E+3|
| SKR   =   1.00                 (WITH LOAD NO.)          vy =0.3627E+1|
|                                                                             |
| Section Type:  Slender - about Z axis;     Slender - about Y axis          |
|                                                                             |
|                    Parameters used to calculate RATIO                       |
|                    ----------------------------------                       |
| Ns=0.1048E+4  Msz=0.6566E+2  Msy=0.6566E+2  Mbz=0.6566E+2                  |
|               Mrz=0.6552E+2  Mry=0.6552E+2  Moz=0.6552E+2                  |
|              Nciz=0.1042E+4 Nciy=0.1042E+4  Ncz=0.1042E+4  Ncy=0.1042E+4  |
|              Vvmy=0.3990E+3 Vvmz=0.4200E+3                                 |
| ALPHA,M= 1.082  ALPHA,B=-0.500  ALPHA,SZ= 1.035                           |
|                                                                             |
|                      MAX FORCE/ MOMENT SUMMARY ( KN-METR)                   |
|                      -------------------------                              |
|                AXIAL       SHEAR-Y     SHEAR-Z     MOMENT-Y    MOMENT-Z    |
|     VALUE       -2.7          4.9         0.4        12.8        26.8      |
|   LOCATION       1.3          0.0         0.0         1.3         1.3      |
|    LOADING       103          102         109         109         102      |
|*****************************************************************************|
|*                          DESIGN SUMMARY ( KN-METR)                        *|
|*                          --------------                                   *|
|*       RESULT/      CRITICAL COND/     RATIO/         LOADING/             *|
|*         FX               MY             MZ           LOCATION             *|
|*        ======================================================             *|
|*          PASS        AS-8.3.4          0.672           102                *|
|*          2.71 T          12.8          -26.8           1.28               *|
|*****************************************************************************|
|-----------------------------------------------------------------------------|
```

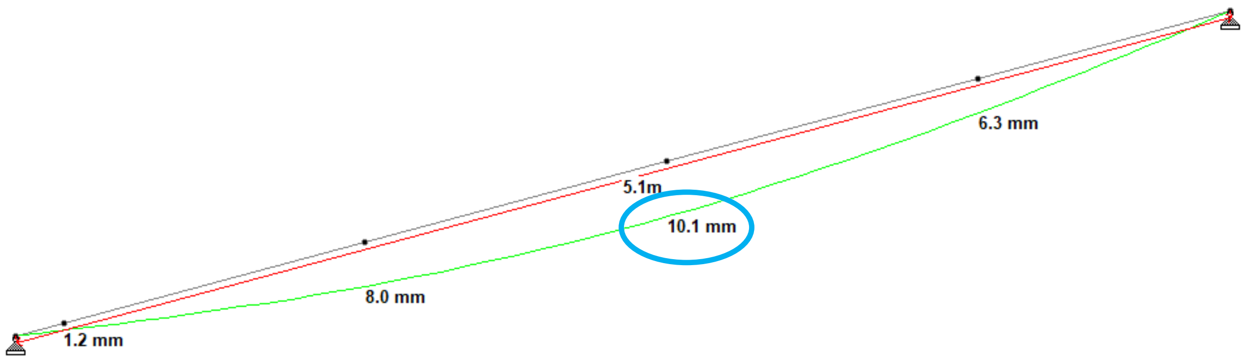
- Beam no.: 408 - Design calculation for Maximum Shear Force

```
|-----------------------------------------------------------------------------|
|                                                     Y        PROPERTIES     |
|*************                                        |        IN  CM  UNIT   |
|             *  |=============================|                 ------------ |
|MEMBER 408   *  |                             |               AX=0.3836E+2 |
|             *  | ST  200X200X5.0SHS          |        --Z    AY=0.1900E+2 |
|DESIGN CODE  *  |                             |               AZ=0.2000E+2 |
|AS4100 1998  *  ===============================               PY=0.2789E+3 |
|             *                                                PZ=0.2789E+3 |
|             *  |<---LENGTH (ME=     0.20 --->|               RY=0.7926E+1 |
|*************                                                 RZ=0.7926E+1 |
|                                                              Iw=0.0000E+0 |
|                 4.1( KN-METR)                                              |
|PARAMETER         |                                 ?102FORCE/MOMENT       |
|IN  NEWTON MM     |                             ?102     IN  KN METRE      |
|---------------   +                     ?102?102         -------------     |
| KL/R-Y=    2.5   |                 ?102                 PNC=0.9431E+3|
| KL/R-Z=    2.5   +                                      PNT=0.1208E+4|
| UNL   =  200.0   |             ?102                     pn =-.2350E+2|
| MAIN  =  180.0   +         ?102                         MNZ=0.5910E+2|
| PHI   =   0.90   |     ?102                             mnz=-.4088E+1|
| FULT  =  430.0   +?102   ?102                           MNY=0.5910E+2|
| FYLD  =  350.0   |   ?205                               mny=0.8971E+1|
| NSF   =   1.00   +---+---+---+---+---+---+---+---+---+---|  VZ =0.3780E+3|
| SKT   =   1.00    -0.2                                  vz =-.1429E+2|
| SKL   =   1.00              ABSOLUTE MZ ENVELOPE        VY =0.3591E+3|
| SKR   =   1.00                 (WITH LOAD NO.)          vy =0.2222E+2|
|                                                                             |
| Section Type:  Slender - about Z axis;     Slender - about Y axis          |
|                                                                             |
|                    Parameters used to calculate RATIO                      |
|                    ----------------------------------                      |
| Ns=0.1048E+4  Msz=0.6566E+2  Msy=0.6566E+2  Mbz=0.6566E+2                  |
|               Mrz=0.6439E+2  Mry=0.6439E+2  Moz=0.6439E+2                  |
|              Nciz=0.1048E+4 Nciy=0.1048E+4  Ncz=0.1048E+4  Ncy=0.1048E+4   |
|              Vvmy=0.3990E+3 Vvmz=0.4200E+3                                 |
| ALPHA,M= 1.937  ALPHA,B=-0.500  ALPHA,SZ= 1.039                            |
|                                                                             |
|                      MAX FORCE/ MOMENT SUMMARY ( KN-METR)                   |
|                      -------------------------                              |
|                 AXIAL      SHEAR-Y     SHEAR-Z    MOMENT-Y     MOMENT-Z     |
|      VALUE      -23.5        22.4        14.3        11.8          4.1      |
|    LOCATION       0.2         0.0         0.0         0.0          0.2      |
|     LOADING       103         102         109         110          102      |
|*****************************************************************************|
|*                         DESIGN SUMMARY ( KN-METR)                        *|
|*                         --------------                                   *|
|*       RESULT/     CRITICAL COND/     RATIO/        LOADING/              *|
|*         FX             MY             MZ           LOCATION              *|
|*         ======================================================           *|
|*        PASS        AS-8.3.4          0.240           102                 *|
|*       23.50 T         9.0            -4.1            0.20                *|
|*****************************************************************************|
|-----------------------------------------------------------------------------|
```

8.3.9. DEFLECTION CHECK

- **<u>Deflection</u>**

Refer below image shows deflection diagram for governing serviceability load combination **202. 1.0 G + 0.7 Q + 1.0 T**.

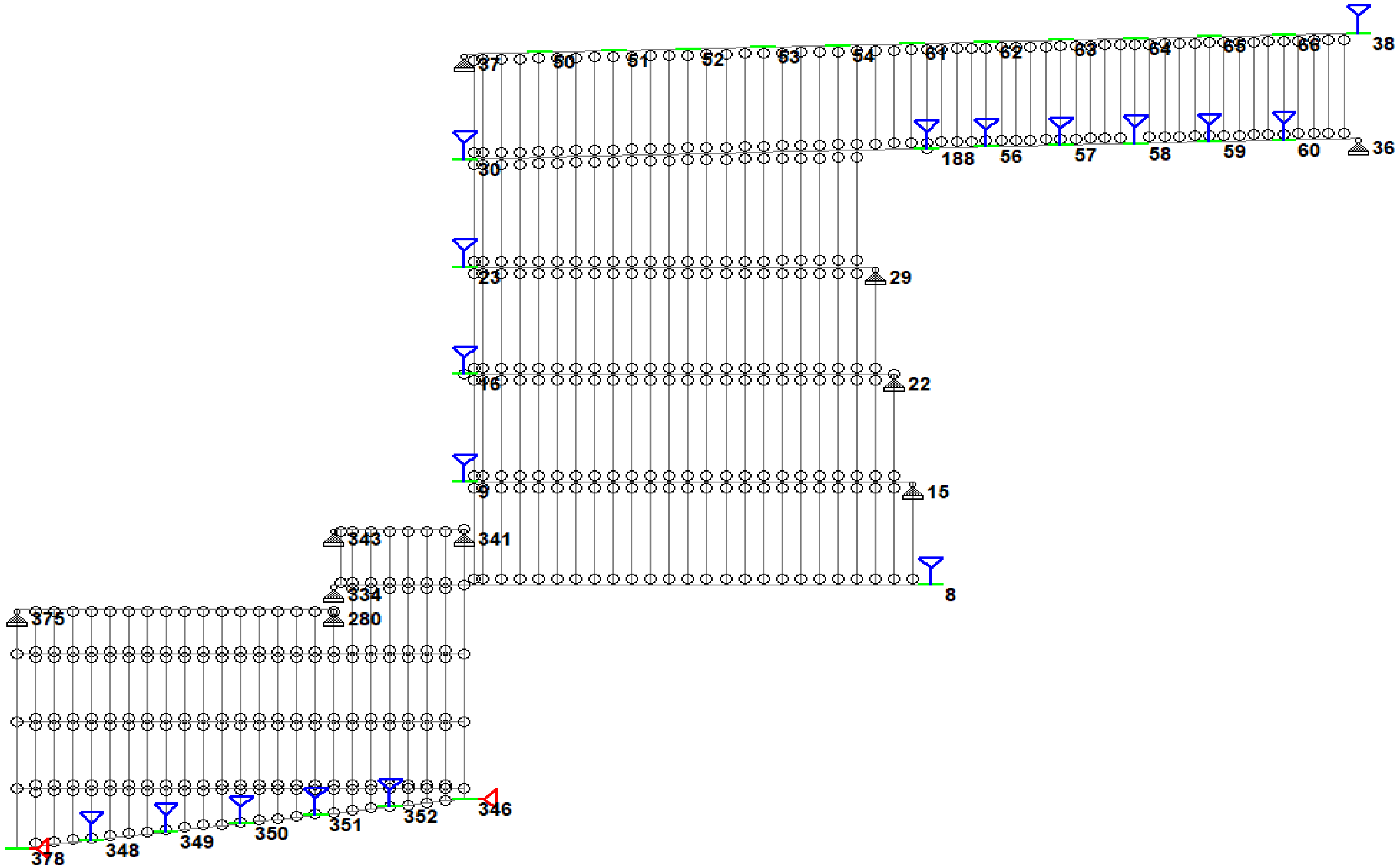


From above displacement diagram,
Maximum net vertical deflection of member in Y direction = 10.1 mm

Permissible Vertical deflection	= Span /250 = 5100 mm / 250 = 20.4 mm
Actual maximum Vertical deflection	= 10.1 mm ≤ 20.4 mm ……. (Hence, OK)

8.4. **SUPPORT REACTION**

Refer below image showing supports.



Supports with Node Numbers

SUPPORT REACTIONS FOR ULS LOAD COMBINATIONS:

Node	L/C	Force-X kN	Force-Y kN	Force-Z kN
8	101	0	7.57	20.46
	102	0	9.77	15.75
	103	0	4.27	14.86
	104	0	1.37	16.75
	105	0	6.25	23.71
	106	0	3.35	25.59
	107	0	6.82	18.58
	108	0	6.82	18.57
	109	0	6.82	18.49
	110	0	6.82	18.65
9	101	0	9.32	2.74
	102	0	14.5	3.71
	103	0	3.25	3.89
	104	0	-1.31	3.51
	105	0	7.3	2.07
	106	0	2.74	1.68
	107	0	9.39	3.13
	108	0	9.39	3.13
	109	0	9.39	2.62
	110	0	9.39	3.64
15	101	-0.13	9.14	3.83
	102	-0.13	14.34	2.64
	103	-0.13	3.11	2.42
	104	-0.13	-1.41	2.89
	105	-0.13	7.15	4.65
	106	-0.13	2.63	5.12
	107	-1	9.26	3.35
	108	0.74	9.26	3.35
	109	-0.13	9.26	2.98
	110	-0.13	9.26	3.72
16	101	0	8.91	2.41
	102	0	14	3.24
	103	0	2.99	3.4
	104	0	-1.42	3.07
	105	0	6.96	1.84
	106	0	2.54	1.5
	107	0	9.03	2.74
	108	0	9.03	2.75
	109	0	9.03	2.27
	110	0	9.03	3.21
22	101	-0.04	8.67	-0.46
	102	-0.02	13.8	0.63

	103	-0.01	2.78	0.84
	104	-0.02	-1.58	0.4
	105	-0.06	6.75	-1.22
	106	-0.07	2.38	-1.66
	107	-0.86	8.86	-0.03
	108	0.8	8.86	-0.02
	109	-0.03	8.86	-0.51
	110	-0.03	8.86	0.46
23	101	0	8.6	2.9
	102	0	13.53	3.73
	103	0	2.89	3.89
	104	0	-1.38	3.56
	105	0	6.72	2.32
	106	0	2.45	1.99
	107	0	8.73	3.23
	108	0	8.73	3.23
	109	0	8.73	2.78
	110	0	8.73	3.69
29	101	-0.15	8.56	-32.09
	102	-0.2	13.59	-14.25
	103	-0.2	2.77	-10.9
	104	-0.19	-1.53	-18.02
	105	-0.12	6.67	-44.38
	106	-0.1	2.37	-51.5
	107	-0.98	8.74	-24.98
	108	0.64	8.74	-24.92
	109	-0.16	8.74	-26.46
	110	-0.17	8.74	-23.43
30	101	0	8.8	4.37
	102	0	13.81	5.2
	103	0	2.97	5.35
	104	0	-1.38	5.02
	105	0	6.87	3.81
	106	0	2.52	3.48
	107	0	8.91	4.69
	108	0	8.91	4.71
	109	0	8.91	4.3
	110	0	8.91	5.11
36	101	-0.39	0.91	7.13
	102	-0.39	1.09	7.34
	103	-0.39	0.58	7.39
	104	-0.39	0.27	7.3
	105	-0.39	0.76	6.98
	106	-0.39	0.45	6.89

	107	-1.97	0.79	7.25
	108	1.18	0.79	7.18
	109	-0.39	0.79	7.03
	110	-0.39	0.79	7.4
37	101	0.11	0.58	-7.4
	102	0.14	0.74	-1.8
	103	0.14	0.33	-0.73
	104	0.21	0.12	-2.97
	105	0.14	0.48	-11.27
	106	0.21	0.26	-13.51
	107	-2.49	0.52	-5.09
	108	2.86	0.52	-5.22
	109	0.18	0.52	-5.51
	110	0.18	0.52	-4.81
38	101	0	0.65	10.17
	102	0	0.85	11.97
	103	0	0.37	12.32
	104	0	0.11	11.6
	105	0	0.54	8.92
	106	0	0.29	8.2
	107	0	0.59	10.89
	108	0	0.59	10.9
	109	0	0.59	10.75
	110	0	0.59	11.03
50	101	0	2.23	0
	102	0	3.23	0
	103	0	0.98	0
	104	0	-0.02	0
	105	0	1.79	0
	106	0	0.79	0
	107	0	2.15	0
	108	0	2.15	0
	109	0	2.15	0
	110	0	2.15	0
51	101	0	1.9	0
	102	0	2.77	0
	103	0	0.82	0
	104	0	-0.04	0
	105	0	1.52	0
	106	0	0.67	0
	107	0	1.84	0
	108	0	1.84	0
	109	0	1.84	0
	110	0	1.84	0

52	101	0	1.99	0
	102	0	2.89	0
	103	0	0.86	0
	104	0	-0.04	0
	105	0	1.59	0
	106	0	0.7	0
	107	0	1.92	0
	108	0	1.92	0
	109	0	1.92	0
	110	0	1.92	0
53	101	0	1.96	0
	102	0	2.85	0
	103	0	0.84	0
	104	0	-0.03	0
	105	0	1.57	0
	106	0	0.69	0
	107	0	1.9	0
	108	0	1.9	0
	109	0	1.9	0
	110	0	1.9	0
54	101	0	1.95	0
	102	0	2.83	0
	103	0	0.84	0
	104	0	-0.03	0
	105	0	1.56	0
	106	0	0.68	0
	107	0	1.88	0
	108	0	1.88	0
	109	0	1.88	0
	110	0	1.88	0
56	101	0	-6.15	-1.32
	102	0	-10.4	-0.68
	103	0	-1.49	-0.46
	104	0	1.85	-0.76
	105	0	-4.69	-1.7
	106	0	-1.36	-2
	107	0	-6.53	-1.1
	108	0	-6.53	-1.1
	109	0	-6.53	-1.14
	110	0	-6.53	-1.05
57	101	0	4.03	4.3
	102	0	5.7	7.18
	103	0	1.88	7.78
	104	0	0.14	6.6

	105	0	3.25	2.35
	106	0	1.51	1.17
	107	0	3.83	5.43
	108	0	3.83	5.43
	109	0	3.83	5.31
	110	0	3.83	5.55
58	101	0	2.2	0.31
	102	0	2.79	4.7
	103	0	1.27	5.52
	104	0	0.45	3.77
	105	0	1.82	-2.71
	106	0	1	-4.46
	107	0	1.96	2.06
	108	0	1.96	2.08
	109	0	1.96	1.62
	110	0	1.96	2.52
59	101	0	2.54	-0.05
	102	0	3.37	4.37
	103	0	1.36	5.2
	104	0	0.36	3.43
	105	0	2.09	-3.11
	106	0	1.08	-4.87
	107	0	2.33	1.71
	108	0	2.33	1.73
	109	0	2.33	1.48
	110	0	2.33	1.96
60	101	0	2.81	-18.31
	102	0	3.64	-14.49
	103	0	1.57	-13.77
	104	0	0.49	-15.3
	105	0	2.32	-20.94
	106	0	1.24	-22.46
	107	0	2.54	-16.79
	108	0	2.54	-16.77
	109	0	2.54	-16.99
	110	0	2.54	-16.57
61	101	0	1.86	0
	102	0	2.69	0
	103	0	0.82	0
	104	0	-0.01	0
	105	0	1.49	0
	106	0	0.66	0
	107	0	1.79	0
	108	0	1.79	0

	109	0	1.79	0
	110	0	1.79	0
62	101	0	1.88	0
	102	0	2.71	0
	103	0	0.83	0
	104	0	0	0
	105	0	1.51	0
	106	0	0.67	0
	107	0	1.81	0
	108	0	1.81	0
	109	0	1.81	0
	110	0	1.81	0
63	101	0	1.93	0
	102	0	2.8	0
	103	0	0.85	0
	104	0	-0.02	0
	105	0	1.55	0
	106	0	0.69	0
	107	0	1.86	0
	108	0	1.86	0
	109	0	1.86	0
	110	0	1.86	0
64	101	0	1.95	0
	102	0	2.82	0
	103	0	0.85	0
	104	0	-0.01	0
	105	0	1.56	0
	106	0	0.69	0
	107	0	1.88	0
	108	0	1.88	0
	109	0	1.88	0
	110	0	1.88	0
65	101	0	1.87	0
	102	0	2.72	0
	103	0	0.82	0
	104	0	-0.02	0
	105	0	1.5	0
	106	0	0.66	0
	107	0	1.81	0
	108	0	1.81	0
	109	0	1.81	0
	110	0	1.81	0
66	101	0	2.14	0
	102	0	3.06	0

	103	0	0.97	0
	104	0	0.04	0
	105	0	1.72	0
	106	0	0.79	0
	107	0	2.05	0
	108	0	2.05	0
	109	0	2.05	0
	110	0	2.05	0
188	101	0	18.17	-0.42
	102	0	28.12	8.52
	103	0	6.47	10.24
	104	0	-2.35	6.65
	105	0	14.26	-6.56
	106	0	5.44	-10.16
	107	0	18.25	3.11
	108	0	18.25	3.17
	109	0	18.25	1.97
	110	0	18.25	4.3
280	101	0	2.16	4.09
	102	0	2.85	4.09
	103	0	1.16	4.09
	104	0	0.31	4.09
	105	0	1.77	4.09
	106	0	0.92	4.09
	107	-0.12	1.97	4.09
	108	0.12	1.97	4.09
	109	0	1.97	3.97
	110	0	1.97	4.2
334	101	-17.97	1.69	8.15
	102	-17.95	2.46	8.15
	103	-17.95	0.72	8.15
	104	-17.95	-0.04	8.15
	105	-17.98	1.35	8.15
	106	-17.99	0.59	8.15
	107	-18.39	1.63	8.15
	108	-17.53	1.63	8.15
	109	-17.97	1.63	8.06
	110	-17.96	1.63	8.24
341	101	15.46	14.61	19.79
	102	15.46	19.95	20.58
	103	15.46	7.35	20.73
	104	15.46	1.31	20.41
	105	15.46	11.89	19.24
	106	15.46	5.85	18.92

	107	14.84	13.61	20.1
	108	16.08	13.61	20.11
	109	15.46	13.61	18.6
	110	15.46	13.61	21.61
343	101	0	1.01	8.03
	102	0	1.33	8.03
	103	0	0.55	8.03
	104	0	0.15	8.03
	105	0	0.83	8.03
	106	0	0.43	8.03
	107	-0.09	0.92	8.03
	108	0.09	0.92	8.03
	109	0	0.92	7.94
	110	0	0.92	8.12
346	101	-15.35	16.05	0
	102	-15.34	22.76	0
	103	-15.34	7.4	0
	104	-15.34	0.44	0
	105	-15.36	12.93	0
	106	-15.36	5.97	0
	107	-16.25	15.29	0
	108	-14.45	15.29	0
	109	-15.35	15.29	0
	110	-15.34	15.29	0
348	101	0	3.28	-15.56
	102	0	4.03	-15.56
	103	0	2.02	-15.56
	104	0	0.84	-15.56
	105	0	2.74	-15.56
	106	0	1.57	-15.56
	107	0	2.88	-15.56
	108	0	2.88	-15.57
	109	0	2.88	-15.58
	110	0	2.88	-15.54
349	101	0	1.53	1.46
	102	0	1.71	1.46
	103	0	1.07	1.46
	104	0	0.59	1.46
	105	0	1.3	1.46
	106	0	0.82	1.46
	107	0	1.27	1.46
	108	0	1.27	1.46
	109	0	1.27	1.47
	110	0	1.27	1.45

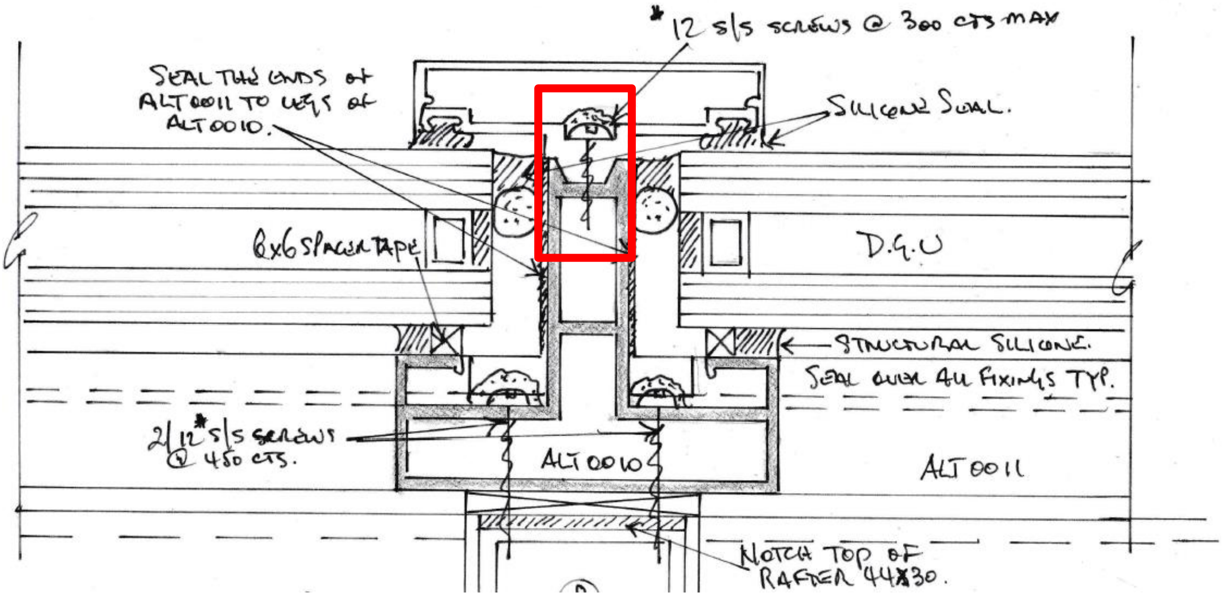
350	101	0	1.85	-1.74
	102	0	2.08	-1.74
	103	0	1.28	-1.74
	104	0	0.7	-1.74
	105	0	1.57	-1.74
	106	0	0.98	-1.74
	107	0	1.55	-1.74
	108	0	1.55	-1.74
	109	0	1.55	-1.74
	110	0	1.55	-1.74
351	101	0	1.77	-7.37
	102	0	1.99	-7.37
	103	0	1.24	-7.38
	104	0	0.68	-7.37
	105	0	1.51	-7.37
	106	0	0.95	-7.37
	107	0	1.48	-7.37
	108	0	1.48	-7.37
	109	0	1.48	-7.36
	110	0	1.48	-7.38
352	101	0	1.48	-29.25
	102	0	1.42	-29.24
	103	0	1.23	-29.24
	104	0	0.86	-29.24
	105	0	1.3	-29.26
	106	0	0.93	-29.27
	107	0	1.14	-29.25
	108	0	1.14	-29.25
	109	0	1.14	-29.3
	110	0	1.14	-29.2
375	101	2.46	13.46	14.15
	102	2.46	19.06	14.15
	103	2.46	6.22	14.15
	104	2.46	0.39	14.15
	105	2.46	10.84	14.15
	106	2.46	5.01	14.15
	107	1.9	12.81	14.15
	108	3.02	12.81	14.15
	109	2.45	12.81	12.99
	110	2.46	12.81	15.31
378	101	15.72	8.93	0
	102	15.71	12.66	0
	103	15.71	4.12	0
	104	15.72	0.24	0

	105	15.73	7.19	0
	106	15.74	3.32	0
	107	15.1	8.5	0
	108	16.34	8.5	0
	109	15.73	8.5	0
	110	15.71	8.5	0

9. CONNECTION DESIGN

9.1. CONNECTION-1_ Connection design of Extrusions

Part-1



Maximum Wind pressure on Glass = 1.088 kN/m^2

UDL on frame due to wind load = 1.088 kN/m^2 x 1.2m

= 1.306 kN/m

C/C distance between two screws = 300mm

Maximum pullout on one screw = 1.306 kN/m x 0.3m

= 0.39 kN

Pull-out capacity of 8G head self-drilling screw = 2.3 kN to 9.5 kN

Hence, Provided 8G screws are safe in pullout.

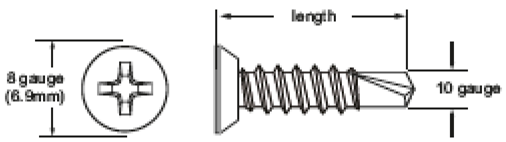
TECHNICAL DATA SHEET:

UNDERCUT 8G HEAD SELF DRILLING SCREWS

ZINC PLATED

Undercut screws have a flat underside surface unlike conventional countersunk screw head types – this is designed to fix metal hinges or metal plates securely through a countersink hole. This offers better installation than a standard countersunk head as the flat underside sits hard up against the surface.

These 8g head diameter type offered by Allfasteners feature a smaller heard on a heavier 10g self drilling screw thread.



- lengths 16mm and 20mm
- phillips drive
- self drill point no.3
- carbon steel heat treated to AS3566.1
- electroplated zinc

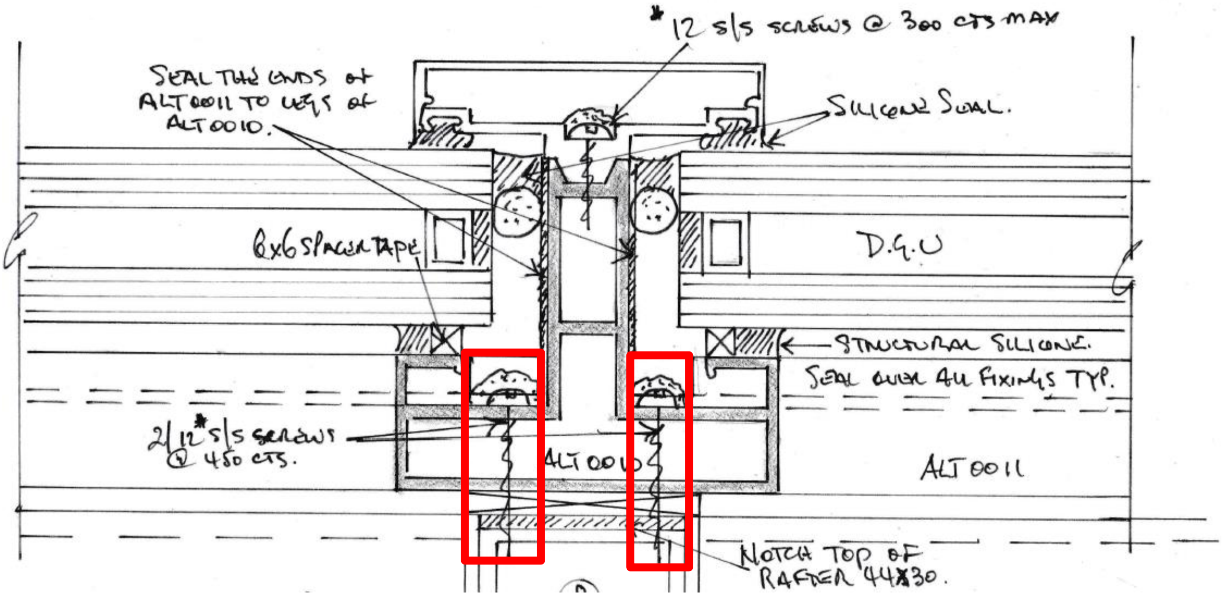
Gauge	Threads	Driver	Drill Capacity
10g	16	P2	0.75 - 3.5mm

CHARACTERISTICS

MECHANICAL PROPERTIES			
Screw	Single Shear (N)	Axial Tensile (N)	Torsional Strength (Nm)
10g-16	5700	12100	8.4

PULL OUT DATA (kN) G450 Steel					
1.0mm	1.2mm	1.5mm	1.9mm	2.4mm	3.2mm
2.3	2.8	3.5	4.3	8.3	9.5

Part-2



Maximum Wind pressure on Glass = 1.088 kN/m^2

UDL on frame due to wind load = 1.088 kN/m^2 x 1.2m

= 1.306 kN/m

C/C distance between two screws = 300mm

Maximum pullout force = 1.306 kN/m x 0.3m

= 0.39 kN

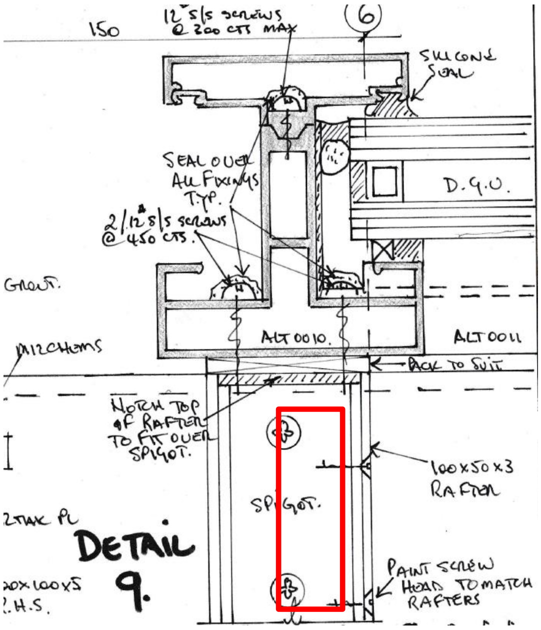
Pullout force on single bolt = 0.195 kN

Pull-out capacity of 8G head self-drilling screw = 2.3 kN to 9.5 kN

Hence, Provided 8G screws are safe in pullout.

9.2. **CONNECTION-2_ 100x50x3 Aluminium RHS to 200x100x5 Steel Member**

<u>Part-1</u>



→ From analysis results, maximum governing end force is

Fx =	9.771	kN
Fy =	0.272	kN
Fz =	0.126	kN

→ **PART-1**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fy^2 + Fz^2)}$ =		0.299767	kN
Number of screws provided (n)		=	2	Nos.
Shear force to be resisted per screw		=	0.149883	kN
Shear capacity of 8G hear self drilling screws		=	5.7	kN

Hence, provided screws is safe in shear

→ **PULLOUT CHECK**

Axial force	=	9.771	kN
Number of screws provided (n)	=	2	Nos.
Axial force to be resisted per screw	=	4.8855	kN
Pullout capacity of 8G hear self drilling screws	=	9.5	kN

Hence, provided screws is safe in Pull-out

→ COMBINE SHEAR & TENSION CHECK

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left[\frac{0.1}{5.7}\right]^2 + \left[\frac{4.9}{9.5}\right]^2 = 0.2652 \quad <= \quad 1$$

Hence, Safe in combine shear and Tension effects

→ CHECK FOR PLY

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* <= \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_F) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	310	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 19.4432 kN $\qquad$ 12.4 kN

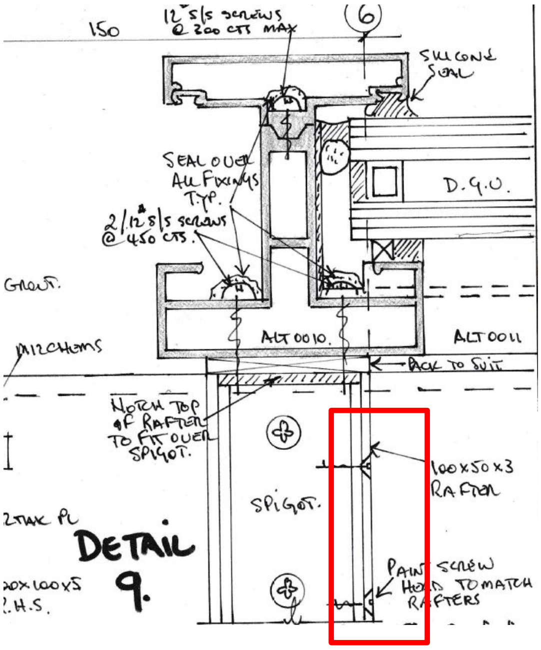
Nominal Bearing capacity of ply V_b = 12.4

$V_b^* <= \phi \times V_b$

5.7 < 11.16

Hence, Ply Safe in Shear

Part-2



→ **PART-2**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fx^2+Fy^2)}$ =		9.774785	kN
Number of screws provided (n)		=	2	Nos.
Shear force to be resisted per screw		=	4.887393	kN
Shear capacity of 8G hear self drilling screws		=	5.7	kN

Hence, provided screws is safe in shear

→ **CHECK FOR PLY**

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* \le \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_F) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	310	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 19.4432 kN $\qquad$ 12.4 kN

Nominal Bearing capacity of ply V_b = 12.4

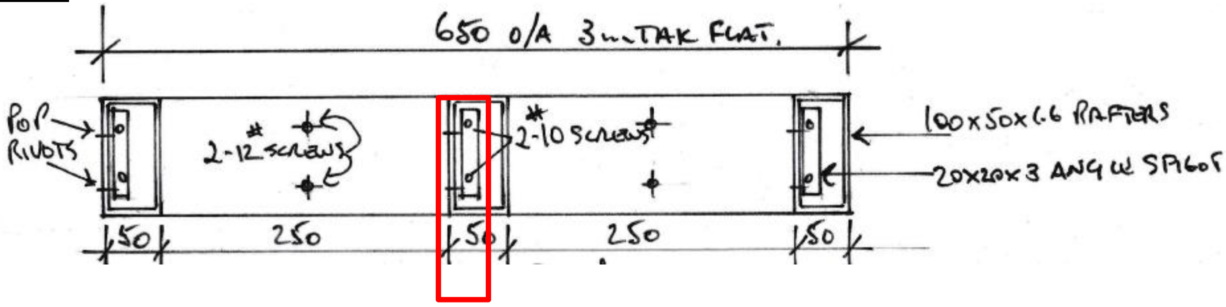
$V_b^* \le \phi \times V_b$

5.7 < 11.16

Hence, Ply Safe in Shear

9.3. CONNECTION-3_ 100x50x4 to 200x100x5

Part-1



CONNECTION 3

→ From analysis results, maximum governing end force is

Fx =	15.589	kN
Fy =	0.333	kN
Fz =	0.161	kN

→ **PART-1**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fy^2 + Fz^2)}$ =	0.369878	kN
Number of screws provided (n)	=	2	Nos.
Shear force to be resisted per screw	=	0.184939	kN
Shear capacity of 8G hear self drilling screws	=	5.7	kN

Hence, provided screws is safe in shear

→ **PULLOUT CHECK**

Axial force	=	15.589	kN
Number of screws provided (n)	=	2	Nos.
Axial force to be resisted per screw	=	7.7945	kN
Pullout capacity of 8G hear self drilling screws	=	9.5	kN

Hence, provided screws is safe in Pull-out

→ **COMBINE SHEAR & TENSION CHECK**

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left[\frac{0.2}{5.7}\right]^2 + \left[\frac{7.8}{9.5}\right]^2 = 0.6742 \quad <= \quad 1$$

Hence, Safe in combine shear and Tension effects

→ **CHECK FOR PLY**

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* \le \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_f) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	310	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\quad\quad$ $V_b = a_e \times t_p \times f_{up}$

= 19.4432 kN $\quad\quad$ 12.4 kN

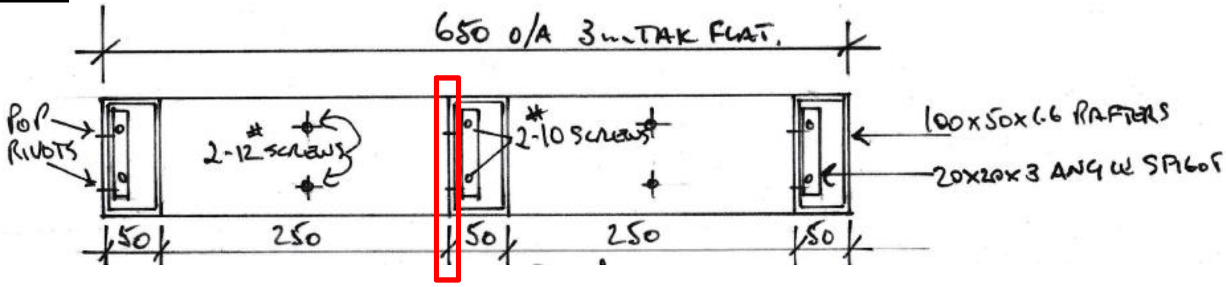
Nominal Bearing capacity of ply V_b = $\quad$ 12.4

$V_b^* \le \phi \times V_b$

5.7 < 11.16

Hence, Ply Safe in Shear

Part-2



→ **PART-2**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fx^2 + Fy^2)}$ =		15.59256	kN
Number of screws provided (n)		=	2	Nos.
Shear force to be resisted per screw		=	7.796278	kN
Shear capacity of 8G hear self drilling screws		=	5.7	kN

Check the shear design

→ **CHECK FOR PLY**

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* <= \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_f) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	310	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 19.4432 kN $\qquad$ 12.4 kN

Nominal Bearing capacity of ply V_b = $\qquad$ 12.4

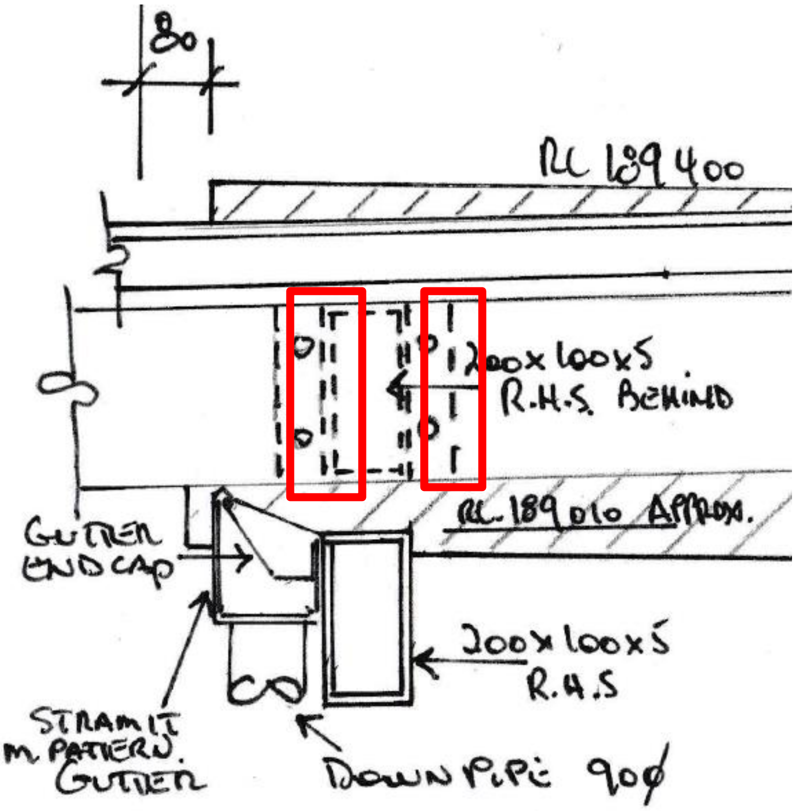
$V_b^* <= \phi \times V_b$

5.7 < 11.16

Hence, Ply Safe in Shear

9.4. **CONNECTION-4_ 100x50x3 Aluminium RHS to 200x100x5**

Part-1



→ From analysis results, maximum governing end force is

Fx =	18.685	kN
Fy =	0.342	kN
Fz =	7.886	kN

→ **PART-1**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fy^2 + Fz^2)}$ =		7.893412	kN
Number of screws provided (n)		=	4	Nos.
Shear force to be resisted per screw		=	1.973353	kN
Shear capacity of 8G hear self drilling screws		=	5.7	kN

Hence, provided screws is safe in shear

→ **PULLOUT CHECK**

Axial force	=	18.685	kN
Number of screws provided (n)	=	4	Nos.
Axial force to be resisted per screw	=	4.67125	kN
Pullout capacity of 8G hear self drilling screws	=	9.5	kN

Hence, provided screws is safe in Pull-out

→ **COMBINE SHEAR & TENSION CHECK**

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left[\frac{2.0}{5.7}\right]^2 + \left[\frac{4.7}{9.5}\right]^2 = \quad 0.3616 \quad <= \quad 1$$

Hence, Safe in combine shear and Tension effects

→ **CHECK FOR PLY**

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* <= \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_f) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	510	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 31.9872 kN $\qquad$ 20.4 kN

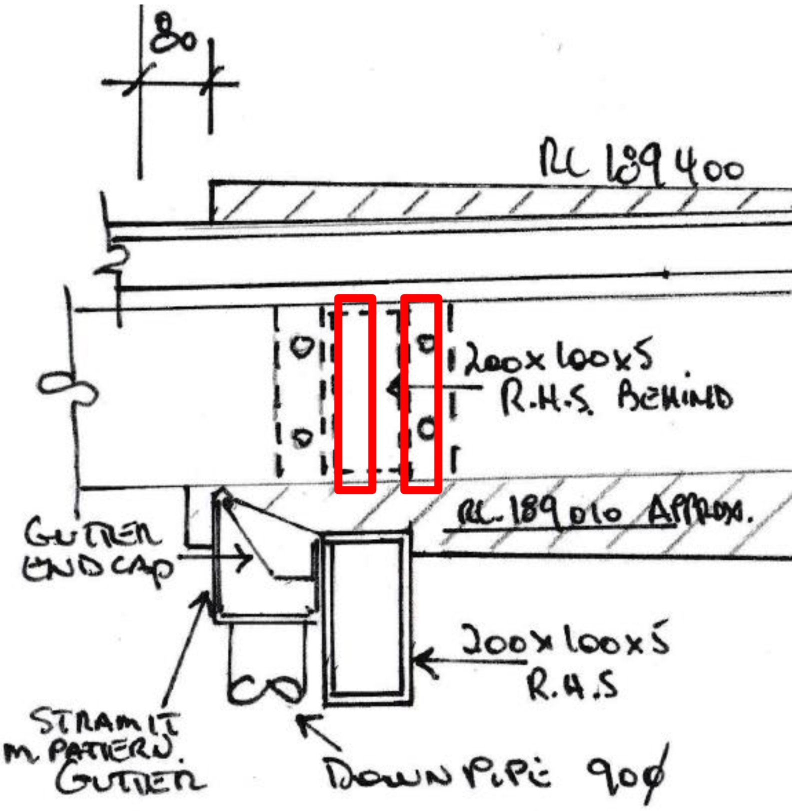
Nominal Bearing capacity of ply V_b = $\qquad$ 20.4

$V_b^* <= \phi \times V_b$

5.7 < 18.36

Hence, Ply Safe in Shear

Part-2



→ **PART-2**

→ **SHEAR CHECK**

Resultant shear force=	$\sqrt{(Fx^2 + Fy^2)}$ =		18.68813	kN
Number of screws provided (n)		=	4	Nos.
Shear force to be resisted per screw		=	4.672032	kN
Shear capacity of 8G hear self drilling screws		=	5.7	kN

Hence, provided screws is safe in shear

→ CHECK FOR PLY

Ply in bearing subjected to design bearing force due to screw in shear

$V_b^* <= \phi \times V_b$

where ϕ = Capacity factor

V_b = Nominal Bearing capacity of ply

Diameter of screw (d_f) =	4.9	mm
Thickness of ply (t_p) =	4	mm
Tensile strength of ply (f_{up}) =	310	MPa
Minimum distance from edge of hole to edge of ply (a_e) =	10	mm

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 19.4432 kN $\qquad$ 12.4 kN

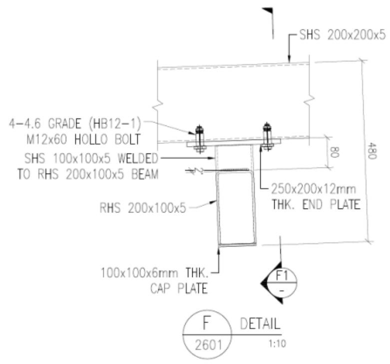
Nominal Bearing capacity of ply V_b = 12.4

$V_b^* <= \phi \times V_b$

5.7 < 11.16

Hence, Ply Safe in Shear

9.5. **CONNECTION-5_ Detail F**

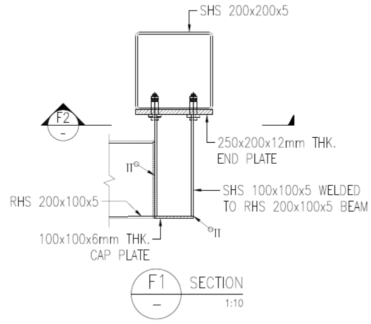


Check for Bolt

Fx = 5.245 kN
Fy = 10.011 kN
Fz = 2.259 kN

Tension force in bolts due to Fy = 10.011 kN/ 4 Nos.
= 2.51 kN

Moment due to Fx = 5.245 kN x 0.15m
= 0.34 kN.m



Tension force on bolt due to moment = 0.34kN.m / 0.1m
= 3.4 kN / 2 Nos.
= 1.7 kN

Moment due to Fz = 2.259 kN x 0.215m
= 0.49 kN.m

Tension force on bolt due to moment = 0.49 kN.m / 0.18m
= 2.73 kN / 2 Nos.
= 1.365 kN

Total tension force on single bolt = 2.51 kN + 1.7 kN + 1.365 kN
= 5.575 kN

Hollo bolt M-12 tension capacity = 10.5 kN > 5.575 kN..Hence Bolt Safe in Tension
Resultant shear force = sqrt(5.245^2 + 2.259^2)
= 5.72 kN

Shear force per bolt = 5.72 kN / 4 Nos.
= 1.43 kN

Hollo bolt M-12 shear capacity = 15 kN > 1.43 kN..Hence Bolt Safe in Shear

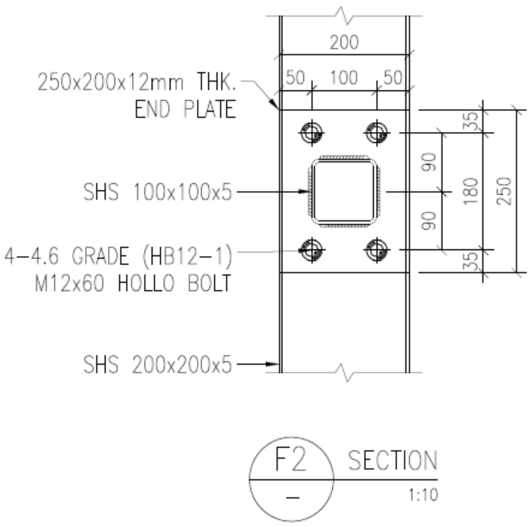
Combined Tension & Shear check,

= (5.575 / 10.5) + (1.43 / 15)
= 0.627 < 1

Hence bolts are safe in combined check.

Check for Plate

My = 0.009 kN.m
Mx = 0.12 kN.m



Flexural capacity of plate in Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((250 x 12^2)/6)
= 1.35 kN.m > 0.34 kN.m ………. Hence OK

Flexural capacity of plate in X-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((200 x 12^2)/6)
= 0.87 kN.m > 0.49 kN.m ………. Hence OK

Check for 6mm Weld

Fx = 5.245 kN Shear
Fy = 10.011 kN Axial
Fz = 2.259 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25} = 233 \text{ N/mm}^2$

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_{YZ} = [Fx + Fz] / [L_w x thickness weld]
= [5.245+2.259] x 10^3 / [400 x 4.242]
= 4.42 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [10.011] x 10^3 / [400 x 4.242]
= 5.9 N/mm^2

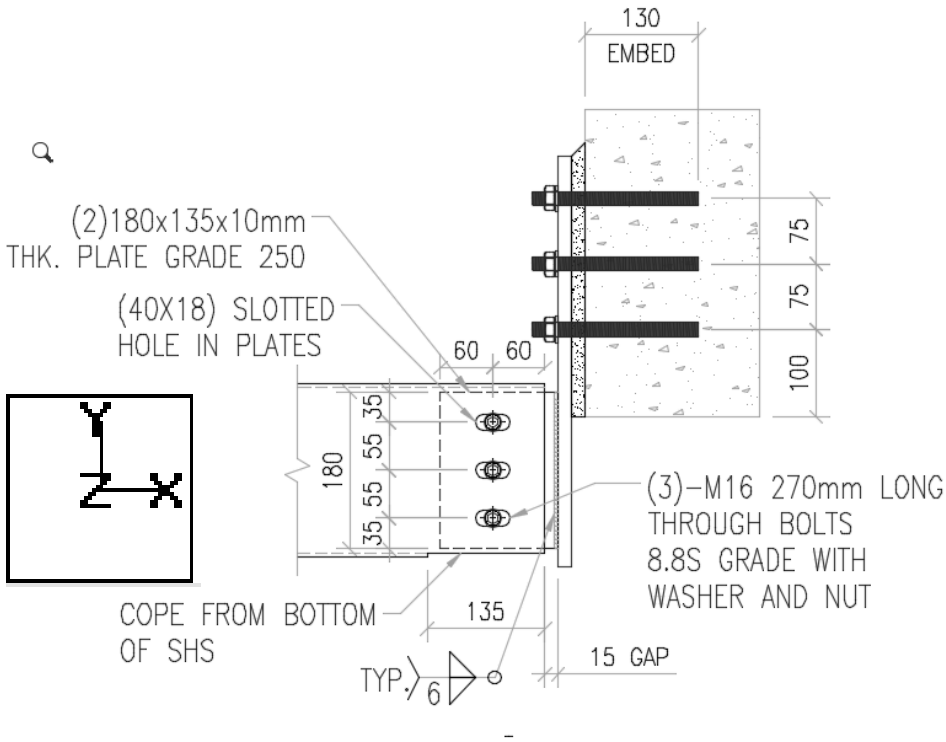
Bending stress in the Weld = Moment / Section Modulus

R_b = $(M_x + M_y)$ / Zx x weld thickness
Here, Z_x = bxd+(d^2/3) for unit weld length
= $(0.009+0.12) \times 10^6$ / [(100x100)+100^2/3 * 4.242]
= 5.34 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [\ (R_x+R_b)^2 + 3(R_{yz})^2\ ]^{1/2}$
$= [\ (5.9+5.34)^2 + 3(4.42)^2\ ]^{1/2}$
$= 13.6\ N/mm^2 < 233\ N/mm^2$ (Hence, OK)

9.6. **CONNECTION-6_ Design for node no. 8**



<u>Check for Plate</u>

For plate, governing reactions are:
Fz = 25.49 kN
Fy = 3.35 kN
Fx = 0 kN

Moment due to Fz,
My = 25.49 kN x 0.075m
= 1.912 kN.m

My will be transferred in each fin plates (places at 80mm c/c) as axial forces,
Axial forces due to Moment My,
Fx = 1.912 / 0.08 = 24 kN

Moment due to Fy
Mz = 3.35/2 kN x 0.075m
= 0.125 kN.m

Considering 2-10mm thk fin plate
Axial Tension capacity of plate in X-direction,
= 0.9 x Ag x Fy
= 0.9 x ((180-3*18) x 10) x 250
= 283 kN > 24 kN………. Hence OK

Flexural capacity of plate about Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((10 x 180^2)/6)
= 12.15 kN.m > 0.125 kN.m ………. Hence OK

Combined axial & bending capacity of plate,
=(24/283) + (0.125/12.15)
= 0.09 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Force on each fin plate connection,
Fz = 25.49/2 = 12.74 kN Shear force
Fy = 3.35/2 = 1.675 kN Shear force
Fx = 24 kN Axial force
Mz = 0.125 kN.m

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Shear stress in the Weld = Load / Effective area of weld

R_{yz} = [Fy+Fz] / [L_w x thickness weld]
= [12.74+1.675] x 10^3 / [180 x 4.242]
= 18.88 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_x = [FX] / [L_w x thickness weld]
= [24] x 10^3 / [180 x 4.242]
= 31.43 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

R_{b2} = (M_z) / Zz x weld thickness
Here, Z_z = (d^2/6) for unit weld length
= (0.125) x 10^6 / [180^2/6 * 4.242]
= 5.45 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

f_e = [$(R_x+R_{b1}+R_{b2})^2 + 3(Ryz)^2$]$^{1/2}$
= [$(31.43+5.45)^2 + 3(18.88)^2$]$^{1/2}$
= 49.29 N/mm^2 < 233 N/mm² (Hence, OK)

Check for 8.8 M16-dia. Bolt

CONNECTION 2

→ From analysis results, maximum govering end force in

Fx =	25.49	kN
Fy =	3.35	kN
Fz =	0	kN

→ **SHEAR CHECK**

shear force=	3.35	kN
Number of bolts provided (n) =	3	Nos.
Shear force to be resisted per bolt (V_f*) =	1.11667	kN

Bolt in shear subjected to Design shear force V_f* shall setisfy

$V_f^* \le \phi V_f$

The nominal shear capacity of bolt is V_f

where ϕ = Capacity factor as per Table 3.4

V_f = Nominal shear capacity of bolt

Nominal shear force $\mathbf{V_f = 0.62 \times f_{uf} \times K_r \times (n_n \times A_c + n_x \times A_o)}$

Diameter of bolt (d_F) =	16	mm
Minor Diameter of bolt as per AS 1275 (d_c) =	13.546	mm
Minimum Tensile strngth of bolt (f_{uf}) =	830	MPa
Nominal shear capacity of a bolt Vf=	514.6	
Reduction factor (K_r) =	1	
Number of shear planes with threads interception (n_n) =	2	
Number of shear planes without threads interception (nx) =	0	
Minor diameter area of bolts as per AS 1275 (A_c) =	144.1	mm^2
Nominal plain shank area of bolt (A_o) =	201.1	mm^2
Nominal shear force (V_f) =	92.0	kN
Capacity factor as per Table 3.4 (ϕ) =	0.8	

$\phi \times V_f$ = 73.6 kN **>** 1.1 kN

Hence, provided bolts is safe in shear

→ **TENSION CHECK**

Bolt in Tension subjected to Design tension force N_{tf}^* shall satisfy

$N_{tf}^* <= \phi \times N_{tf}$

The nominal tensile capacity of bolt is N_{tf}

where ϕ = Capacity factor as per Table 3.4

N_{tf} = Nominal tensile capacity of bolt

Design tension force per bolt (N_{tf}^*)=	8.5	kN		
Nominal tensile force **$N_{tf} = A_s \times f_{uf}$**				
Tensile stress area of bolt (AS 1275) (	157.0	mm^2		
Nominal tensile force (N_{tf}) =	130.3	kN		
$\phi \times N_{tf}$ =	104.2	kN	**>**	8.5 kN

Hence, provided bolts is safe for Tension capacity

→ **BOLT SUBJECTED TO COMBINE SHEAR & TENSION CHECK**

Bolt required to resist both design shear force and Design Tensile force at a same shall (

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

$$\left[\frac{1.1}{73.6}\right]^2 + \left[\frac{8.5}{104.2}\right]^2 = \quad 0.0069 \quad <= \quad 1$$

Hence, Safe in combine shear and Tension effects

→ **CHECK FOR PLY**

Ply in bearing subjected to design bearing force V_b^* due to bolt in shear

$V_b^* <= \phi \times V_b$

where ϕ = Capacity factor as per Table 3.4

V_b = Nominal Bearing capacity of ply

Diameter of bolt (d_f) =	16	mm
Thickness of ply (t_p) =	10	mm
Tensile strength of ply (f_{up}) =	515	MPa
Minimum distance from edge of hole to edge of ply (a_e)	30	mm

a_e

Nominal Bearing capacity of ply V_b

$V_b = 3.2 \times d_f \times t_p \times f_{up}$ $\qquad$ $V_b = a_e \times t_p \times f_{up}$

= 263.68 kN $\qquad$ 154.5 kN

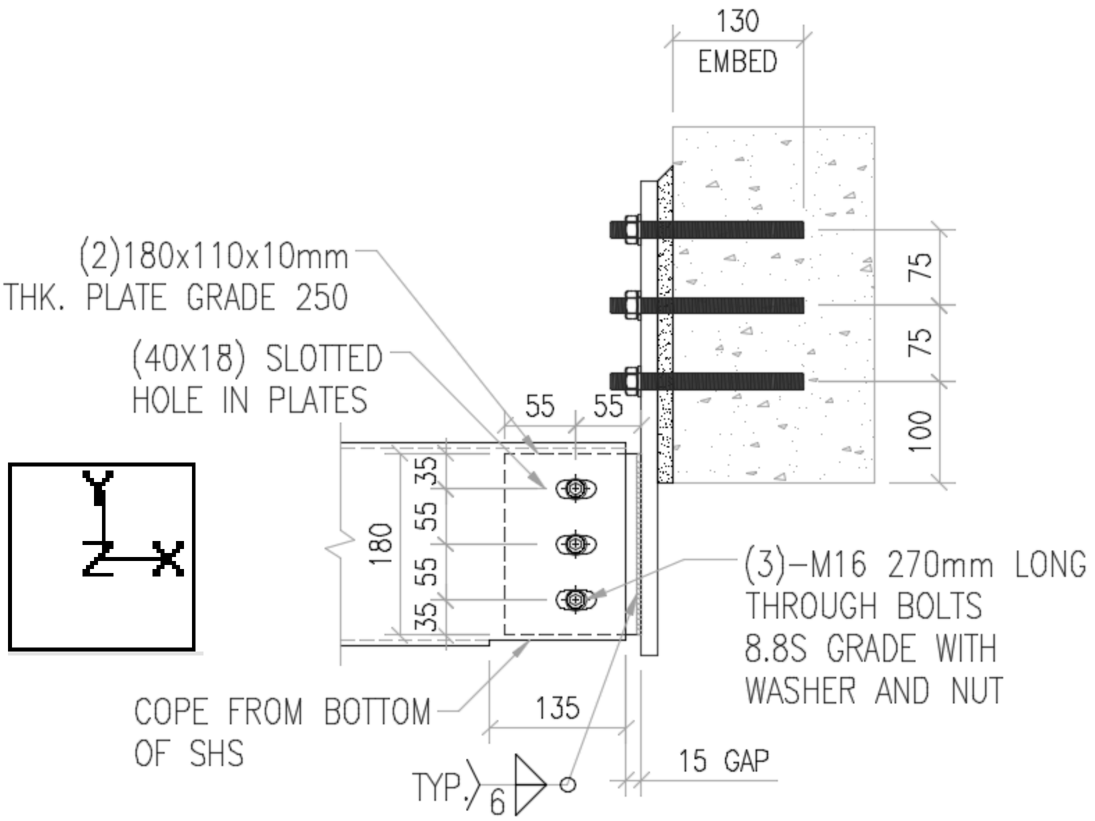
Nominal Bearing capacity of ply V_b = 154.5

$V_b^* <= \phi \times V_b$

73.6 < 139.05

Ply Safe in Shear

9.7. **CONNECTION-7_ Design for node no. 9,16,23,30**



<u>Check for Plate</u>

For plate, governing reactions are (node-9):
Fz = 3.78 kN
Fy = 14.5 kN
Fx = 0 kN

Moment due to Fz,
My = 3.78 kN x 0.070m
= 0.265 kN.m

My will be transferred in each fin plates (places at 80mm c/c) as axial forces,
Axial forces due to Moment My,
Fx = 0.265 / 0.08 = 3.3 kN

Moment due to Fy
Mz = 14.5/2 kN x 0.070m
= 0.50 kN.m

Considering 2-10mm thk fin plate

Axial Tension capacity of plate in X-direction,
= 0.9 x Ag x Fy
= 0.9 x ((180-3*18) x 10) x 250
= 283 kN > 3.3 kN………. Hence OK

Flexural capacity of plate about Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((10 x 180^2)/6)
= 12.15 kN.m > 0.5 kN.m ………. Hence OK

Combined axial & bending capacity of plate,
=(3.3/283) + (0.5/12.15)
= 0.05 < 1 ………. Hence SAFE in combined action

<u>Check for 6mm Weld</u>

Force on each fin plate connection,
Fz = 3.78/2 = 1.89 kN Shear force
Fy = 14.5/2 = 7.25 kN Shear force
Fx = 3.3 kN Axial force
Mz = 0.5 kN.m

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Shear stress in the Weld = Load / Effective area of weld

R_{yz} = [Fy+Fz] / [L_w x thickness weld]
= [7.25+1.89] x 10^3 / [180 x 4.242]
= 11.97 N/mm^2

Direct xial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_x = [FX] / [L_w x thickness weld]
= [3.3] x 10^3 / [180 x 4.242]
= 4.32 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

R_{b2} = (M_z) / Zz x weld thickness
Here, Z_z = (d^2/6) for unit weld length
= (0.5) x 10^6 / [180^2/6 * 4.242]
= 21.8 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [(R_x+R_{b1}+R_{b2})^2 + 3(Ryz)^2]^{1/2}$
$= [(4.32+21.8)^2 + 3(11.97)^2]^{1/2}$
$= 33.35$ N/mm^2 < 233 N/mm² (Hence, OK)

Check for 8.8 M16-dia. Bolt

CONNECTION 2

→ From analysis results, maximum govering end force in

Fx =	3.78	kN	Axial force for bolt
Fy =	14.5	kN	
Fz =	0	kN	

→ **SHEAR CHECK**

shear force=	14.5	kN
Number of bolts provided (n) =	3	Nos.
Shear force to be resisted per bolt (V_f*) =	4.83333	kN

Bolt in shear subjected to Design shear force V_f* shall setisfy

$V_f^* \le \phi V_f$

The nominal shear capacity of bolt is V_f

where ϕ = Capacity factor as per Table 3.4

V_f = Nominal shear capacity of bolt

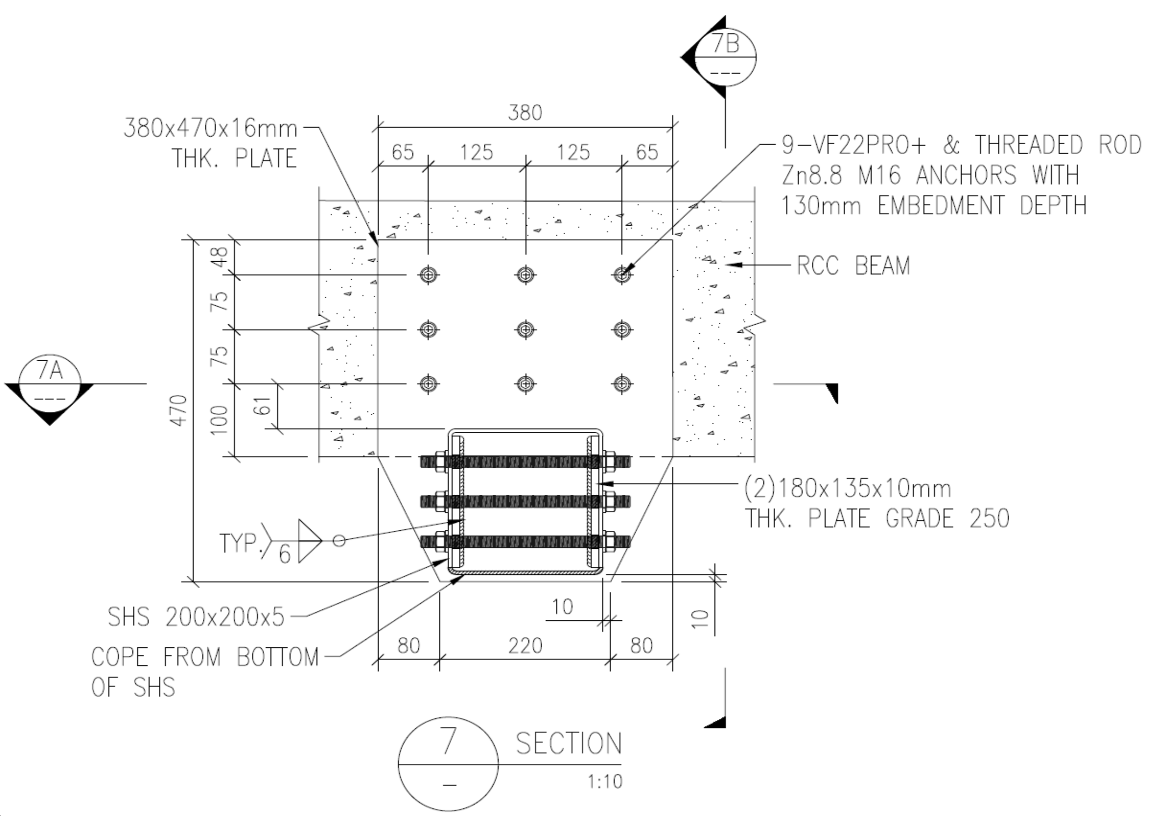
Nominal shear force $\mathbf{V_f = 0.62 \times f_{uf} \times K_r \times (n_n \times A_c + n_x \times A_o)}$

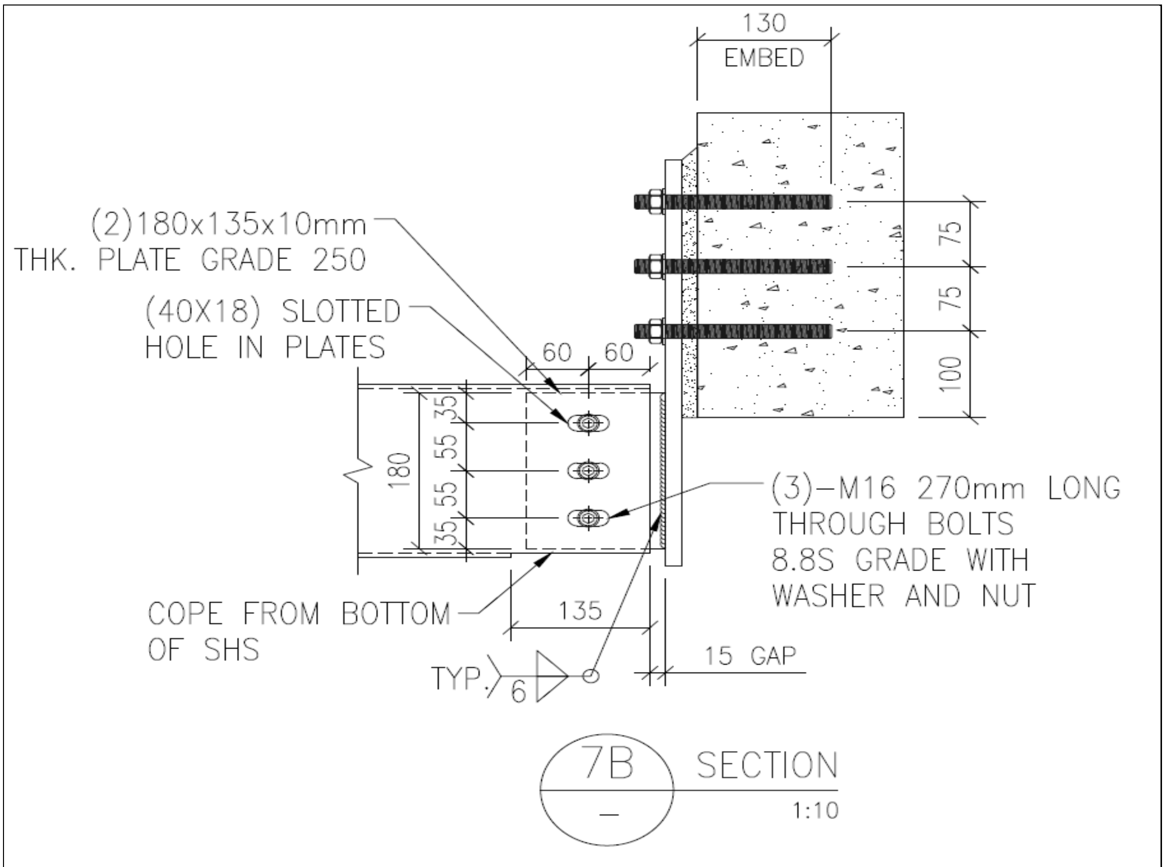
Diameter of bolt (d_F) =	16	mm
Minor Diameter of bolt as per AS 1275 (d_C) =	13.546	mm
Minimum Tensile strngth of bolt (f_{uf}) =	830	MPa
Nominal shear capacity of a bolt Vf=	514.6	
Reduction factor (K_r) =	1	
Number of shear planes with threads interception (n_n) =	2	
Number of shear planes without threads interception (nx) =	0	
Minor diameter area of bolts as per AS 1275 (A_c) =	144.1	mm^2
Nominal plain shank area of bolt (A_o) =	201.1	mm^2
Nominal shear force (V_f) =	92.0	kN
Capacity factor as per Table 3.4 (ϕ) =	0.8	

$\phi \times V_f$ = 73.6 kN **>** 4.8 kN

Hence, provided bolts is safe in shear

9.8. **CONNECTION-8_ Design for node no. 378**





Check for Plate

For plate, governing reactions are:
Fx = 15.758 kN
Fy = 12.66 kN
Fz = 0 kN

Moment due to Fx,
Mz = 15.758 kN x 0.075m
= 1.182 kN.m

Mz will be transferred in each fin plates (places at 180mm c/c) as axial forces,
Axial forces due to Moment My,
Fx = 1.182 / 0.18 = 6.57 kN

Moment due to Fy
Mx = 12.66/2 kN x 0.075m
= 0.475 kN.m

Considering 2-10mm thk fin plate

Axial Tension capacity of plate in Z-direction,
= 0.9 x Ag x Fy
= 0.9 x ((180-3*18) x 10) x 250
= 283 kN > 6.57 kN………. Hence OK

Flexural capacity of plate in y-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((10 x 180^2)/6)
= 12.15 kN.m > 0.475 kN.m ………. Hence OK

Combined axial & bending capacity of plate,
=(6.57/283) + (0.475/12.15)
= 0.06 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Force on each fin plate connection,
Fx = 15.758/2 = 7.88 kN Shear
Fy = 12.66/2 = 6.33 kN Shear
Fz = 6.57 kN Axial force
Mz = 0.475 kN.m

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses fb = $\frac{Mx}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Shear stress in the Weld = Load / Effective area of weld

R_{yz} = [Fy+Fz] / [L_w x thickness weld]

= [6.33+7.88] x 10^3 / [180 x 4.242]

= 18.61 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_z = [Fz] / [L_w x thickness weld]

= [6.57] x 10^3 / [180 x 4.242]

= 8.60 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

R_{b2} = (M_z) / Zx x weld thickness

Here, Z_x = (d^2/6) for unit weld length

= (0.475) x 10^6 / [180^2/6 * 4.242]

= 20.74 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

f_e = [$(R_z+R_{b1}+R_{b2})^2 + 3(R_{yz})^2$]$^{1/2}$

= [$(8.6+20.74)^2 + 3(18.61)^2$]$^{1/2}$

= 43.59 N/mm^2 < 233 N/mm² (Hence, OK)

Check for 8.8 M16-dia. Bolt

CONNECTION 2

→ From analysis results, maximum govering end force in

Fx =	15.758	kN	Axial force for bolt
Fy =	12.66	kN	
Fz =	0	kN	

→ **SHEAR CHECK**

shear force=	12.66	kN
Number of bolts provided (n) =	3	Nos.
Shear force to be resisted per bolt (V_f*) =	4.22	kN

Bolt in shear subjected to Design shear force V_f* shall setisfy

$V_f^* <= \phi V_f$

The nominal shear capacity of bolt is V_f

where ϕ = Capacity factor as per Table 3.4

V_f= Nominal shear capacity of bolt

Nominal shear force **$V_f = 0.62 \times f_{uf} \times K_r \times (n_n \times A_c + n_x \times A_o)$**

Diameter of bolt (d_F) =	16	mm
Minor Diameter of bolt as per AS 1275 (d_C) =	13.55	mm
Minimum Tensile strngth of bolt (f_{uf}) =	830	MPa
Nominal shear capacity of a bolt Vf=	514.6	
Reduction factor (K_r) =	1	
Number of shear planes with threads interception (n_n) =	2	
Number of shear planes without threads interception (nx) =	0	
Minor diameter area of bolts as per AS 1275 (A_c) =	144.1	mm^2
Nominal plain shank area of bolt (A_0) =	201.1	mm^2
Nominal shear force (V_f) =	92.0	kN
Capacity factor as per Table 3.4 (ϕ) =	0.8	

$\phi \times V_f$ = 73.6 kN **>** 4.2 kN

Hence, provided bolts is safe in shear

→ **TENSION CHECK**

Bolt in Tension subjected to Design tension force N_{tf}* shall satisfy

$N_{tf}^* \le \phi \times N_{tf}$

The nominal tensile capacity of bolt is N_{tf}

where ϕ = Capacity factor as per Table 3.4

N_{tf} = Nominal tensile capacity of bolt

Design tension force per bolt (N_{tf}*)=	5.3	kN		
Nominal tensile force **$N_{tf} = A_s \times f_{uf}$**				
Tensile stress area of bolt (AS 1275) (A_S)=	157.0	mm^2		
Nominal tensile force (N_{tf}) =	130.3	kN		
$\phi \times N_{tf}$ =	104.2	kN	**>**	5.3 kN

Hence, provided bolts is safe for Tension capacity

→ **BOLT SUBJECTED TO COMBINE SHEAR & TENSION CHECK**

Bolt required to resist both design shear force and Design Tensile force at a same shall confirm

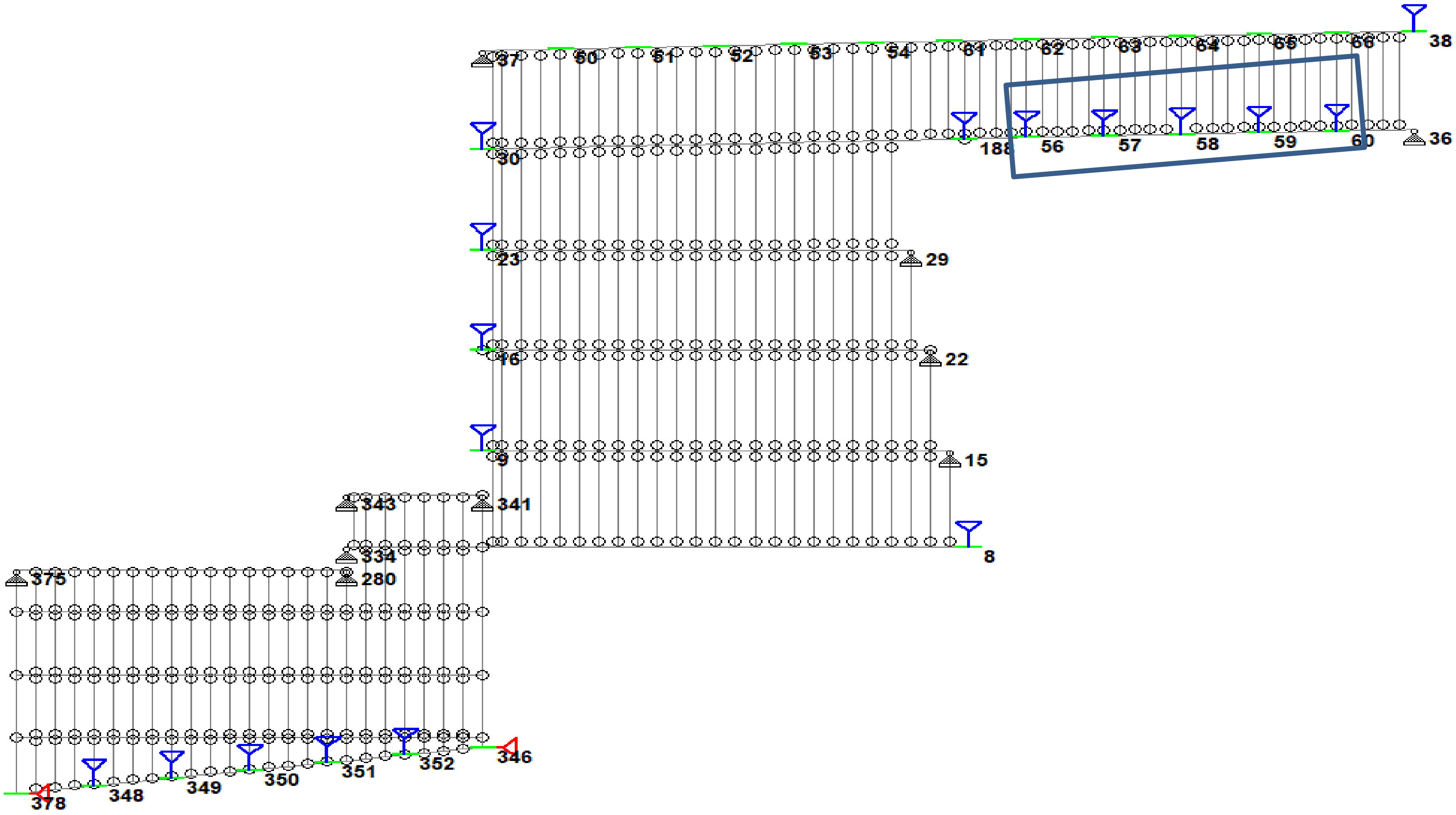
$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \le 1.0$$

$$\left[\frac{4.2}{73.6}\right]^2 + \left[\frac{5.3}{104.2}\right]^2 = 0.0058 \quad <= \quad 1$$

Hence, Safe in combine shear and Tension effects

9.9. **End plate and Embed design-Type-1**

Below image show location of End plate and Embed design Type-1.



Design of base plate

Input data:

Factored vertical force (P) =	6.67	kN

Bolt data:-

No. of bolts =	2	mm
Bolt dia. =	12	mm
Hole dia. (d_h) =	14	mm
Provided edge distance =	40	mm
Provided bolt spacing =	170	mm

Base plate data:-

Yield strength of base plate (f_y) =	250	MPa
Base plate width =	250	mm
Base plate length =	110	mm
Base plate thickness =	12	mm
Max projection from column face =	75	mm

Base plate design:-

Pressure = P/Base plate area = 6.67/ (0.25 x 0.11)

= 242.55 kN/m^2

Maximum moment on base plate = 242.55 x 0.075^2 /2

= 0.68 kN.m/m length of base plate

$S = M/f_y$ = 0.68 x 10^6 /250

= 2720 mm^3

calculate thickness considering unit length of plate (b = 1000 mm)

Thickness of base $plate_{(req)}$ = sqrt((4 x s)/b)

= (4 x 2720) / 1000)^0.5

3.3 mm ≤ 12 mm

(Hence OK)

Check for 6mm Weld

Fx = 6.67 kN Axial
Fy = 4.267 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25} = 233\ \text{N/mm}^2$

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $= \sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_Y = [Fy] / [L_w x thickness weld]
= [4.267] x 10^3 / [400 x 4.242]
= 2.52 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [6.67] x 10^3 / [400 x 4.242]
= 3.93 N/mm^2

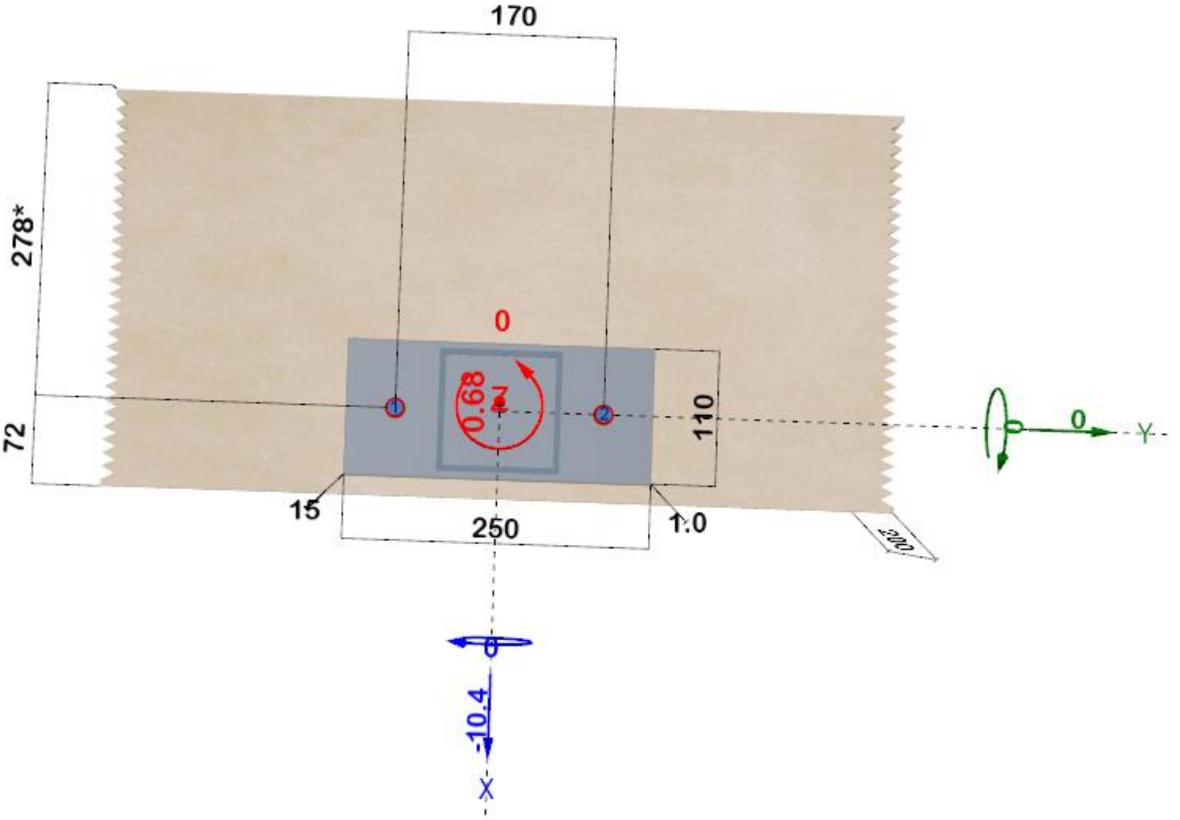
Bending stress in the Weld = Moment / Section Modulus

R_b = (M_x) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (0.57) x 10^6 / [(100+100)^3/6 * 4.242]
= 0.11 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [\ (R_x+R_b)^2 + 3(R_{yz})^2\ ]^{1/2}$
$= [\ (3.93+0.11)^2 + 3(2.52)^2\ ]^{1/2}$
= 5.95 N/mm^2 < 233 N/mm² (Hence, OK)

Check for Anchor



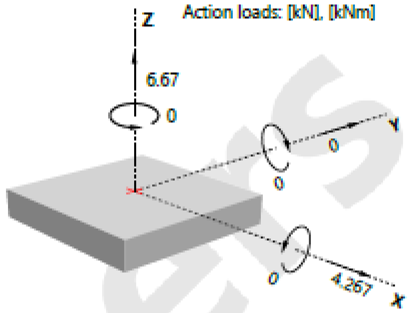
Node number 56 reactions for anchor design is:
Fx = 10.4 kN
Fy = 0.00 kN
Fz = 0.68 kN

Company:		E-mail:	
Designer:		Phone:	
Address:		Fax:	
Project:		Date:	4/13/2024
Comments:		Page:	1 / 7

1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Galv 8.8 M12
  Injection anchor Vinylester
  Hot-dip/Mechanically galvanized
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 100 mm
- Drilled hole Φ x h_0 = 14.0 x 100 mm



Base material:

- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f'_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

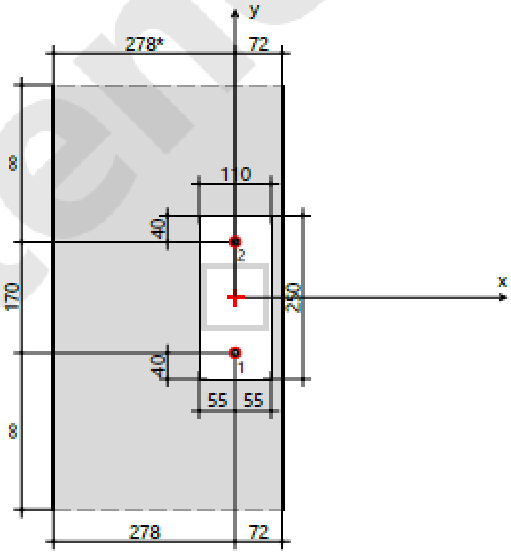
- Predominantly static and quasi-static design loads, α_{sus}=0.6

Installation:

- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

Base plate:

- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, f_{yd}= $\phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 1.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 110 x 250 mm

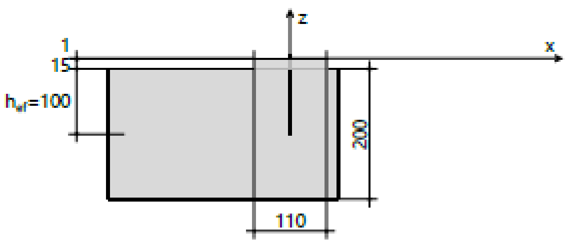


Profile:

- Square Hollow Section: 100x5.0 SHS
  H x W x T x FT [mm]: 100 x 100 x 5.0 x 0.0
  Action point [mm]: [0, 0]
  Rotation counterclockwise: 0°

Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	L-y
1	0.0	-85.0		
2	0.0	85.0		



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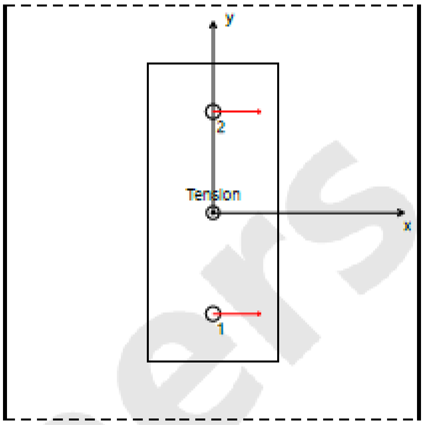
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	3.335	2.134	2.134	0.000
2	3.335	2.134	2.134	0.000



Maximum concrete compressive strain [‰]: 0.0000
Maximum concrete compressive stress: 0.00 [N/mm²]
Resultant tension force in (x/y=0.0/0.0): 6.670 [kN]
Resultant compression force in (x/y=0.0/0.0): 0.000 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \leq f_{yd}$

b) $N^h_r \approx N^h_e$

N^h_r: highest anchor tension force on flexurally rigid base plate

N^h_e: highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	1,2	3.335	44.667	7.5	√
Combined failure	1,2	6.670	16.045	**41.6**	√
Concrete cone failure	1,2	6.670	26.473	25.2	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\quad \beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
67.0	0.667	44.667	3.335	0.075

Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{ec,Np} \cdot \psi_{re,Np}$ $N^0_{Rk,p} = \psi_{sus} \cdot \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\psi_{A,Np} = A_{p,N}/A^0_{p,N}$ $N_{Rd,Np} = N_{Rk,Np} \cdot \phi_{p,N}$

$s_{cr,Np} = 7.3 \cdot d \cdot (\psi_{sus} \cdot \tau_{Rk,ucr})^{0.5} \leq 3 \cdot l_b$ $\psi_{g,Np} = \psi^0_{g,Np} - (s_m / s_{cr,Np})^{0.5} \cdot (\psi^0_{g,Np} - 1) \geq 1.0$

$\psi^0_{g,Np} = n^{0.5} - (n^{0.5} - 1) \cdot (\tau_{Rk} / \tau_{Rk,c})^{1.5} \geq 1.0$ $\tau_{Rk,c} = k_3 \cdot (h_{ef} \cdot f'_c)^{0.5} / (\pi \cdot d)$ $\psi^0_{sus} = 0.73$ $\alpha_{sus} = 0.6$ $\psi_{sus} = 1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.5	1.231	12.0	7.7	40	100.0	270.0	135.0	100.0	0.556	12.918

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
25.524	91080	72901	1.249	0.860	72.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
2	1.257	170.0	1.053	1.0	0.0	0.0	1.000	1.000	1.000	28.880	16.045	6.670	0.416

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $N^0_{Rk,c} = k_1 \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $N_{Rd,c} = N_{Rk,c} \cdot \phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
48.699	104340	90000	1.159	7.7	0.556	100.0	300.0	150.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.844	1.0	0.0	0.0	1.0	1.0	1.0	1.0	47.651	26.473	6.670	0.252

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	1,2	2.134	7.231	29.5	√
Pry-out	1,2	4.267	29.045	14.7	√
Concrete edge failure (x+)	1,2	4.267	10.915	39.1	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $M_{Rk,s}=M^0_{Rk,s}(1-|N^*|/N_{Rd,s})$ $V_{Rd,s}=V_{Rk,s} \cdot \phi_{s,V}$ $\beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s} \cdot \phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
105.0	67.0	0.667	44.667	2.0	15.5	6.0	21.5	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s}(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
3.335	97.160	9.038	7.231	2.134	0.295

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $\quad N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\quad V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $\quad V_{Rd,cp} =V_{Rk,cp} \cdot \phi_{cp,V}$

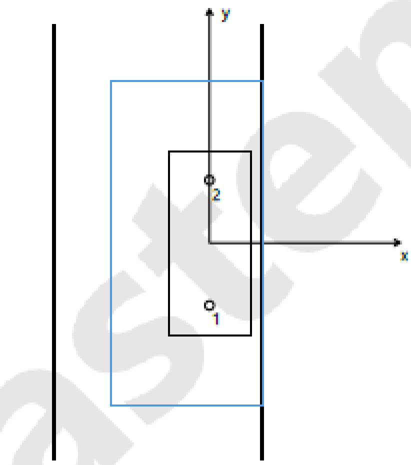
For stand-off installation (overturning moment): $V_{Rd,cp} =V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\quad \alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.754$ $\quad h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
100.0	9.5	270.0	135.0	12.0	100.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
25.524	91080	72901	1.249	1.257	170.0	1.053	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V* [kN]	$\beta_{V,cp}$
0.86	1.0	0.0	0.0	1.0	1.0	1.0	28.880	57.761	29.045	4.267	0.147

Related area for calculation of pry-out failure $A_{p,N}$:



Remark: Edge distance (-x) is not to scale.

Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $\quad V^0_{Rk,c} =k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\quad \psi_{A,V} =A_{c,V}/A^0_{c,V}$ $\quad V_{Rd,c} =V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\quad \alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\quad \beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} =V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\quad \alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.754$ $\quad h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
100.0	1.7	40	0.667	72.0	-	0.118	0.070	12.146	1.000	12.0	100.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V* [kN]	$\beta_{V,c}$
41688	23328	1.787	1.000	1.000	0.0	1.000	1.000	21.705	10.915	4.267	0.391

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \le 1.0$	-	not applicable
Concrete	1,2	0.416	0.391	$\beta^{1.5}_N + \beta^{1.5}_V \le 1.0$	51.2	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0.075	0.416	0.252	0.000	0.295	0.147	0.391	0.416	0.391	0.512	-
2	0.075	0.416	0.252	0.000	0.295	0.147	0.391	0.416	0.391	0.512	-

$\beta_{N,c,max,E}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,E}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,E}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,E}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N{}^0 = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N{}^\infty = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k{}^h = V^{*h} / 1.4$

Short-term displacement: $\delta_V{}^0 = V_k{}^h \cdot \delta_{V0}$

Long-term displacement: $\delta_V{}^\infty = V_k{}^h \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	$\delta_N{}^0$ [mm]	$\delta_N{}^\infty$ [mm]
3.335	0.885	0.090	0.320	**0.057**	**0.202**

V^{*h} [kN]	$V_k{}^h$ [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	$\delta_V{}^0$ [mm]	$\delta_V{}^\infty$ [mm]
2.134	1.524	0.200	0.300	**0.305**	**0.457**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

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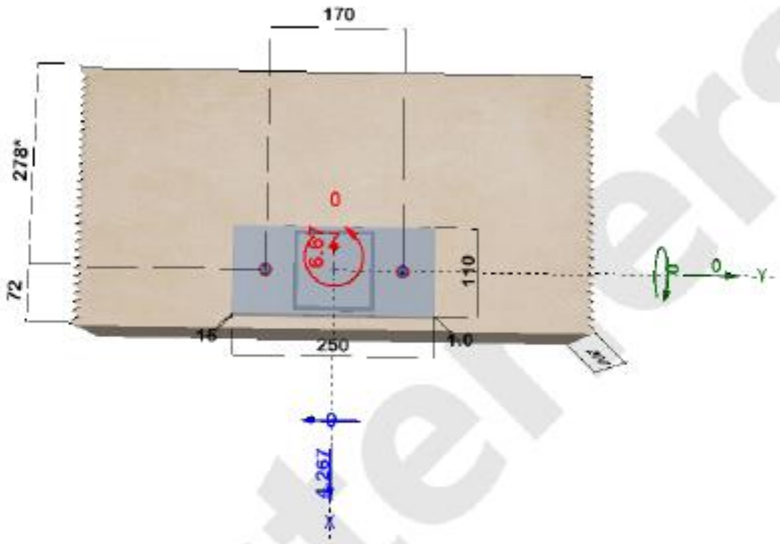
Anchorage figure in 3D:



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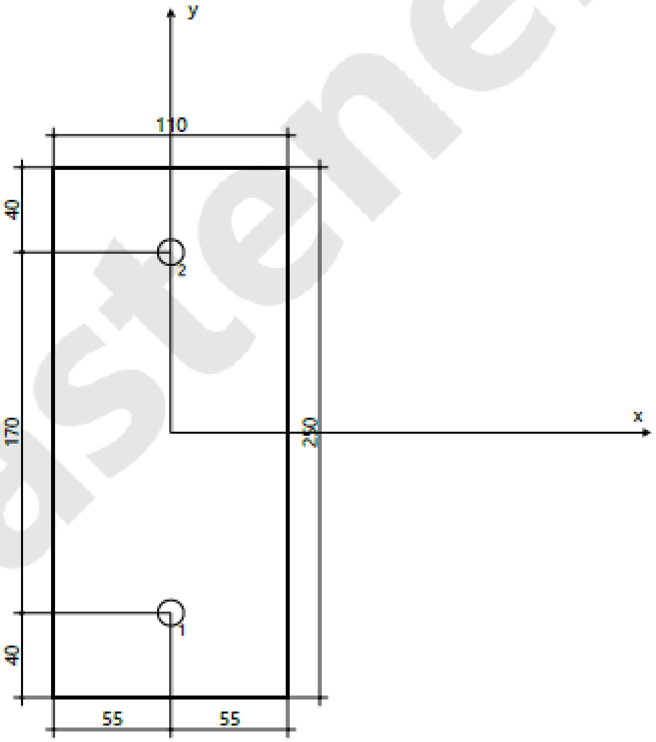
Anchor: **VF22PRO+ & Threaded Rod Galv 8.8 M12**

Drilled hole:	$d_0 \times h_0$ = 14 x 100 mm
Embedment depth:	h_{nom} = 100 mm
Effective anchorage depth:	h_{ef} = 100 mm
Installation torque:	T_{inst} = 40 Nm

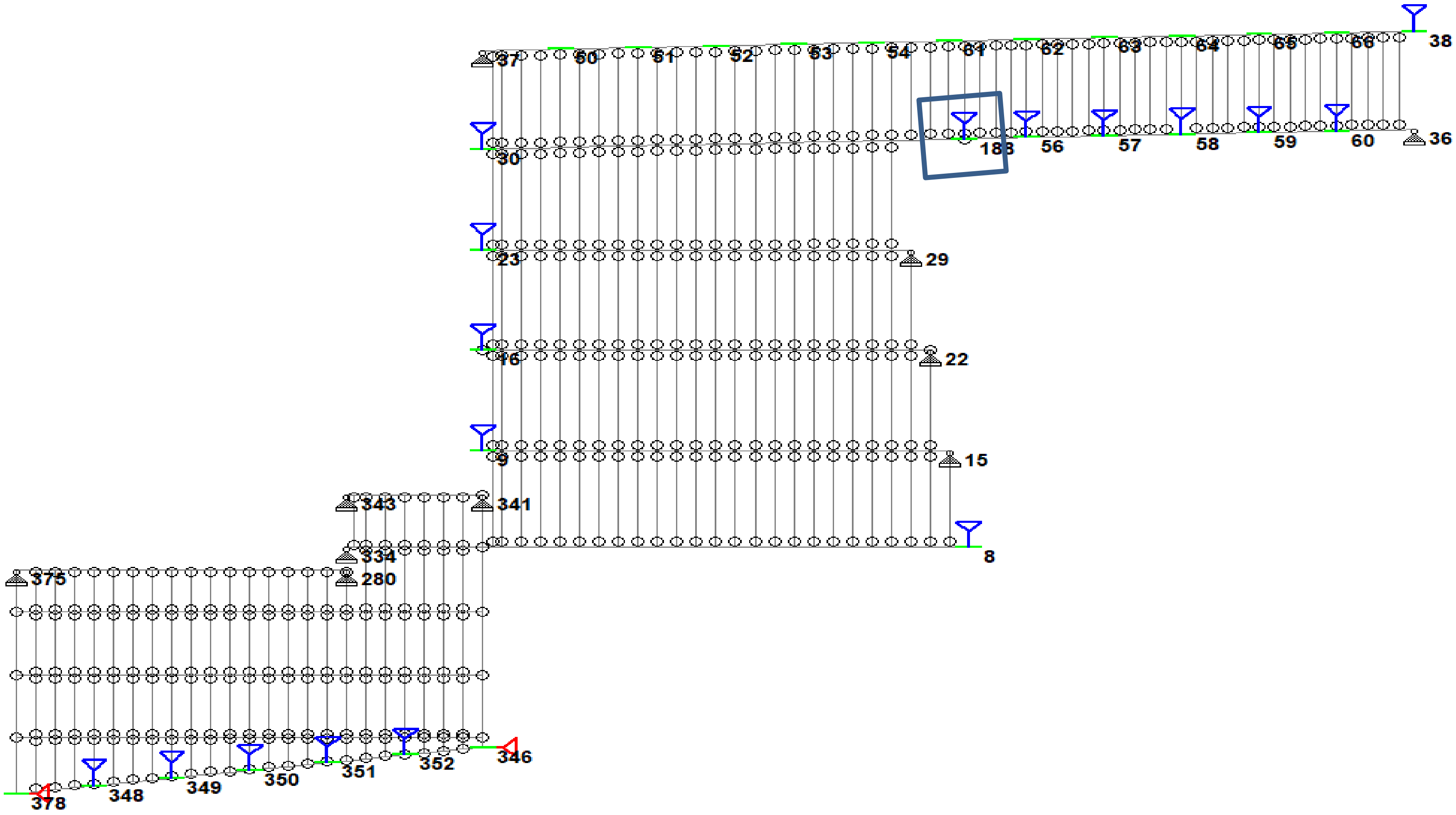
Base plate: **G250**

Thickness:	t = 1 mm
Clearance hole:	d_f = 14 mm



9.10. **<u>End plate and Embed design-Type-2</u>**

Below image show location of End plate and Embed design Type-2.



Design of base plate

Input data:

Factored vertical force (P) =	6.67	kN

Bolt data:-

No. of bolts =	2	mm
Bolt dia. =	12	mm
Hole dia. (d_h) =	14	mm
Provided edge distance =	40	mm
Provided bolt spacing =	170	mm

Base plate data:-

Yield strength of base plate (f_y) =	250	MPa
Base plate width =	250	mm
Base plate length =	110	mm
Base plate thickness =	12	mm
Max projection from column face =	75	mm

Base plate design:-

Pressure = P/Base plate area = 6.67/ (0.25 x 0.11)

= 242.55 kN/m^2

Maximum moment on base plate = 242.55 x 0.075^2 /2

= 0.68 kN.m/m length of base plate

$S = M/f_y$ = 0.68 x 10^6 /250

= 2720 mm^3

calculate thickness considering unit length of plate (b = 1000 mm)

Thickness of base plate$_{(req)}$ = sqrt((4 x s)/b)

= (4 x 2720) / 1000)^0.5

3.3 mm ≤ 12 mm

(Hence OK)

Check for 6mm Weld

Fx = 6.67 kN Axial
Fy = 4.267 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25} = 233\ \text{N/mm}^2$

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_Y = [Fy] / [L_w x thickness weld]
= [4.267] x 10^3 / [400 x 4.242]
= 2.52 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [6.67] x 10^3 / [400 x 4.242]
= 3.93 N/mm^2

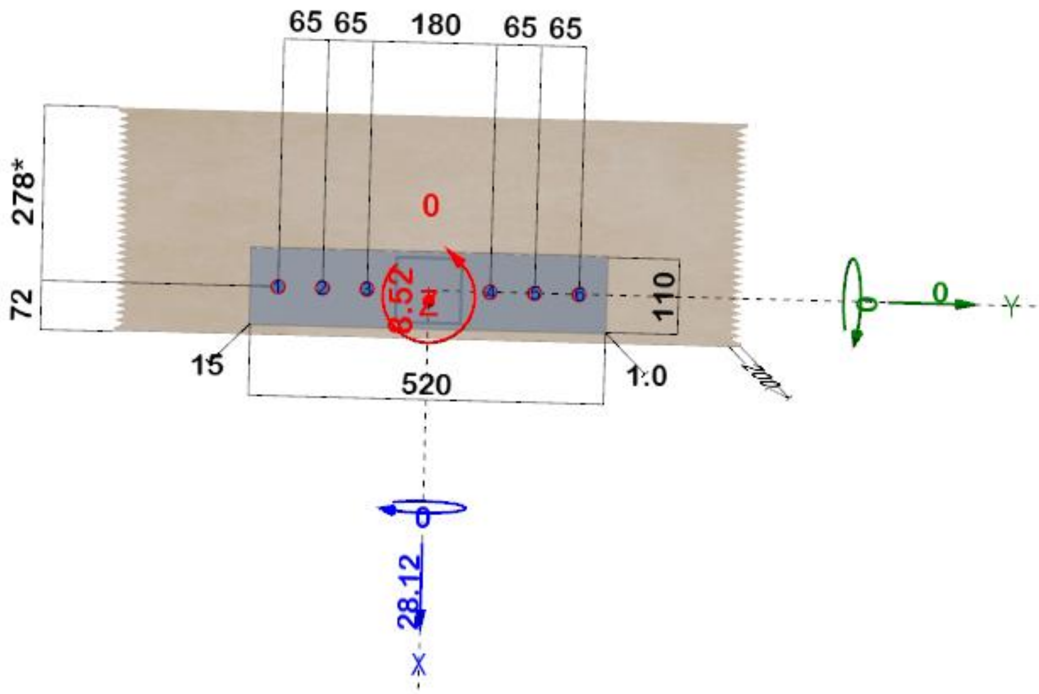
Bending stress in the Weld = Moment / Section Modulus

R_b = (M_x) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (0.57) x 10^6 / [(100+100)^3/6 * 4.242]
= 0.11 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [\ (R_x+R_b)^2 + 3(R_{yz})^2\ ]^{1/2}$
$= [\ (3.93+0.11)^2 + 3(2.52)^2\ ]^{1/2}$
= 5.95 N/mm^2 < 233 N/mm^2 (Hence, OK)

Check for Anchor



Node number 188 reactions for anchor design is:
Fx = 28.12 kN
Fy = 0.00 kN
Fz = 8.52 kN

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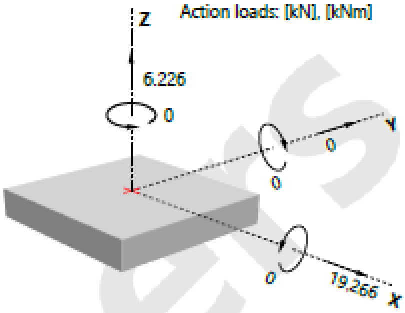
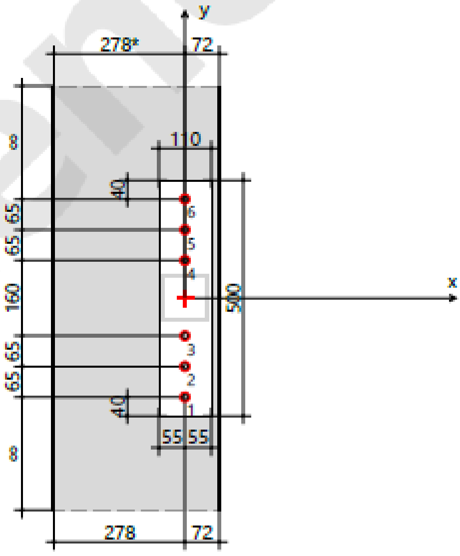
1. Input Data

Selected anchors:
- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M16
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 130 mm
- Drilled hole Φ x h_0 = 18.0 x 130 mm



Base material:
- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f'_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:
- Predominantly static and quasi-static design loads, α_{sus}=0.6

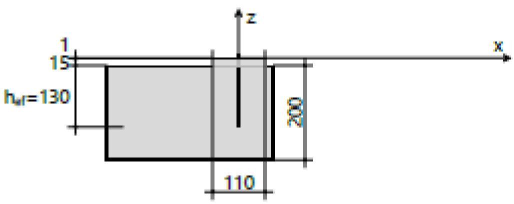


Installation:
- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

Base plate:
- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, f_{yd}= $\phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 1.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 110 x 500 mm

Profile:
- Square Hollow Section: 100x5.0 SHS
  H x W x T x FT [mm]: 100 x 100 x 5.0 x 0.0
  Action point [mm]: [0, 0]
  Rotation counterclockwise: 0°

Coordinates of anchors [mm]:



			Slotted hole	
No.	x	y	L-x	L-y
1	0.0	-210.0		
2	0.0	-145.0		
3	0.0	-80.0		
4	0.0	80.0		
5	0.0	145.0		
6	0.0	210.0		

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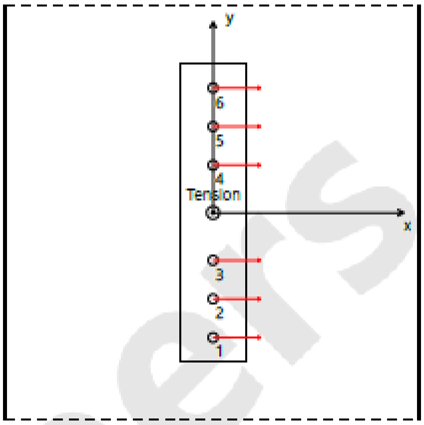
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	1.038	3.211	3.211	0.000
2	1.038	3.211	3.211	0.000
3	1.038	3.211	3.211	0.000
4	1.038	3.211	3.211	0.000
5	1.038	3.211	3.211	0.000
6	1.038	3.211	3.211	0.000



Maximum concrete compressive strain [‰]: 0.0000
Maximum concrete compressive stress: 0.00 [N/mm²]
Resultant tension force in (x/y=0.0/0.0): 6.226 [kN]
Resultant compression force in (x/y=0.0/0.0): 0.000 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \le f_{yd}$
b) $N^h_r \approx N^h_e$
N^h_r : highest anchor tension force on flexurally rigid base plate
N^h_e : highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	1,2,3,4,5,6	1.038	84.000	1.2	√
Combined failure	1,2,3,4,5,6	6.226	31.366	**19.8**	√
Concrete cone failure	1,2,3,4,5,6	6.226	46.230	13.5	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
126.0	0.667	84.000	1.038	0.012

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Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{ec,Np} \cdot \psi_{re,Np}$ $N^0_{Rk,p} = \psi_{sus} \cdot \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\psi_{A,Np} = A_{p,N}/A^0_{p,N}$ $N_{Rd,Np} = N_{Rk,Np} \cdot \phi_{p,N}$

$s_{cr,Np} = 7.3 \cdot d \cdot (\psi_{sus} \cdot \tau_{Rk,ucr})^{0.5} \leq 3 \cdot l_b$ $\psi_{g,Np} = \psi^0_{g,Np} - (s_m / s_{cr,Np})^{0.5} \cdot (\psi^0_{g,Np} - 1) \geq 1.0$

$\psi^0_{g,Np} = n^{0.5} - (n^{0.5} - 1) \cdot (\tau_{Rk} / \tau_{Rk,c})^{1.5} \geq 1.0$ $\tau_{Rk,c} = k_3 \cdot (h_{ef} \cdot f'_c)^{0.5} / (\pi \cdot d)$ $\psi^0_{sus} = 0.73$ $\alpha_{sus} = 0.6$ $\psi_{sus} = 1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.0	1.231	16.0	7.7	40	130.0	350.4	175.2	130.0	0.556	11.046

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
44.242	190314	122780	1.550	0.823	72.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
6	1.754	420.0	1.0	1.0	0.0	0.0	1.000	1.000	1.000	56.458	31.366	6.226	0.198

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $N^0_{Rk,c} = k_1 \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $N_{Rd,c} = N_{Rk,c} \cdot \phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
72.183	216270	152100	1.422	7.7	0.556	130.0	390.0	195.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.811	1.0	0.0	0.0	1.0	1.0	1.0	1.0	83.215	46.230	6.226	0.135

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	1,2,3,4,5,6	3.211	17.887	18.0	√
Pry-out	1,2,3,4,5,6	19.266	59.412	32.4	√
Concrete edge failure (x+)	1,2,3,4,5,6	19.266	21.177	**91.0**	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $M_{Rk,s}=M^0_{Rk,s}(1-|N^*|/N_{Rd,s})$ $V_{Rd,s}=V_{Rk,s} \cdot \phi_{s,V}$ $\beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s} \cdot \phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
266.0	126.0	0.667	84.000	2.0	15.5	8.0	23.5	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s}(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
1.038	262.714	22.359	17.887	3.211	0.180

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $V_{Rd,cp}=V_{Rk,cp} \cdot \phi_{cp,V}$

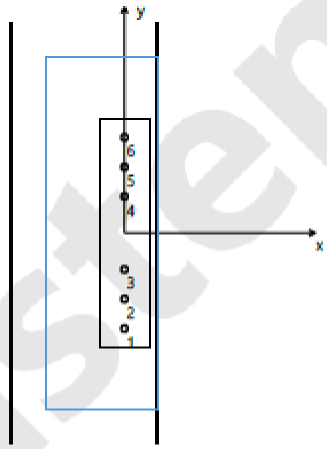
For stand-off installation (overturning moment): $V_{Rd,cp}=V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\alpha_h=(h_h - a_3) / (e_1 + h_h)= 0.789$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
130.0	9.0	350.4	175.2	16.0	130.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
44.242	190314	122780	1.55	1.754	420.0	1.0	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V^* [kN]	$\beta_{V,cp}$
0.823	1.0	0.0	0.0	1.0	1.0	1.0	56.458	112.916	59.412	19.266	0.324

Related area for calculation of pry-out failure $A_{p,N}$:



Remark: Edge distance (-x) is not to scale.

Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c}=k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V}=A_{c,V}/A^0_{c,V}$ $V_{Rd,c}=V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c}=V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\alpha_h=(h_h - a_3) / (e_1 + h_h)= 0.789$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
130.0	1.7	40	0.667	72.0	-	0.134	0.074	13.670	1.000	16.0	130.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V^* [kN]	$\beta_{V,c}$
68688	23328	2.944	1.000	1.000	0.0	1.000	1.000	40.249	21.177	19.266	0.910

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \leq 1.0$	-	not applicable
Concrete	1,2,3,4,5,6	0.198	0.910	$\beta_N + \beta_V \leq 1.2$	92.4	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,i}$	$\beta_{V,c,max,i}$	$\beta_{combi,c,i}$	$\beta_{combi,s,i}$
1	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-
2	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-
3	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-
4	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-
5	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-
6	0.012	0.198	0.135	0.000	0.180	0.324	0.910	0.198	0.910	0.924	-

$\beta_{N,c,max,i}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,i}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,i}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,i}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^0 = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^\infty = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^h = V^{*h} / 1.4$

Short-term displacement: $\delta_V^0 = V_k^h \cdot \delta_{V0}$

Long-term displacement: $\delta_V^\infty = V_k^h \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	δ_N^0 [mm]	δ_N^∞ [mm]
1.038	0.159	0.050	0.180	**0.006**	**0.020**

V^{*h} [kN]	V_k^h [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	δ_V^0 [mm]	δ_V^∞ [mm]
3.211	2.294	0.110	0.170	**0.252**	**0.390**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

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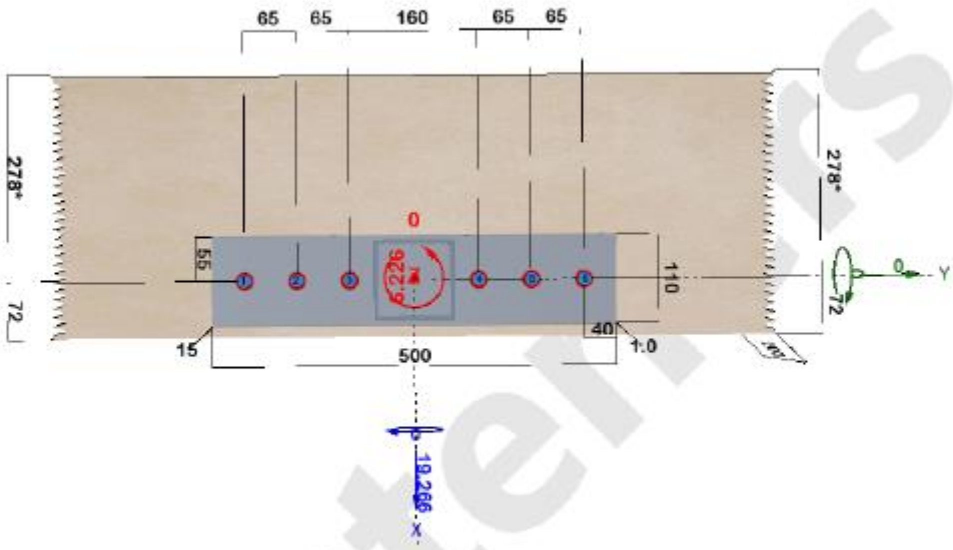
Anchorage figure in 3D:



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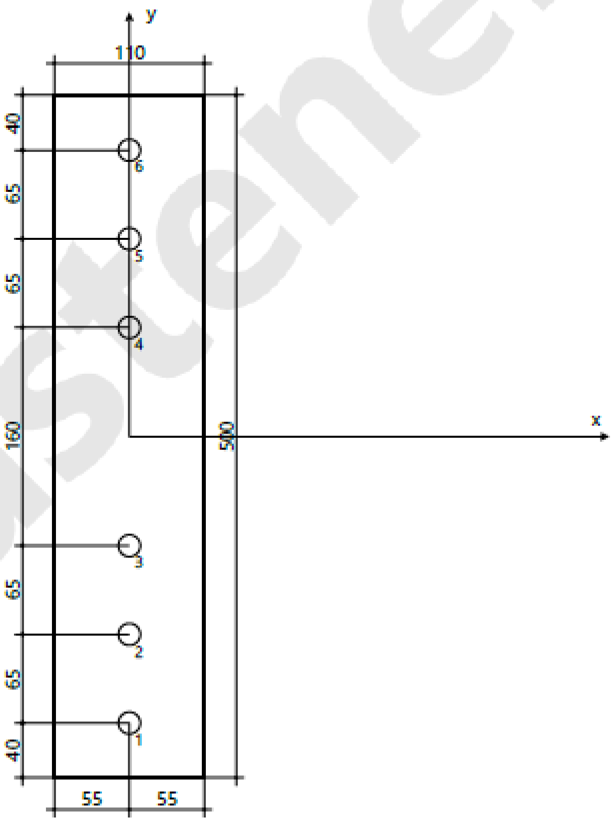
Anchor: **VF22PRO+ & Threaded Rod Zn 8.8 M16**

Drilled hole: d_0 x h_0 = 18 x 130 mm
Embedment depth: h_{nom} = 130 mm
Effective anchorage depth: h_{ef} = 130 mm
Installation torque: T_{inst} = 80 Nm

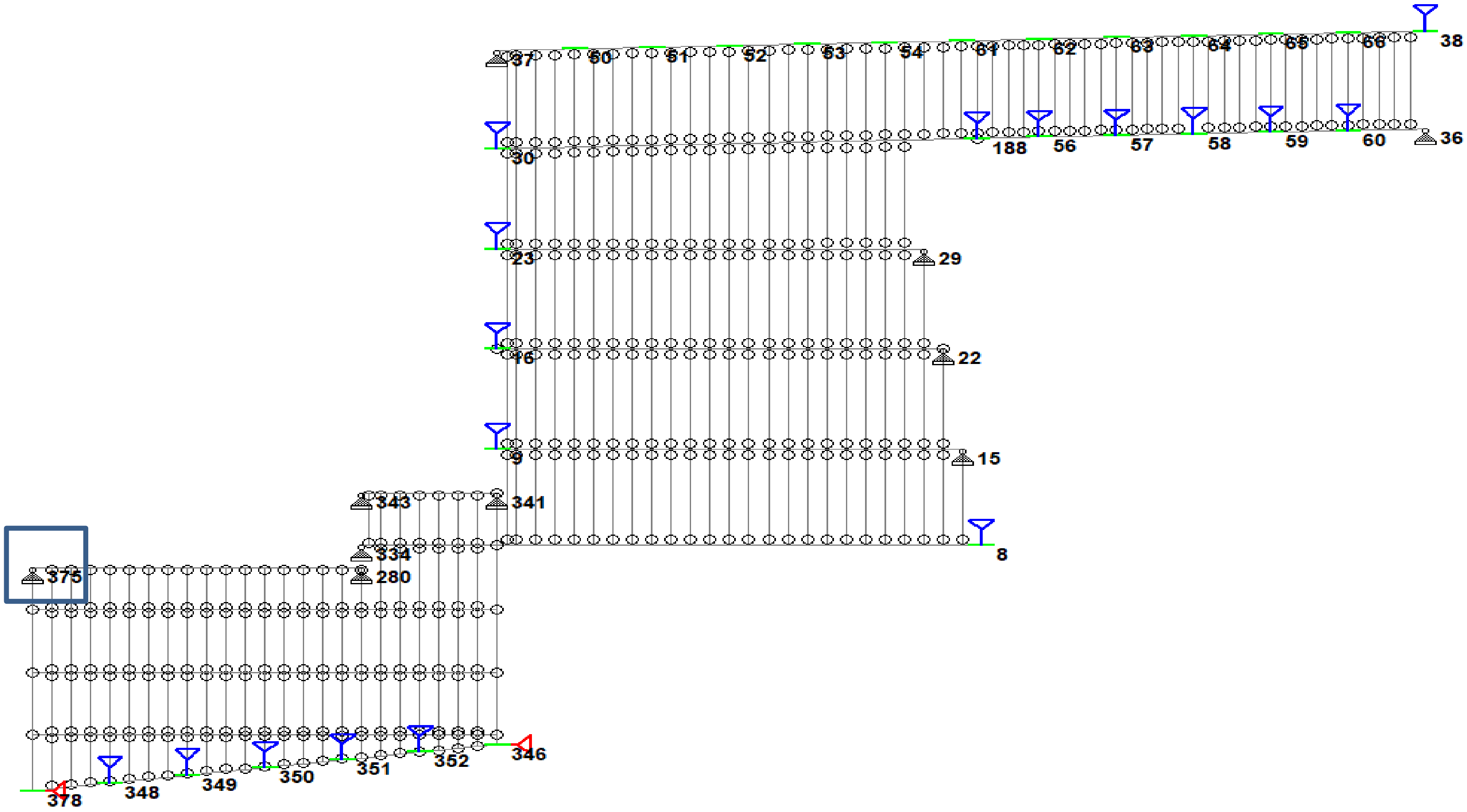
Base plate: **G250**

Thickness: t = 1 mm
Clearance hole: d_f = 18 mm



9.11. **<u>End plate and Embed design-Type-3</u>**

Below image show location of End plate and Embed design Type-3.



Check for Plate
Fx = 2.759 kN
Fy = 19.056 kN
Fz = 14.205 kN

Moment due to Fx,
My = 2.759 kN x 0.14m
= 0.39 kN.m

Mz = 2.759 kN x 0.14m
= 0.39 kN.m

Moment due to Fy,
Mx = 19.056 kN x 0.070m
= 1.34 kN.m

Moment due to Fz,
Mx = 14.205 kN x 0.075m
= 1.07 kN.m

Flexural capacity of plate in Y-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((300 x 12^2)/6)
= 1.62 kN.m > 0.39 kN.m ………. Hence OK

Flexural capacity of plate in Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((300 x 12^2)/6)
= 1.62 kN.m > 0.39 kN.m ………. Hence OK

Flexural capacity of plate in X-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((12 x 300^2)/6)
= 40.5 kN.m > 1.34 kN.m ………. Hence OK

Axial Tension capacity of plate in Z-direction,
= 0.9 x Ag x Fy
= 0.9 x (300 x 12) x 250
= 810 kN > 14.205 kN………. Hence OK

Combined axial & bending capacity of plate,
=(14.205/810) + (1.34/40.5) + (0.39/1.62)
= 0.3 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Fx = 2.759 kN Axial
Fy = 19.056 kN Shear
Fz = 14.205 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses fb = $\frac{Mx}{Zx}$

Direct stress fv = $\frac{Fz}{te \times l}$

Combined Bending & shear stress =$\sqrt{(f\mathrm{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_{YZ} = [Fy + Fz] / [L_w x thickness weld]
= [19.056 + 14.205] x 10^3 / [800 x 4.242]
= 9.8 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [2.759] x 10^3 / [800 x 4.242]
= 0.813 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

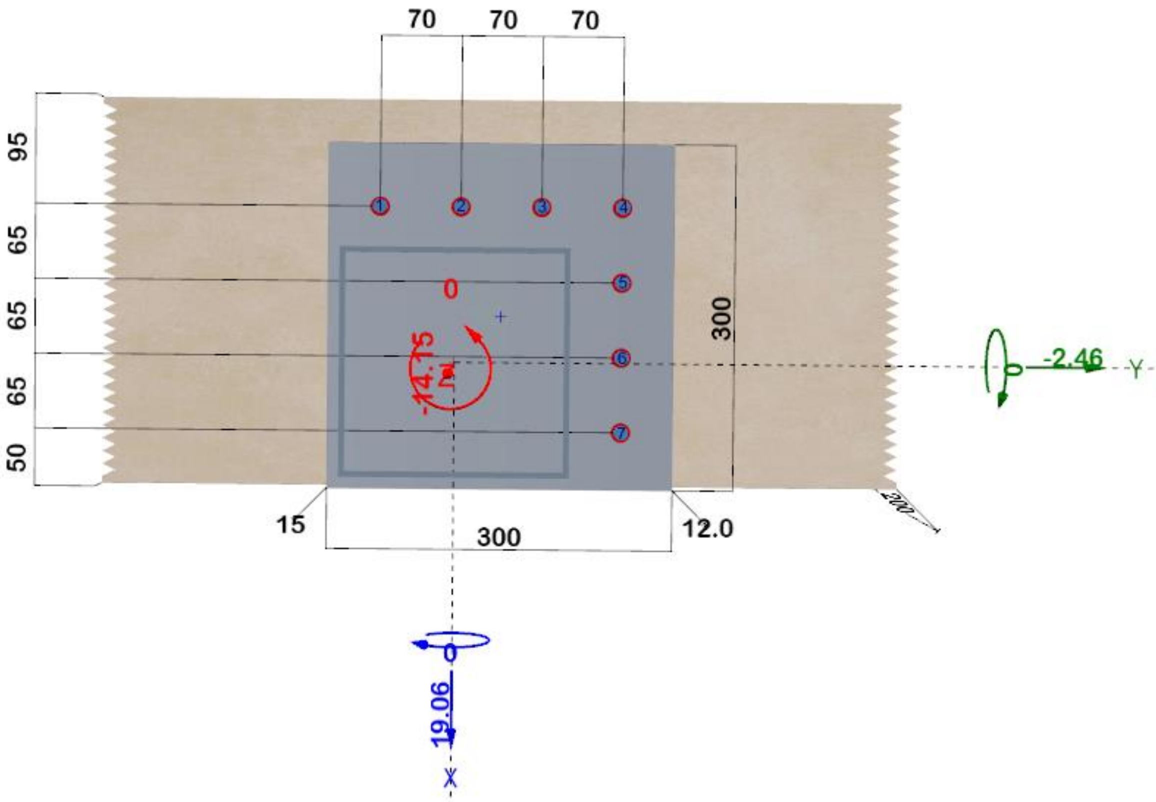
R_{b1} = (M_x) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (1.34) x 10^6 / [(200+200)^3/6 * 4.242]
= 0.034 N/mm^2

R_{b2} = (M_z) / Zx x weld thickness
Here, Z_x = bxd+(d^2/3) for unit weld length
= (0.39) x 10^6 / [(200x200)+200^2/3 * 4.242]
= 1.73 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

f_e = [$(R_x+R_{b1}+R_{b2})^2 + 3(R_{yz})^2$]$^{1/2}$
= [$(0.813+0.034+1.73)^2 + 3(9.8)^2$]$^{1/2}$
= 17.2 N/mm^2 < 233 N/mm^2 (Hence, OK)

Check for Anchor



Node number 375 reactions for anchor design is:
Fx = 19.06 kN
Fy = 2.46 kN
Fz = 14.15 kN

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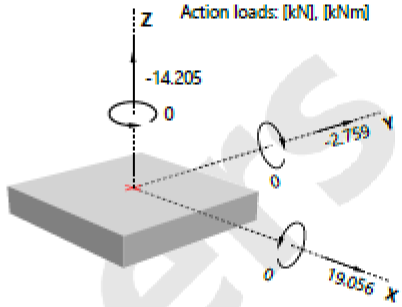
1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M12
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 100 mm
- Drilled hole Φ x h_0 = 14.0 x 100 mm

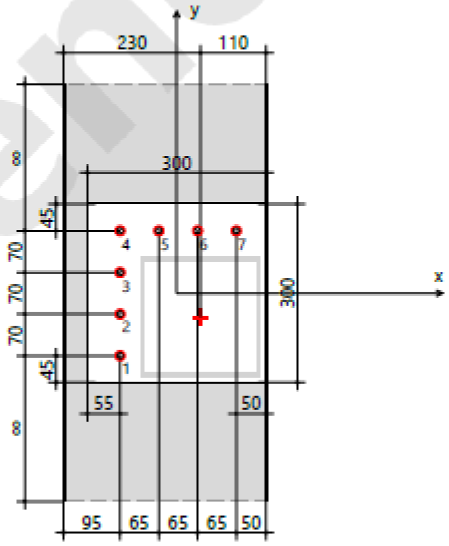


Base material:

- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f'_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

- Predominantly static and quasi-static design loads, α_{sus}=0.6



Installation:

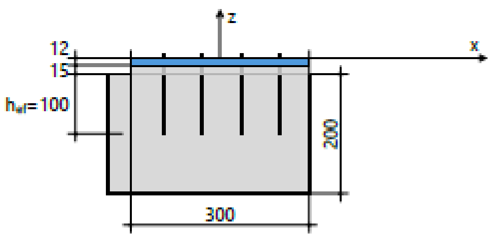
- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

Base plate:

- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, f_{yd}= $\phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 12.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 300 x 300 mm

Profile:

- Square Hollow Section: 200x5.0 SHS
  H x W x T x FT [mm]: 200 x 200 x 5.0 x 0.0
  Action point [mm]: [40, -40]
  Rotation counterclockwise: 90°



Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	L-y
1	-95.0	-105.0		
2	-95.0	-35.0		
3	-95.0	35.0		
4	-95.0	105.0		
5	-30.0	105.0		
6	35.0	105.0		
7	100.0	105.0		

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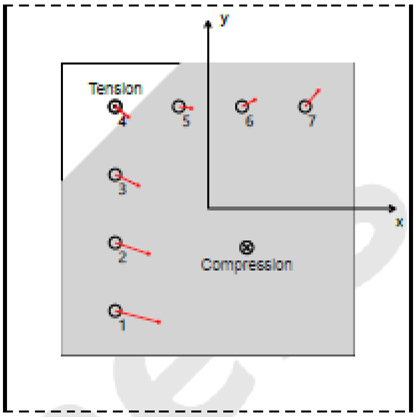
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	0.000	5.493	5.322	-1.360
2	0.000	4.328	4.109	-1.360
3	0.000	3.199	2.896	-1.360
4	0.011	2.163	1.682	-1.360
5	0.000	1.698	1.682	-0.233
6	0.000	1.905	1.682	0.894
7	0.000	2.629	1.682	2.020



Maximum concrete compressive strain [‰]: 0.0136
Maximum concrete compressive stress: 0.41 [N/mm²]
Resultant tension force in (x/y=-95.0/105.0): 0.011 [kN]
Resultant compression force in (x/y=39.9/-39.9): 14.216 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \leq f_{yd}$

b) $N^h_r \approx N^h_e$

N^h_r: highest anchor tension force on flexurally rigid base plate

N^h_e: highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	4	0.011	44.667	0.0	√
Combined failure	4	0.011	11.006	0.1	√
Concrete cone failure	4	0.011	19.665	0.1	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
67.0	0.667	44.667	0.011	0.000

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Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p}\cdot\psi_{A,Np}\cdot\psi_{s,Np}\cdot\psi_{g,Np}\cdot\psi_{ec,Np}\cdot\psi_{re,Np}$ $N^0_{Rk,p}=\psi_{sus}\cdot\pi\cdot d\cdot l_b\cdot\tau_{Rk}\cdot\psi_c$ [N] $\psi_{A,Np}=A_{p,N}/A^0_{p,N}$ $N_{Rd,Np}=N_{Rk,Np}\cdot\phi_{p,N}$

$s_{cr,Np}=7.3\cdot d\cdot(\psi_{sus}\cdot\tau_{Rk,ucr})^{0.5}\le 3\cdot l_b$ $\psi_{g,Np}=\psi^0_{g,Np}-(s_m/s_{cr,Np})^{0.5}\cdot(\psi^0_{g,Np}-1)\ge 1.0$

$\psi^0_{g,Np}=n^{0.5}-(n^{0.5}-1)\cdot(\tau_{Rk}/\tau_{Rk,c})^{1.5}\ge 1.0$ $\tau_{Rk,c}=k_3\cdot(h_{ef}\cdot f'_c)^{0.5}/(\pi\cdot d)$ $\psi^0_{sus}=0.73$ $\alpha_{sus}=0.6$ $\psi_{sus}=1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.5	1.231	12.0	7.7	40	100.0	270.0	135.0	100.0	0.556	0.000

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
25.524	62100	72901	0.852	0.911	95.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
1	-		1.0	1.0	0.0	0.0	1.000	1.000	1.000	19.810	11.006	0.011	0.001

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c}\cdot\psi_{A,N}\cdot\psi_{s,N}\cdot\psi_{re,N}\cdot\psi_{ec,N}\cdot\psi_{M,N}$ $N^0_{Rk,c}=k_1\cdot(f'_c)^{0.5}\cdot h_{ef}^{1.5}$ [N] $\psi_{A,N}=A_{c,N}/A^0_{c,N}$ $N_{Rd,c}=N_{Rk,c}\cdot\phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
48.699	73500	90000	0.817	7.7	0.556	100.0	300.0	150.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.89	1.0	0.0	0.0	1.0	1.0	1.0	1.0	35.396	19.665	0.011	0.001

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	1	5.493	6.222	88.3	√
Pry-out	3	3.199	3.492	91.6	√
Concrete edge failure (x+)	1,2,3,4,5,6,7	21.283	25.521	83.4	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M\cdot M_{Rk,s}/l$ $M_{Rk,s}=M^0_{Rk,s}(1-|N^*|/N_{Rd,s})$ $V_{Rd,s}=V_{Rk,s}\cdot\phi_{s,V}$ $\beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s}\cdot\phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
105.0	67.0	0.667	44.667	2.0	21.0	6.0	27.0	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s}(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M\cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
0.000	105.000	7.778	6.222	5.493	0.883

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $\quad N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\quad V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $\quad V_{Rd,cp} = V_{Rk,cp} \cdot \phi_{cp,V}$

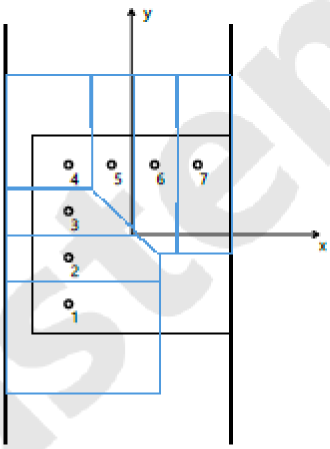
For stand-off installation (overturning moment): $V_{Rd,cp} = V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\quad \alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.71$ $\quad h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
100.0	9.5	270.0	135.0	12.0	100.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
25.524	11569	72901	0.159	1.0	-	1.0	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V* [kN]	$\beta_{V,cp}$
0.911	1.0	0.0	0.0	1.0	1.0	1.0	3.691	7.381	3.492	3.199	0.916

Related area for calculation of pry-out failure $A_{p,N}$:



Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $\quad V^0_{Rk,c} = k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\quad \psi_{A,V} = A_{c,V}/A^0_{c,V}$ $\quad V_{Rd,c} = V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\quad \alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\quad \beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} = V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\quad \alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.71$ $\quad h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
100.0	1.7	40	0.667	245.0	-	0.064	0.055	62.170	1.000	12.0	100.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V* [kN]	$\beta_{V,c}$
189000	270113	0.700	1.356	1.028	45.4	0.890	1.000	53.943	25.521	21.283	0.834

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \le 1.0$	-	not applicable
Concrete	3	0.000	0.916	$\beta_N + \beta_V \le 1.2$	76.3	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0.000	0.000	0.000	0.000	0.883	0.465	0.834	0.000	0.834	0.695	-
2	0.000	0.000	0.000	0.000	0.696	0.915	0.834	0.000	0.915	0.763	-
3	0.000	0.000	0.000	0.000	0.514	0.916	0.834	0.000	0.916	0.763	-
4	0.000	0.001	0.001	0.000	0.348	0.330	0.834	0.001	0.834	0.696	-
5	0.000	0.000	0.000	0.000	0.273	0.395	0.834	0.000	0.834	0.695	-
6	0.000	0.000	0.000	0.000	0.306	0.357	0.834	0.000	0.834	0.695	-
7	0.000	0.000	0.000	0.000	0.423	0.444	0.834	0.000	0.834	0.695	-

$\beta_{N,c,max,E}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,E}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,E}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,E}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^{\,0} = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^{\,\infty} = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^{\,h} = V^{*h} / 1.4$

Short-term displacement: $\delta_V^{\,0} = V_k^{\,h} \cdot \delta_{V0}$

Long-term displacement: $\delta_V^{\,\infty} = V_k^{\,h} \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	$\delta_N^{\,0}$ [mm]	$\delta_N^{\,\infty}$ [mm]
0.011	0.003	0.090	0.320	0.000	0.001

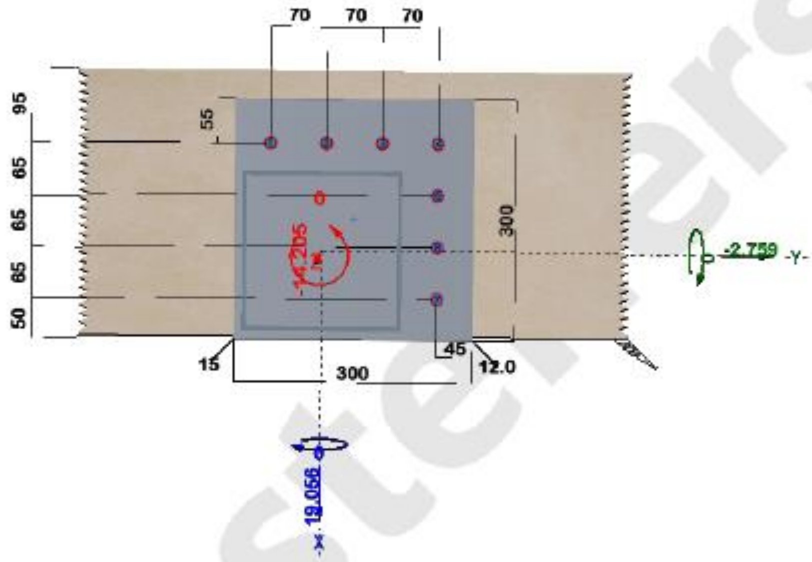
V^{*h} [kN]	$V_k^{\,h}$ [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	$\delta_V^{\,0}$ [mm]	$\delta_V^{\,\infty}$ [mm]
5.493	3.924	0.200	0.300	0.785	1.177

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

Anchorage figure in 3D:



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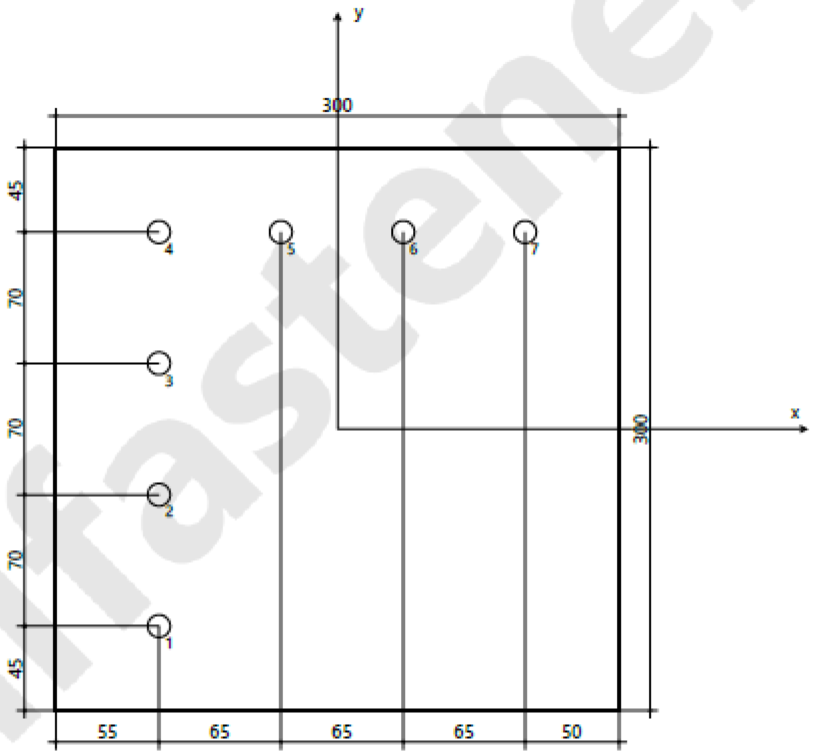
Anchor: VF22PRO+ & Threaded Rod Zn 8.8 M12

Drilled hole:	$d_0 \times h_0$ = 14 x 100 mm
Embedment depth:	h_{nom} = 100 mm
Effective anchorage depth:	h_{ef} = 100 mm
Installation torque:	T_{inst} = 40 Nm

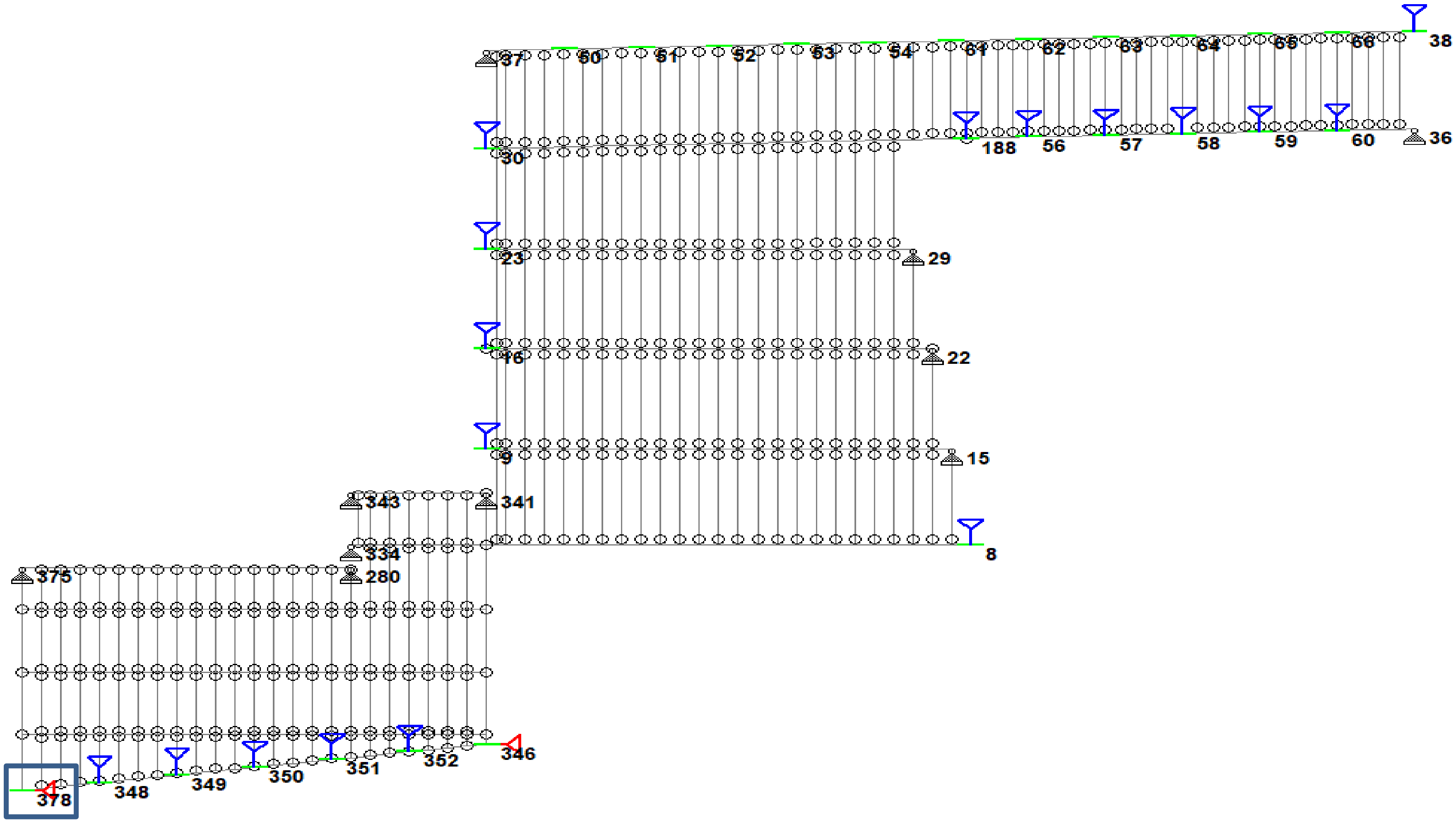
Base plate: G250

Thickness:	t = 12 mm
Clearance hole:	d_f = 14 mm

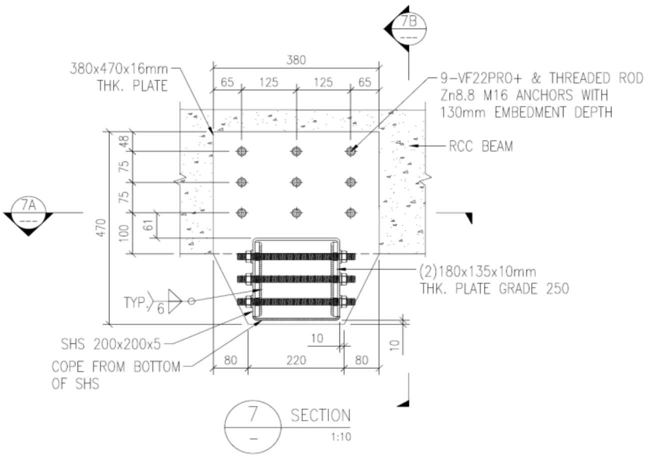


9.12. <u>**End plate and Embed design-Type-4**</u>

Below image show location of End plate and Embed design Type-4.



Check for Plate



For plate, governing reactions is:
Fx = 16.933 kN
Fy = 8.503 kN

Moment due to Fx,
Mz = 16.933 kN x 0.162m
= 2.75 kN.m

Flexural capacity of plate in Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((380 x 16^2)/6)
= 3.65 kN.m > 2.75 kN.m ………. Hence OK

Axial Tension capacity of plate in Y-direction,
= 0.9 x Ag x Fy
= 0.9 x (380 x 16) x 250
= 1368 kN > 8.503 kN………. Hence OK

Combined axial & bending capacity of plate,
=(8.503/1368) + (2.75/3.65)
= 0.76 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Fx = 16.933 kN Axial
Fy = 8.503 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_{YZ} = [Fy] / [L_w x thickness weld]
= [8.503] x 10^3 / [600 x 4.242]
= 3.34 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [16.933] x 10^3 / [600 x 4.242]
= 6.65 N/mm^2

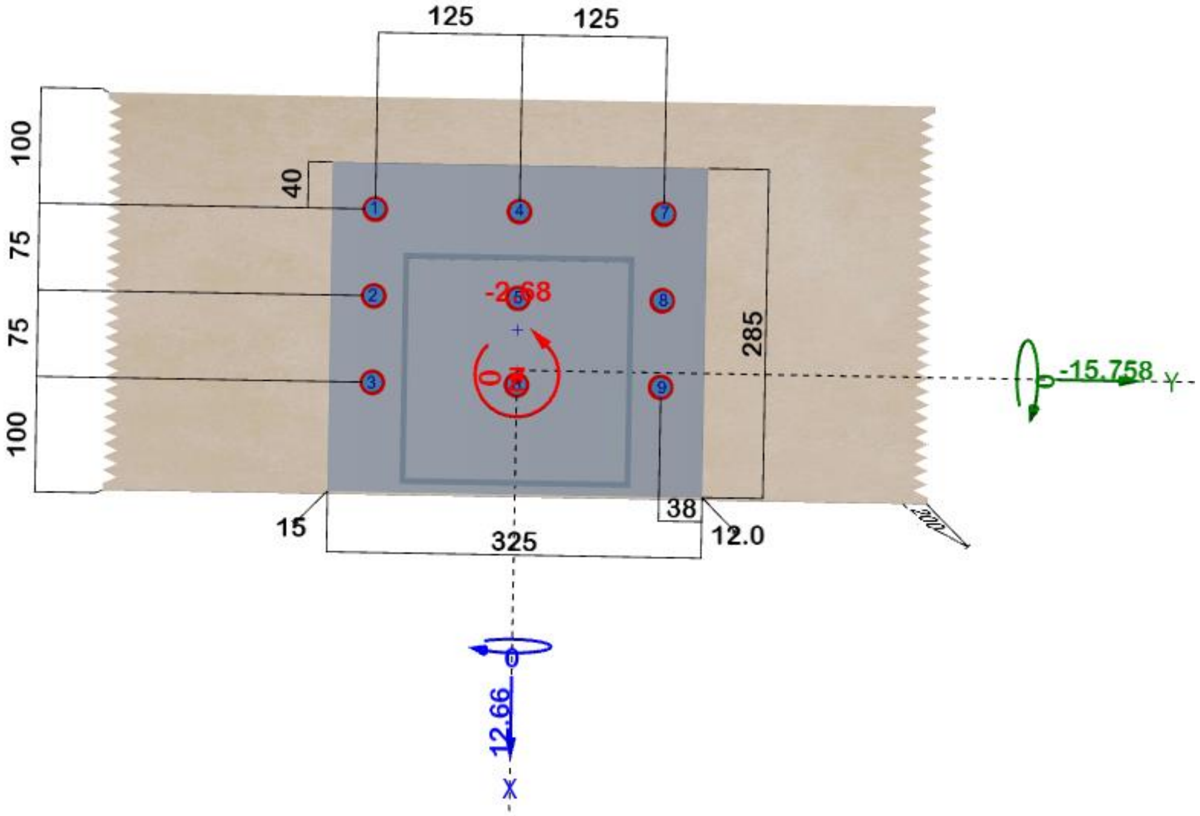
Bending stress in the Weld = Moment / Section Modulus

R_{b2} = (M_z) / Zx x weld thickness
Here, Z_x = bxd+(d^2/3) for unit weld length
= (2.75) x 10^6 / [(100x200)+200^2/3 * 4.242]
= 19.45 N/mm^2

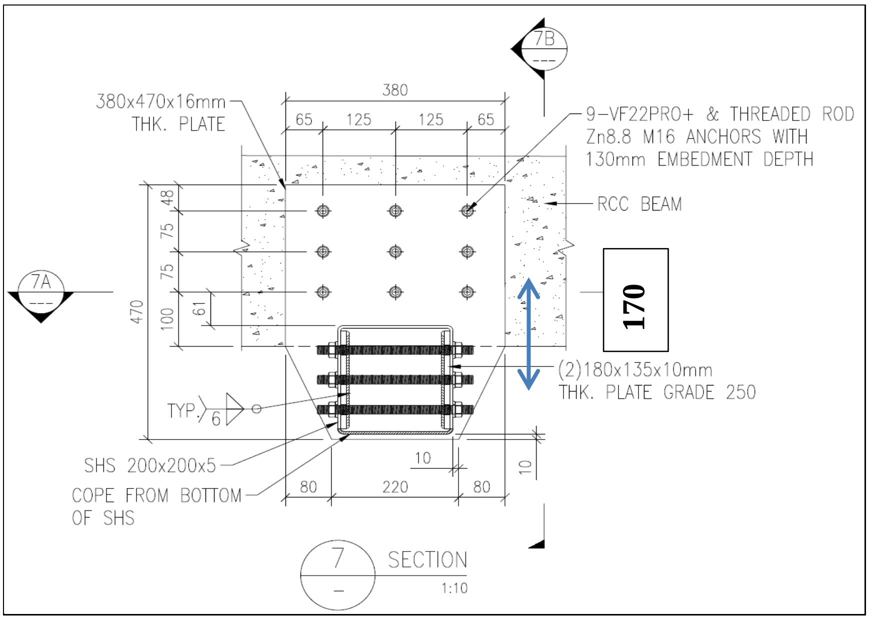
Check for combined bending and shear stress in the Fillet weld,

f_e = $[(R_x+R_{b1}+R_{b2})^2 + 3(R_{yz})^2]^{1/2}$
= $[(6.65+19.45)^2 + 3(3.34)^2]^{1/2}$
= 26.74 N/mm^2 < 233 N/mm² (Hence, OK)

Check for Anchor



Node number 378 reactions for anchor design is:
Fx = 12.66 kN
Fy = 15.71 kN
Fz = 0 kN
Moment(Mz) due to eccentricity = c/c distance of anchor to member center x Fy
= 0.170 x 15.758 = 2.68 kN.m



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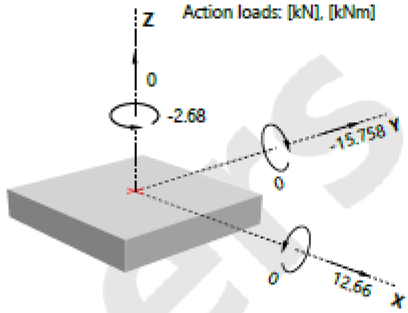
1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M16
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 130 mm
- Drilled hole Φ x h_0 = 18.0 x 130 mm



Base material:

- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

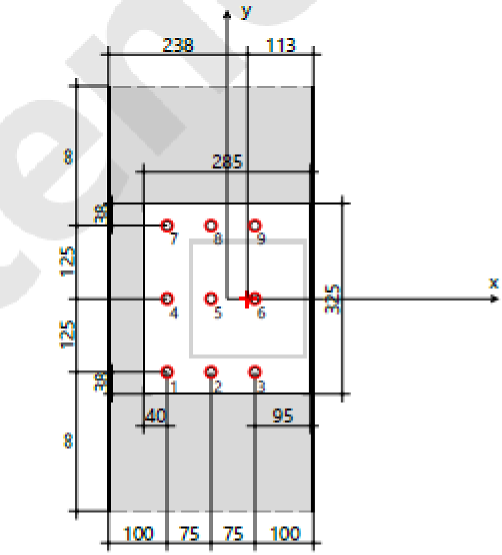
- Predominantly static and quasi-static design loads, α_{sus}=0.6

Installation:

- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

Base plate:

- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, $f_{yd} = \phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 12.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 285 x 325 mm

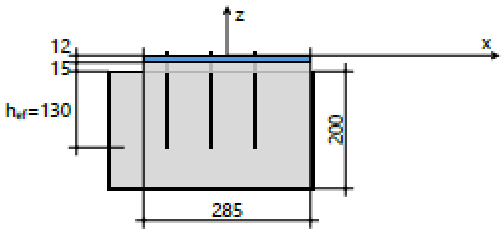


Profile:

- Square Hollow Section: 200x5.0 SHS
  H x W x T x FT [mm]: 200 x 200 x 5.0 x 0.0
  Action point [mm]: [35, 0]
  Rotation counterclockwise: 90°

Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	L-y
1	-102.5	-125.0		
2	-27.5	-125.0		
3	47.5	-125.0		
4	-102.5	0.0		
5	-27.5	0.0		
6	47.5	0.0		
7	-102.5	125.0		
8	-27.5	125.0		
9	47.5	125.0		



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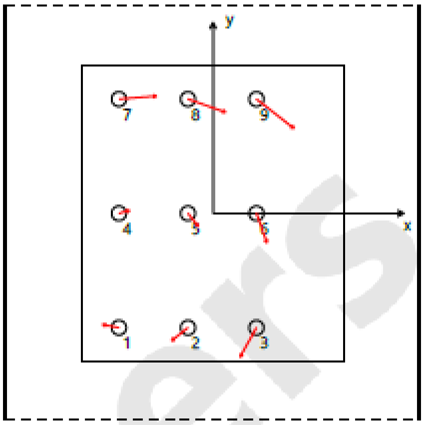
2. Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	0.000	2.224	-2.186	0.405
2	0.000	2.801	-2.186	-1.751
3	0.000	4.477	-2.186	-3.907
4	0.000	1.464	1.407	0.405
5	0.000	2.246	1.407	-1.751
6	0.000	4.152	1.407	-3.907
7	0.000	5.016	5.000	0.405
8	0.000	5.297	5.000	-1.751
9	0.000	6.345	5.000	-3.907



Resultant tension force in (x/y=0/0): 0 [kN]
Resultant compression force in (x/y=0/0): 0 [kN]
Remark: The edge distance is not to scale.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	-	-	-	-	not applicable
Combined failure	-	-	-	-	not applicable
Concrete cone failure	-	-	-	-	not applicable
Splitting failure	-	-	-	-	not applicable

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	9	6.345	14.676	43.2	√
Pry-out	8	5.297	6.421	82.5	√
Concrete edge failure (x+)	1,2,3,4,5,6,7,8,9	26.099	31.698	82.3	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $\quad M_{Rk,s}=M^0_{Rk,s}\,(1-|N^*|/N_{Rd,s})$ $\quad V_{Rd,s}=V_{Rk,s}\cdot\phi_{s,V}$ $\quad \beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s}\cdot\phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
266.0	126.0	0.667	84.000	2.0	21.0	8.0	29.0	0.8

N^* [kN]	$M_{Rk,s}=M^0_{Rk,s}\,(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M\cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V^* [kN]	$\beta_{V,s}$
0.000	266.000	18.345	14.676	6.345	0.432

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p} = N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $V_{Rk,cp} = k_8 \cdot N_{Rk,p}$ $V_{Rd,cp} = V_{Rk,cp} \cdot \phi_{cp,V}$

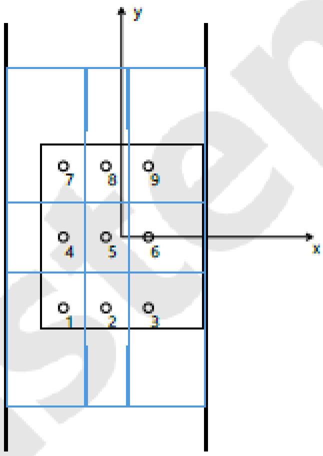
For stand-off installation (overturning moment): $V_{Rd,cp} = V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
130.0	9.0	350.4	175.2	16.0	130.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
44.242	17775	122780	0.145	1.0	-	1.0	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V^* [kN]	$\beta_{V,cp}$
1.0	1.0	0.0	0.0	1.0	1.0	1.0	6.403	12.806	6.421	5.297	0.825

Related area for calculation of pry-out failure $A_{p,N}$:



Concrete edge failure, direction x+

$V_{Rk,c} = V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} = k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} = A_{c,V}/A^0_{c,V}$ $V_{Rd,c} = V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} = V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
130.0	1.7	40	0.667	250.0	-	0.072	0.058	68.740	1.000	16.0	130.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V^* [kN]	$\beta_{V,c}$
200000	281250	0.711	1.369	1.210	105.5	0.780	1.000	63.215	31.698	26.099	0.823

3.3 Combined tension and shear

Interaction is not necessary.

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0.000	0.000	0.000	0.000	0.152	0.216	0.823	0.000	0.823	-	-
2	0.000	0.000	0.000	0.000	0.191	0.436	0.823	0.000	0.823	-	-
3	0.000	0.000	0.000	0.000	0.305	0.438	0.823	0.000	0.823	-	-
4	0.000	0.000	0.000	0.000	0.100	0.270	0.823	0.000	0.823	-	-
5	0.000	0.000	0.000	0.000	0.153	0.663	0.823	0.000	0.823	-	-
6	0.000	0.000	0.000	0.000	0.283	0.770	0.823	0.000	0.823	-	-
7	0.000	0.000	0.000	0.000	0.342	0.488	0.823	0.000	0.823	-	-
8	0.000	0.000	0.000	0.000	0.361	0.825	0.823	0.000	0.825	-	-
9	0.000	0.000	0.000	0.000	0.432	0.621	0.823	0.000	0.823	-	-

$\beta_{N,c,max,E}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,E}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,E}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,E}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^0 = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^\infty = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^h = V^{*h} / 1.4$

Short-term displacement: $\delta_V^0 = V_k^h \cdot \delta_{V0}$

Long-term displacement: $\delta_V^\infty = V_k^h \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	δ_N^0 [mm]	δ_N^∞ [mm]
0.000	0.000	0.050	0.180	**0.000**	**0.000**

V^{*h} [kN]	V_k^h [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	δ_V^0 [mm]	δ_V^∞ [mm]
6.345	4.532	0.110	0.170	**0.499**	**0.770**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

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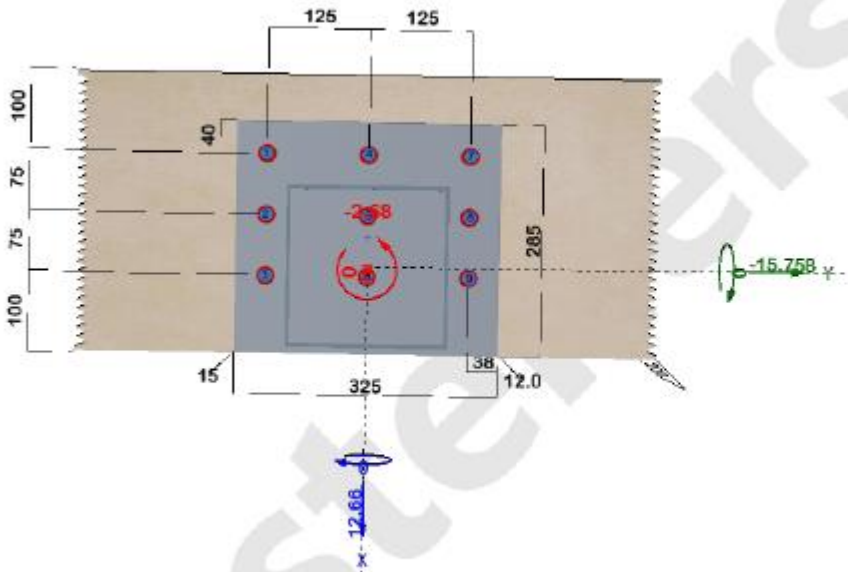
Anchorage figure in 3D:



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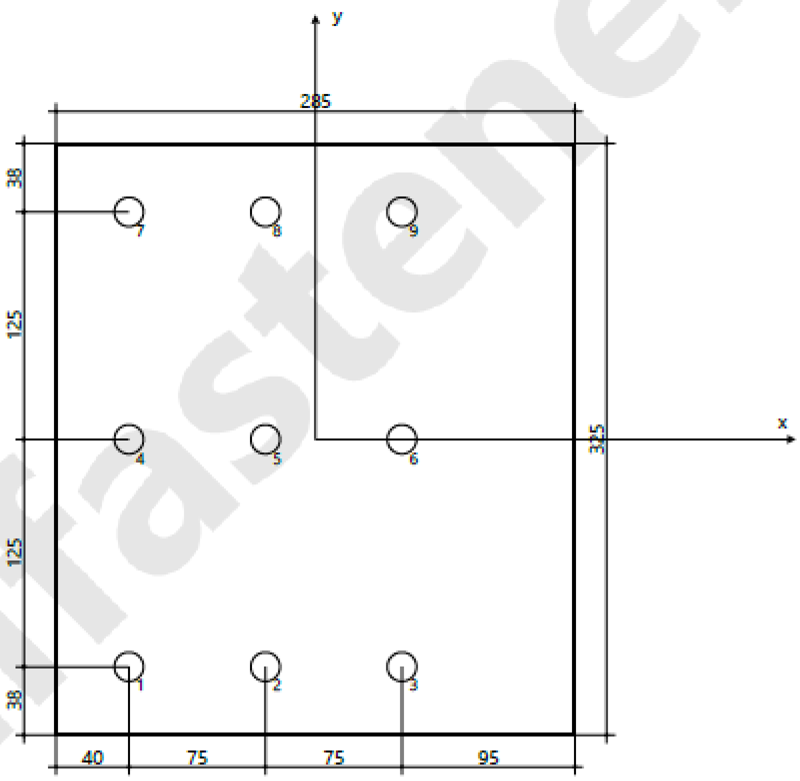
Anchor: **VF22PRO+ & Threaded Rod Zn 8.8 M16**

Drilled hole: $d_0 \times h_0$ = 18 x 130 mm
Embedment depth: h_{nom} = 130 mm
Effective anchorage depth: h_{ef} = 130 mm
Installation torque: T_{inst} = 80 Nm

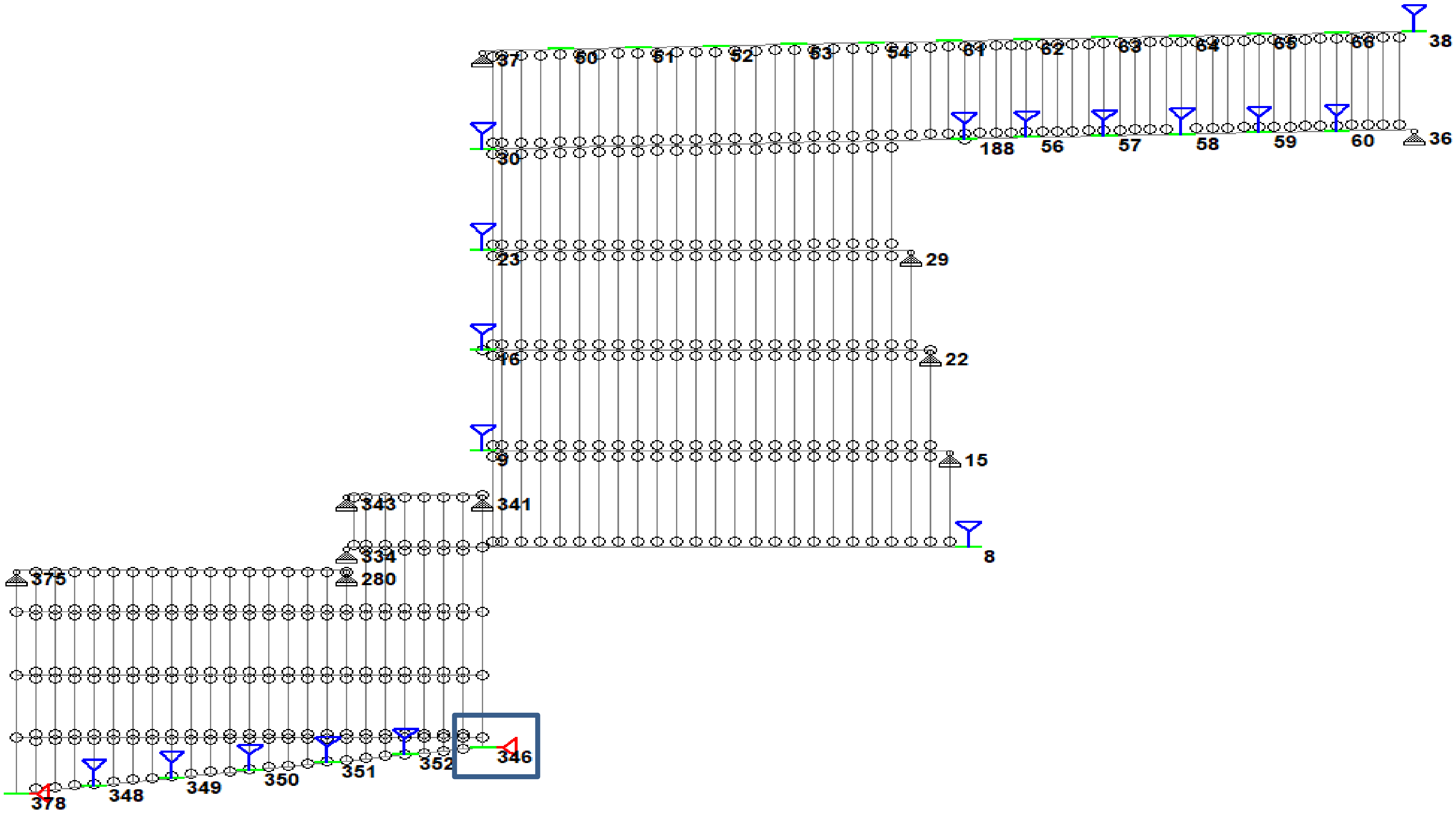
Base plate: **G250**

Thickness: t = 12 mm
Clearance hole: d_f = 18 mm



9.13. **End plate and Embed design-Type-5**

Below image show location of End plate and Embed design Type-5.



Check for Plate

Fx = 15.214 kN
Fy = 23.255 kN
Fz = 3.615 kN

Moment due to Fx,
Mz = 15.214 kN x 0.14m
= 2.13 kN.m

Moment due to Fz,
Mx = 3.615 kN x 0.23m
= 0.84 kN.m

Flexural capacity of plate in Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((260 x 16^2)/6)
= 2.5 kN.m > 2.13 kN.m ………. Hence OK

Flexural capacity of plate in X-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((16 x 260^2)/6)
= 40.56 kN.m > 0.84 kN.m ………. Hence OK

Axial Tension capacity of plate in Y-direction,
= 0.9 x Ag x Fy
= 0.9 x (260 x 16) x 250
= 936 kN > 23.255 kN………. Hence OK

Combined axial & bending capacity of plate,
=(23.255/936) + (2.13/2.5) + (0.84/40.56)
= 0.89 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Fx = 15.214 kN Axial
Fy = 23.255 kN Shear
Fz = 3.615 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25} = 233\ \text{N/mm}^2$

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_{YZ} = [Fy + Fz] / [L_w x thickness weld]
= [23.255 + 3.615] x 10^3 / [800 x 4.242]
= 7.92 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [15.214] x 10^3 / [800 x 4.242]
= 4.48 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

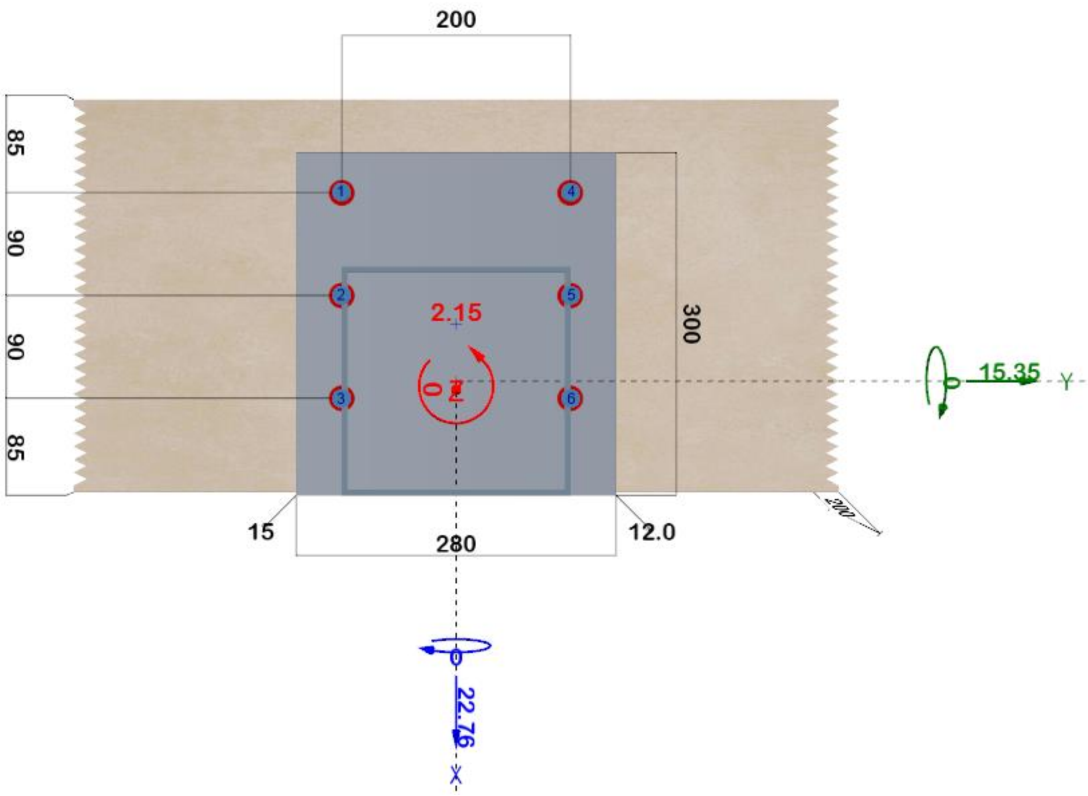
R_{b1} = (M_x) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (0.84) x 10^6 / [(200+200)^3/6 * 4.242]
= 0.018 N/mm^2

R_{b2} = (M_z) / Zx x weld thickness
Here, Z_x = bxd+(d^2/3) for unit weld length
= (2.13) x 10^6 / [(200x200)+200^2/3 * 4.242]
= 9.42 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

f_e = $[(R_x+R_{b1}+R_{b2})^2 + 3(R_{yz})^2]^{1/2}$
= $[(4.48+0.018+9.42)^2 + 3(7.92)^2]^{1/2}$
= 19.69 N/mm^2 < 233 N/mm² (Hence, OK)

Check for Anchor



Node number 346 reactions for anchor design is:
Fx = 22.76 kN
Fy = 15.35 kN
Fz = 0 kN
Moment(Mz) due to eccentricity = c/c distance of anchor to member center x Fy
= 0.140 x 15.35 = 2.15 kN.m

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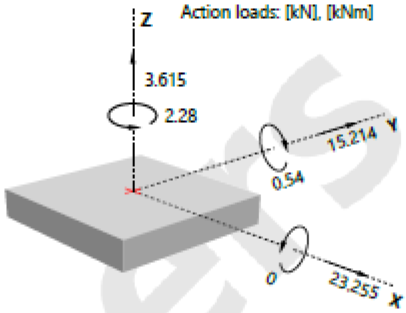
1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M16
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 130 mm
- Drilled hole Φ x h_0 = 18.0 x 130 mm



Base material:

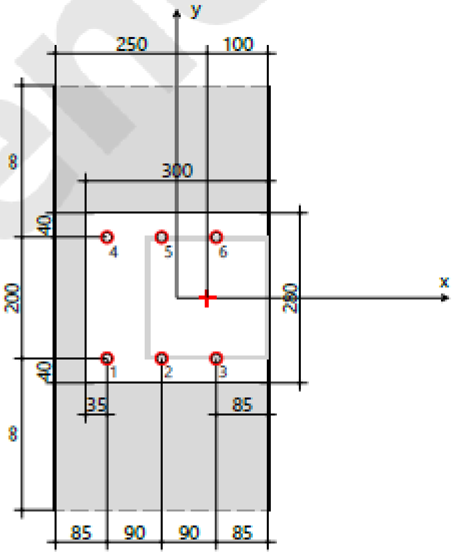
- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

- Predominantly static and quasi-static design loads, α_{sus}=0.6

Installation:

- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

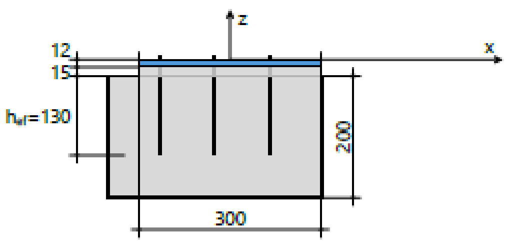


Base plate:

- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, f_{yd}= $\phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 12.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 300 x 280 mm

Profile:

- Square Hollow Section: 200x5.0 SHS
  H x W x T x FT [mm]: 200 x 200 x 5.0 x 0.0
  Action point [mm]: [50, 0]
  Rotation counterclockwise: 90°



Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	L-y
1	-115.0	-100.0		
2	-25.0	-100.0		
3	65.0	-100.0		
4	-115.0	100.0		
5	-25.0	100.0		
6	65.0	100.0		

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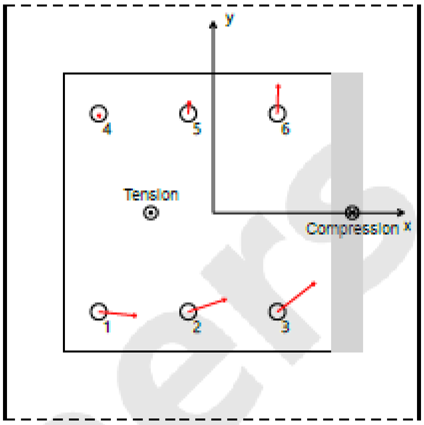
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	1.156	7.620	7.578	-0.797
2	0.712	7.991	7.578	2.536
3	0.268	9.584	7.578	5.868
4	1.156	0.815	0.173	-0.797
5	0.712	2.542	0.173	2.536
6	0.268	5.870	0.173	5.868



Maximum concrete compressive strain [‰]: 0.0046
Maximum concrete compressive stress: 0.14 [N/mm²]
Resultant tension force in (x/y=-62.4/0.0): 4.273 [kN]
Resultant compression force in (x/y=140.7/0.0): 0.658 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \leq f_{yd}$

b) $N^h_r \approx N^h_e$

N^h_r : highest anchor tension force on flexurally rigid base plate

N^h_e : highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	1,4	1.156	84.000	1.4	√
Combined failure	1,2,3,4,5,6	4.273	34.954	**12.2**	√
Concrete cone failure	1,2,3,4,5,6	4.273	37.950	11.3	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
126.0	0.667	84.000	1.156	0.014

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Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{ec,Np} \cdot \psi_{re,Np}$ $N^0_{Rk,p} = \psi_{sus} \cdot \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\psi_{A,Np} = A_{p,N}/A^0_{p,N}$ $N_{Rd,Np} = N_{Rk,Np} \cdot \phi_{p,N}$

$s_{cr,Np} = 7.3 \cdot d \cdot (\psi_{sus} \cdot \tau_{Rk,ucr})^{0.5} \leq 3 \cdot l_b$ $\psi_{g,Np} = \psi^0_{g,Np} - (s_m / s_{cr,Np})^{0.5} \cdot (\psi^0_{g,Np} - 1) \geq 1.0$

$\psi^0_{g,Np} = n^{0.5} - (n^{0.5} - 1) \cdot (\tau_{Rk} / \tau_{Rk,c})^{1.5} \geq 1.0$ $\tau_{Rk,c} = k_3 \cdot (h_{ef} \cdot f'_c)^{0.5} / (\pi \cdot d)$ $\psi^0_{sus} = 0.73$ $\alpha_{sus} = 0.6$ $\psi_{sus} = 1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.0	1.231	16.0	7.7	40	130.0	350.4	175.2	130.0	0.556	11.046

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
44.242	192675	122780	1.569	0.846	85.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
6	1.754	126.7	1.301	1.0	37.4	0.0	0.824	1.000	0.824	62.918	34.954	4.273	0.122

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $N^0_{Rk,c} = k_1 \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $N_{Rd,c} = N_{Rk,c} \cdot \phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
72.183	206500	152100	1.358	7.7	0.556	130.0	390.0	195.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.831	1.0	37.4	0.0	0.839	1.0	0.839	1.0	68.309	37.950	4.273	0.113

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	3	9.584	14.629	65.5	√
Pry-out	2	7.991	8.925	89.5	√
Concrete edge failure (x+)	1,2,3,4,5,6	29.342	29.754	98.6	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $M_{Rk,s}=M^0_{Rk,s} (1-|N^*|/N_{Rd,s})$ $V_{Rd,s}=V_{Rk,s} \cdot \phi_{s,V}$ $\beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s} \cdot \phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
266.0	126.0	0.667	84.000	2.0	21.0	8.0	29.0	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s} (1-|N^*|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
0.268	265.151	18.286	14.629	9.584	0.655

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $V_{Rd,cp} = V_{Rk,cp} \cdot \phi_{cp,V}$

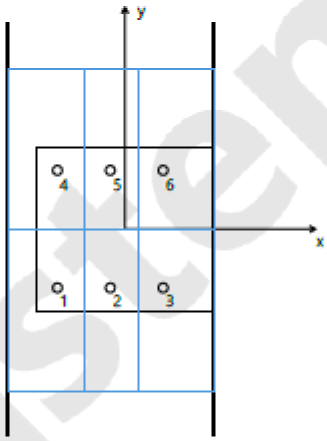
For stand-off installation (overturning moment): $V_{Rd,cp} = V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
130.0	9.0	350.4	175.2	16.0	130.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
44.242	24706	122780	0.201	1.0	-	1.0	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V* [kN]	$\beta_{V,cp}$
1.0	1.0	0.0	0.0	1.0	1.0	1.0	8.899	17.799	8.925	7.991	0.895

Related area for calculation of pry-out failure $A_{p,N}$:



Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} = k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} = A_{c,V}/A^0_{c,V}$ $V_{Rd,c} = V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} = V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\alpha_h = (h_h - a_3) / (e_1 + h_h) = 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
130.0	1.7	40	0.667	265.0	-	0.070	0.057	74.347	1.000	16.0	130.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V* [kN]	$\beta_{V,c}$
199000	316013	0.630	1.410	1.152	111.7	0.781	1.000	59.340	29.754	29.342	0.986

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \le 1.0$	-	not applicable
Concrete	1,2,3,4,5,6	0.122	0.986	$\beta_N + \beta_V \le 1.2$	92.4	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0.014	0.122	0.113	0.000	0.526	0.696	0.986	0.122	0.986	0.924	-
2	0.008	0.122	0.113	0.000	0.549	0.895	0.986	0.122	0.986	0.924	-
3	0.003	0.122	0.113	0.000	0.655	0.882	0.986	0.122	0.986	0.924	-
4	0.014	0.122	0.113	0.000	0.056	0.074	0.986	0.122	0.986	0.924	-
5	0.008	0.122	0.113	0.000	0.175	0.285	0.986	0.122	0.986	0.924	-
6	0.003	0.122	0.113	0.000	0.401	0.540	0.986	0.122	0.986	0.924	-

$\beta_{N,c,max,E}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,E}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,E}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,E}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^{\,0} = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^{\,\infty} = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^{\,h} = V^{*h} / 1.4$

Short-term displacement: $\delta_V^{\,0} = V_k^{\,h} \cdot \delta_{V0}$

Long-term displacement: $\delta_V^{\,\infty} = V_k^{\,h} \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	$\delta_N^{\,0}$ [mm]	$\delta_N^{\,\infty}$ [mm]
1.156	0.177	0.050	0.180	**0.006**	**0.023**

V^{*h} [kN]	$V_k^{\,h}$ [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	$\delta_V^{\,0}$ [mm]	$\delta_V^{\,\infty}$ [mm]
9.584	6.846	0.110	0.170	**0.753**	**1.164**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

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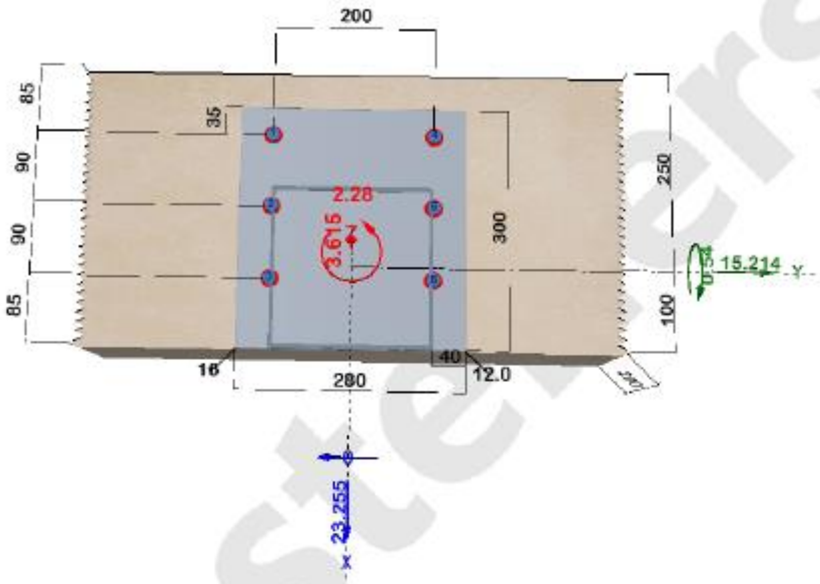
Anchorage figure in 3D:



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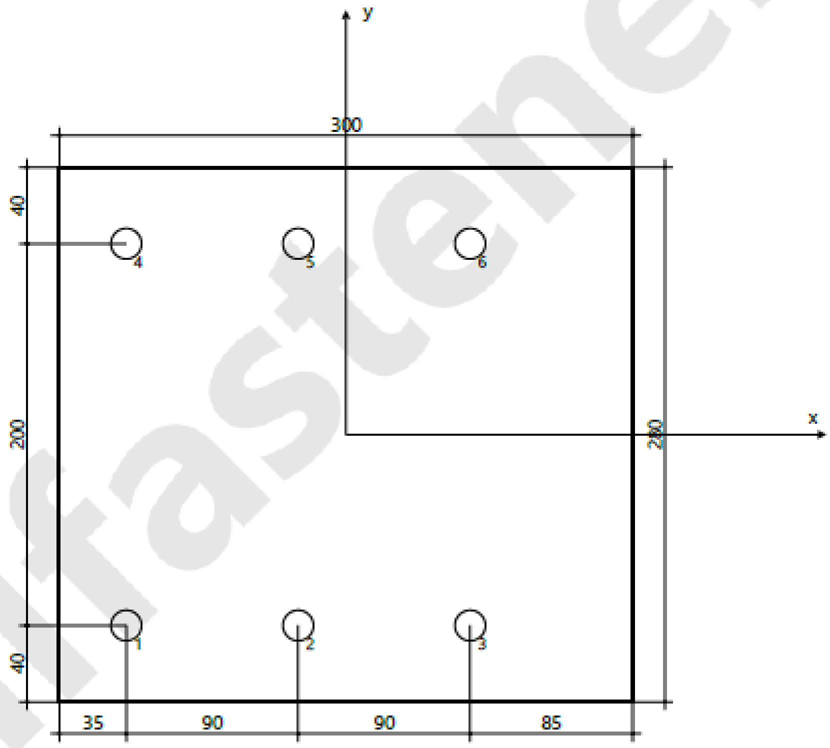
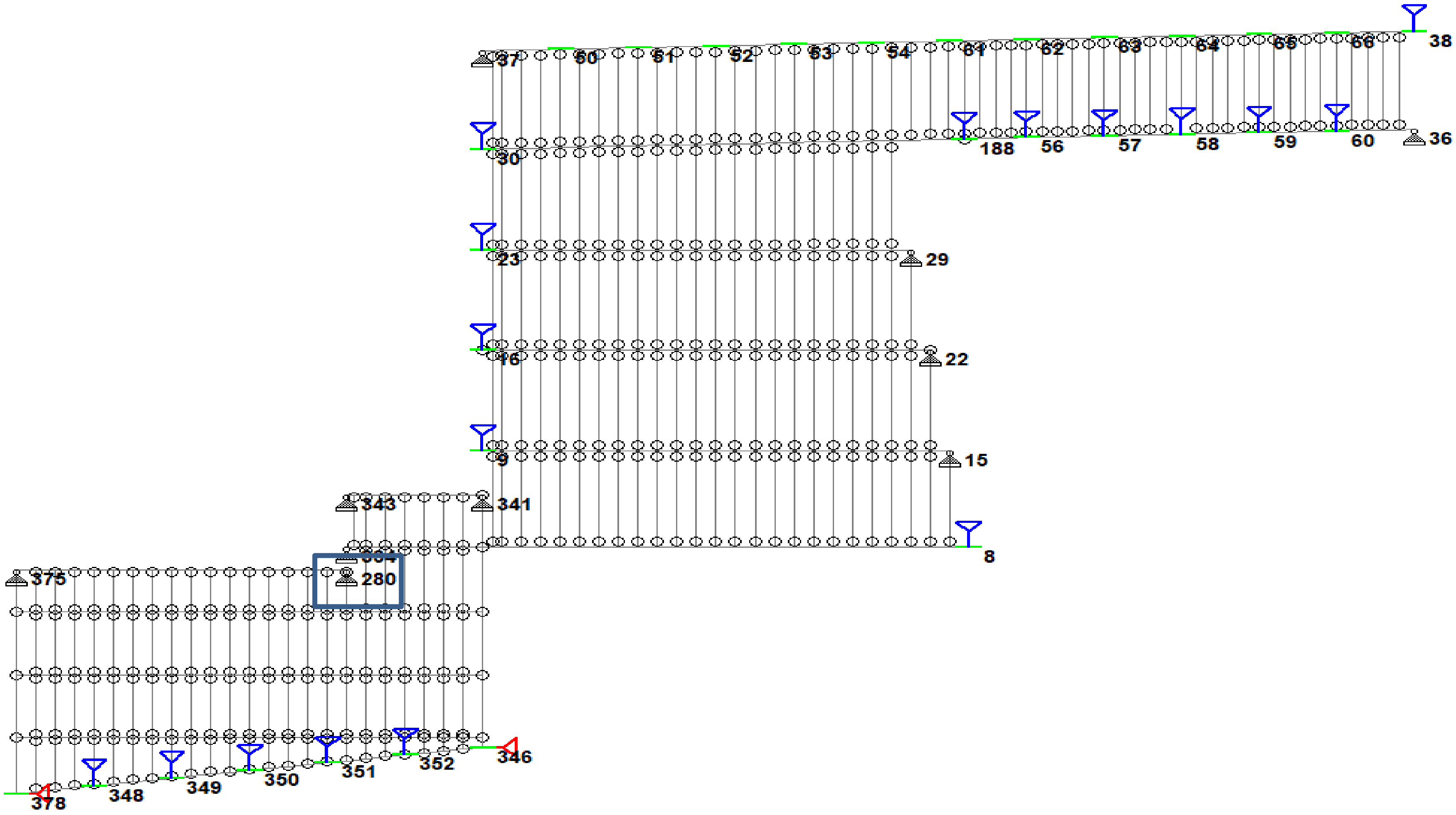
Anchor:	**VF22PRO+ & Threaded Rod Zn 8.8 M16**
Drilled hole:	$d_0 \times h_0$ = 18 x 130 mm
Embedment depth:	h_{nom} = 130 mm
Effective anchorage depth:	h_{ef} = 130 mm
Installation torque:	T_{inst} = 80 Nm

Base plate:	**G250**
Thickness:	t = 12 mm
Clearance hole:	d_f = 18 mm

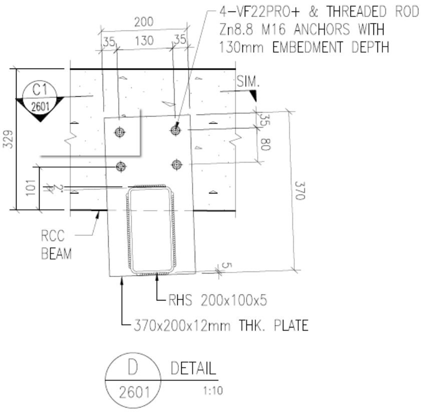


9.14. <u>End plate and Embed design-Type-6</u>

Below image show location of End plate and Embed design Type-6.



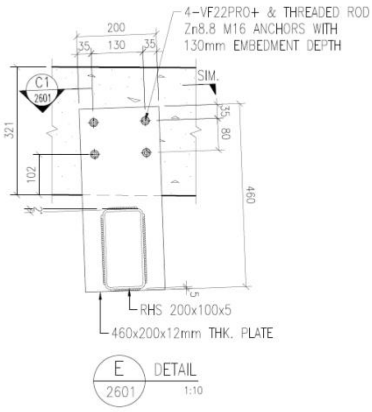
<u>**Check for Plate**</u>



For plate, governing reactions is:
Fx = 0.448 kN
Fy = 9.377 kN
Fz = 2.987 kN

Moment due to Fx,
Mz = 0.448 kN x 0.140m
= 0.063 kN.m

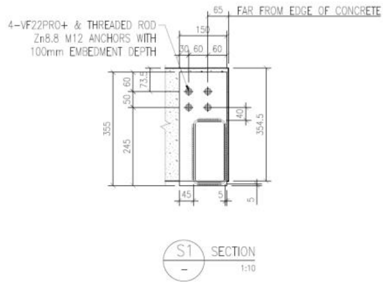
Moment due to Fz,
Mx = 2.987 kN x 0.165m
= 0.5 kN.m



Flexural capacity of plate in Z-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((200 x 12^2)/6)
= 1.08 kN.m > 0.063 kN.m ………. Hence OK

Flexural capacity of plate in X-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((12 x 200^2)/6)
= 18 kN.m > 0.5 kN.m ………. Hence OK

Axial Tension capacity of plate Y-direction,
= 0.9 x Ag x Fy
= 0.9 x (200 x 12) x 250
= 540 kN > 9.377 kN………. Hence OK



Combined axial & bending capacity of plate,
=(9.377/540) + (0.063/1.08) + (0.5/18)
= 0.104 < 1 ………. Hence SAFE in combined action

<u>**Check for 6mm Weld**</u>

Fx = 0.448 kN Axial
Fy = 9.377 kN Shear
Fz = 2.987 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(fb)^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_{YZ} = [Fy + Fz] / [L_w x thickness weld]
= [9.377+2.987] x 10^3 / [600 x 4.242]
= 11 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [0.448] x 10^3 / [600 x 4.242]
= 0.18 N/mm^2

Bending stress in the Weld = Moment / Section Modulus

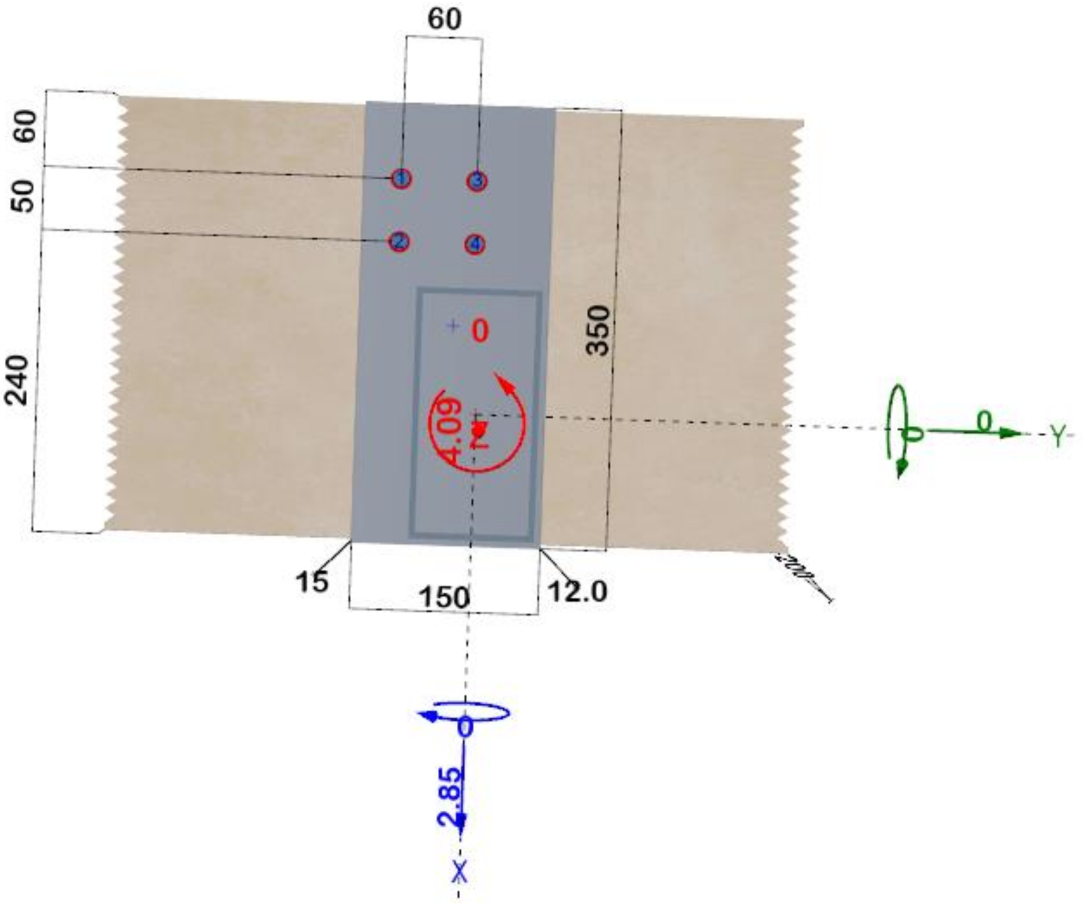
R_{b1} = (M_x) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (0.5) x 10^6 / [(100+200)^3/6 * 4.242]
= 0.026 N/mm^2

R_{b2} = (M_z) / Zx x weld thickness
Here, Z_x = bxd+(d^2/3) for unit weld length
= (0.063) x 10^6 / [(100x200)+200^2/3 * 4.242]
= 0.45 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [(R_x+R_{b1}+R_{b2})^2 + 3(R_{yz})^2]^{1/2}$
$= [(0.18+0.026+0.45)^2 + 3(11)^2]^{1/2}$
= 19.06 N/mm^2 < 233 N/mm^2 (Hence, OK)

Check for Anchor



Node number 280 reactions for anchor design is:
Fx = 2.85 kN
Fy = 0 kN
Fz = 4.09 kN

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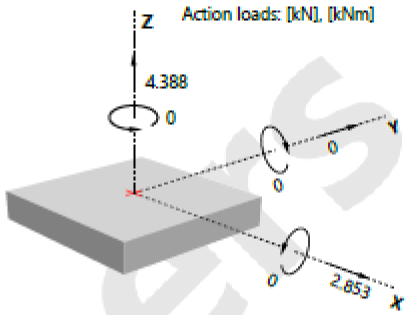
1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M12
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 100 mm
- Drilled hole Φ x h_0 = 14.0 x 100 mm

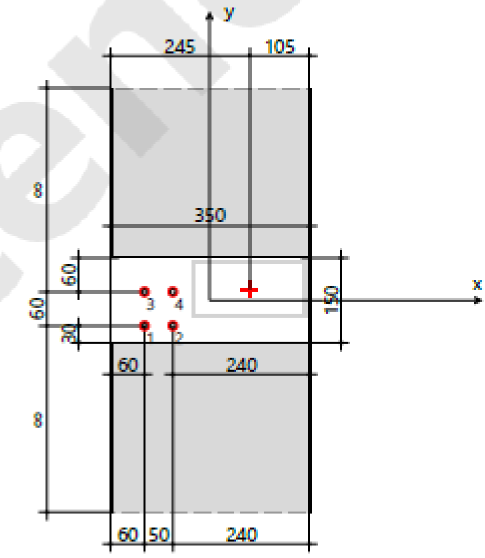


Base material:

- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f'_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

- Predominantly static and quasi-static design loads, α_{sus}=0.6



Installation:

- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling

Base plate:

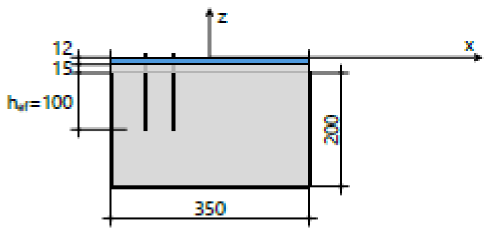
- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, f_{yd}= $\phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 12.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 350 x 150 mm

Profile:

- Rectangular Hollow Section: 200x100x5.0 RHS
  H x W x T x FT [mm]: 200 x 100 x 5.0 x 0.0
  Action point [mm]: [70, 20]
  Rotation counterclockwise: 90°

Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	Slotted hole L-y
1	-115.0	-45.0		
2	-65.0	-45.0		
3	-115.0	15.0		
4	-65.0	15.0		



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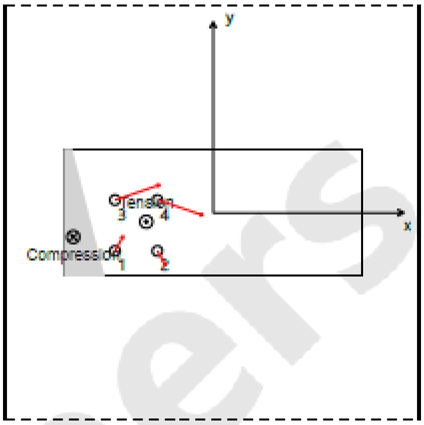
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	1.134	0.466	0.222	0.409
2	3.968	0.466	0.222	-0.409
3	2.002	1.272	1.204	0.409
4	4.835	1.272	1.204	-0.409



Maximum concrete compressive strain [‰]: 0.1525
Maximum concrete compressive stress: 4.57 [N/mm²]
Resultant tension force in (x/y=-78.1/-10.6): 11.940 [kN]
Resultant compression force in (x/y=-164.2/-28.4): 7.552 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \leq f_{yd}$

b) $N^h_r \approx N^h_e$

N^h_r : highest anchor tension force on flexurally rigid base plate

N^h_e : highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	4	4.835	44.667	10.8	√
Combined failure	1,2,3,4	11.940	15.643	76.3	√
Concrete cone failure	1,2,3,4	11.940	20.777	57.5	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
67.0	0.667	44.667	4.835	0.108

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Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{ec,Np} \cdot \psi_{re,Np}$ $N^0_{Rk,p} = \psi_{sus} \cdot \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\psi_{A,Np} = A_{p,N}/A^0_{p,N}$ $N_{Rd,Np} = N_{Rk,Np} \cdot \phi_{p,N}$

$s_{cr,Np} = 7.3 \cdot d \cdot (\psi_{sus} \cdot \tau_{Rk,ucr})^{0.5} \leq 3 \cdot l_b$ $\psi_{g,Np} = \psi^0_{g,Np} - (s_m / s_{cr,Np})^{0.5} \cdot (\psi^0_{g,Np} - 1) \geq 1.0$

$\psi^0_{g,Np} = n^{0.5} - (n^{0.5} - 1) \cdot (\tau_{Rk} / \tau_{Rk,c})^{1.5} \geq 1.0$ $\tau_{Rk,c} = k_3 \cdot (h_{ef} \cdot f'_c)^{0.5} / (\pi \cdot d)$ $\psi^0_{sus} = 0.73$ $\alpha_{sus} = 0.6$ $\psi_{sus} = 1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.5	1.231	12.0	7.7	40	100.0	270.0	135.0	100.0	0.556	12.918

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
25.524	80850	72901	1.109	0.833	60.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
4	1.621	55.0	1.34	1.0	11.9	4.4	0.919	0.969	0.890	28.157	15.643	11.940	0.763

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $N^0_{Rk,c} = k_1 \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $N_{Rd,c} = N_{Rk,c} \cdot \phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
48.699	93600	90000	1.040	7.7	0.556	100.0	300.0	150.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.82	1.0	11.9	4.4	0.927	0.972	0.901	1.0	37.399	20.777	11.940	0.575

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	4	1.272	5.549	22.9	√
Pry-out	3	1.272	3.883	32.8	√
Concrete edge failure (x+)	1,2,3,4	3.363	26.082	12.9	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $M_{Rk,s}=M^0_{Rk,s}(1-|N^*|/N_{Rd,s})$ $V_{Rd,s}=V_{Rk,s} \cdot \phi_{s,V}$ $\beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s} \cdot \phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
105.0	67.0	0.667	44.667	2.0	21.0	6.0	27.0	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s}(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
4.835	93.633	6.936	5.549	1.272	0.229

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $V_{Rd,cp} =V_{Rk,cp} \cdot \phi_{cp,V}$

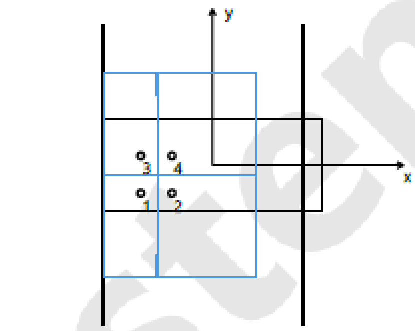
For stand-off installation (overturning moment): $V_{Rd,cp} =V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.71$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
100.0	9.5	270.0	135.0	12.0	100.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
25.524	14064	72901	0.193	1.0	-	1.0	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V* [kN]	$\beta_{V,cp}$
0.833	1.0	0.0	0.0	1.0	1.0	1.0	4.103	8.207	3.883	1.272	0.328

Related area for calculation of pry-out failure $A_{p,N}$:



Remark: Edge distance (+x) is not to scale.

Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} =k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} =A_{c,V}/A^0_{c,V}$ $V_{Rd,c} =V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} =V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.71$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
100.0	1.7	40	0.667	290.0	-	0.059	0.053	78.383	1.000	12.0	100.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V* [kN]	$\beta_{V,c}$
186000	378450	0.491	1.475	1.023	23.6	0.949	1.000	55.128	26.082	3.363	0.129

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \le 1.0$	-	not applicable
Concrete	3	0.763	0.328	$\beta^{1.5}_N + \beta^{1.5}_V \le 1.0$	85.4	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0.025	0.763	0.575	0.000	0.077	0.120	0.129	0.763	0.129	0.713	-
2	0.089	0.763	0.575	0.000	0.082	0.057	0.129	0.763	0.129	0.713	-
3	0.045	0.763	0.575	0.000	0.214	0.328	0.129	0.763	0.328	0.854	-
4	0.108	0.763	0.575	0.000	0.229	0.155	0.129	0.763	0.155	0.728	-

$\beta_{N,c,max,E}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,E}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,E}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,E}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^{0} = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^{\infty} = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^{h} = V^{*h} / 1.4$

Short-term displacement: $\delta_V^{0} = V_k^{h} \cdot \delta_{V0}$

Long-term displacement: $\delta_V^{\infty} = V_k^{h} \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	δ_N^{0} [mm]	δ_N^{∞} [mm]
4.835	1.283	0.090	0.320	**0.082**	**0.293**

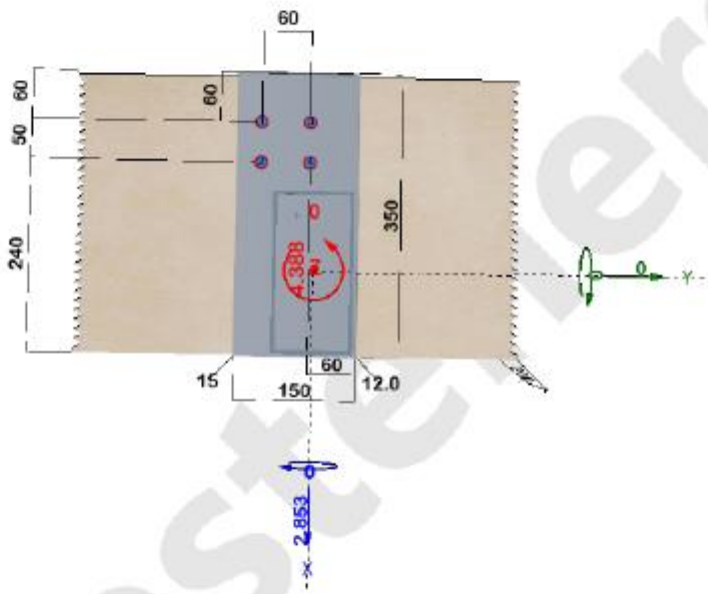
V^{*h} [kN]	V_k^{h} [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	δ_V^{0} [mm]	δ_V^{∞} [mm]
1.272	0.909	0.200	0.300	**0.182**	**0.273**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

Anchorage figure in 3D:



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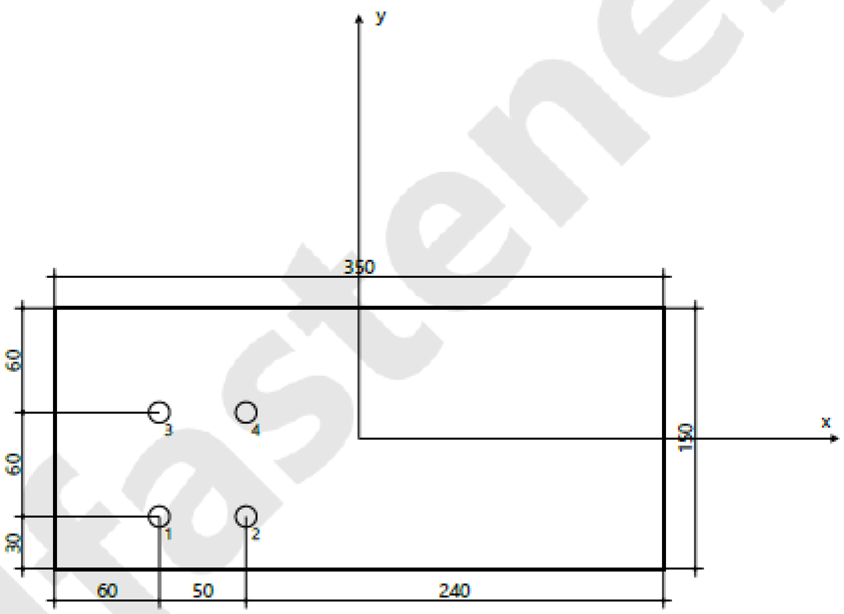
Anchor: **VF22PRO+ & Threaded Rod Zn 8.8 M12**

Drilled hole: $d_0 \times h_0$ = 14 x 100 mm
Embedment depth: h_{nom} = 100 mm
Effective anchorage depth: h_{ef} = 100 mm
Installation torque: T_{inst} = 40 Nm

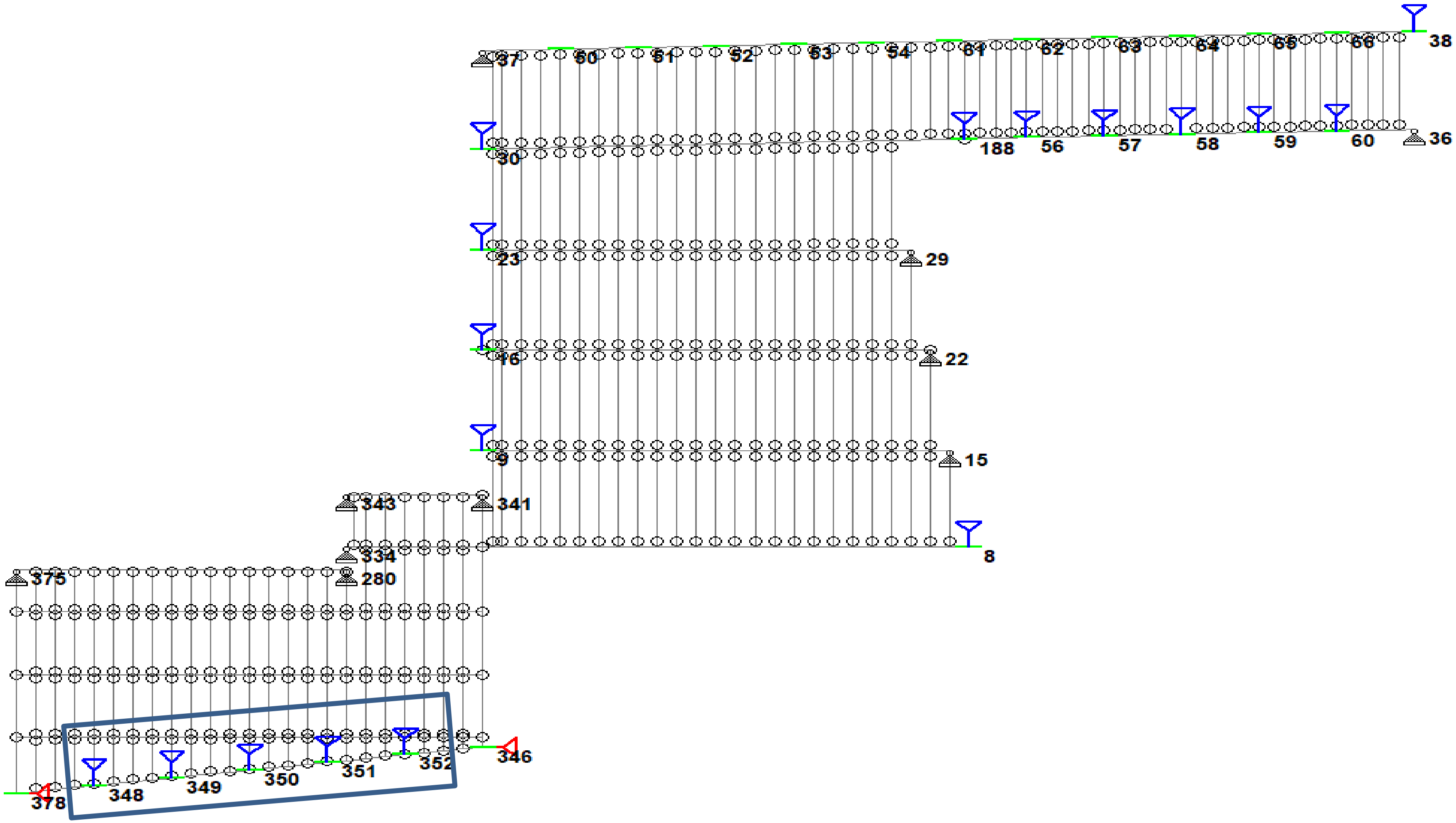
Base plate: **G250**

Thickness: t = 12 mm
Clearance hole: d_f = 14 mm



9.15. **<u>End plate and Embed design-Type-7</u>**

Below image show location of End plate and Embed design Type-7.



Check for Plate

For plate, governing reactions is:
Fx = 25.674 kN
Fy = 0.736 kN

Moment due to Fx,
My = 25.674 kN x 0.040m
= 1.03 kN.m

Flexural capacity of plate in Y-direction,
= 0.9 x Fy x Z
= 0.9 x 250 x ((200 x 12^2)/6)
= 1.08 kN.m > 1.03 kN.m ………. Hence OK

Axial Tension capacity of plate in Y-direction,
= 0.9 x Ag x Fy
= 0.9 x (200 x 12) x 250
= 540 kN > 0.736 kN………. Hence OK

Combined axial & bending capacity of plate,
=(0.736/540) + (1.03/1.08)
= 0.96 < 1 ………. Hence SAFE in combined action

Check for 6mm Weld

Fx = 25.674 kN Axial
Fy = 0.736 kN Shear

Effective throat thickness = 0.707 x 6 = 4.242 mm

Permissible weld stress $= \frac{f_u}{\sqrt{3} \times \beta_w \times \gamma_{M2}} = \frac{430}{\sqrt{3} \times 0.85 \times 1.25}$ = 233 N/mm^2

Bending stresses $f\text{b} = \frac{\text{Mx}}{Zx}$

Direct stress $f\text{v} = \frac{\text{Fz}}{\text{te x l}}$

Combined Bending & shear stress $=\sqrt{(f\text{b})^2 + 3(fv)^2}$

Direct Shear stress in the Weld = Load / Effective area of weld

R_Y = [Fy] / [L_w x thickness weld]
= [0.736] x 10^3 / [400 x 4.242]
= 0.44 N/mm^2

Direct Axial (Compression / Tension) stress in the Weld = Load / Effective area of weld

R_X = [FX] / [L_w x thickness weld]
= [25.674] x 10^3 / [400 x 4.242]
= 15.13 N/mm^2

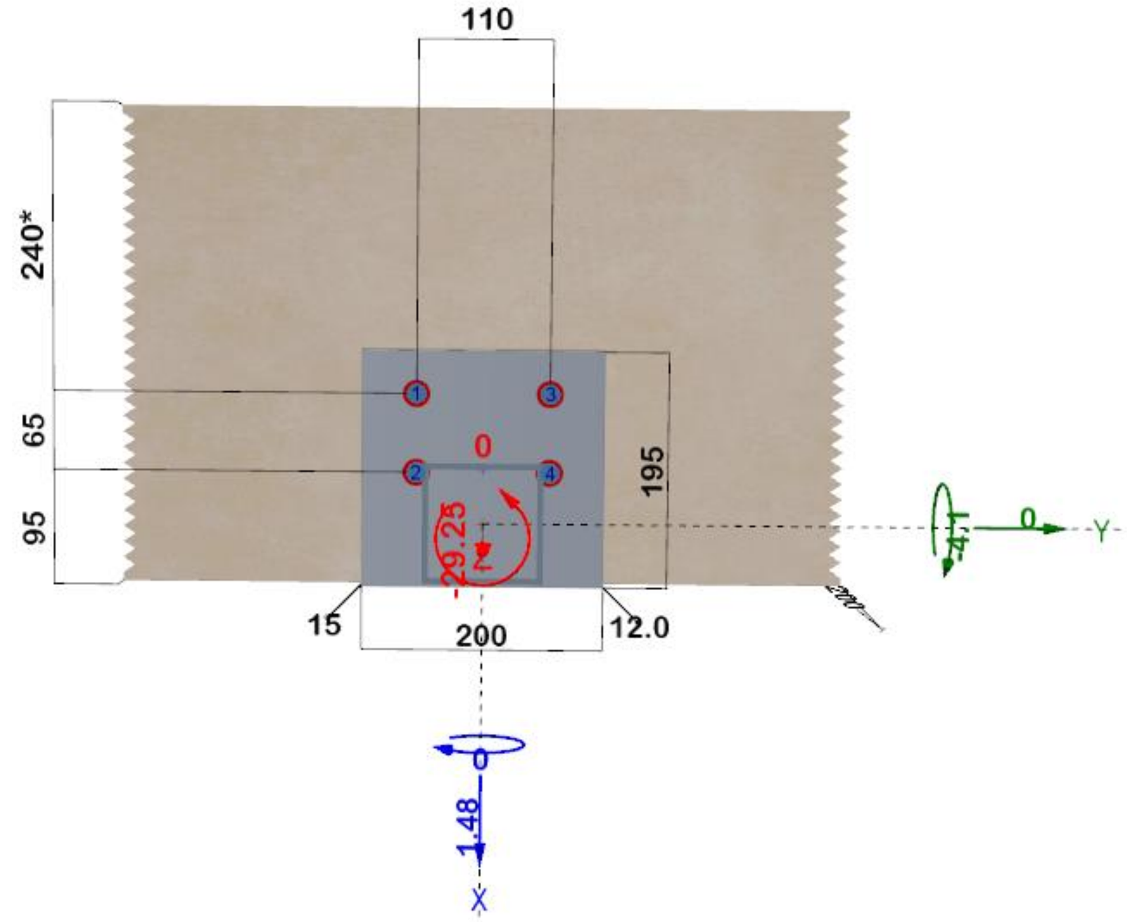
Bending stress in the Weld = Moment / Section Modulus

R_b = (M_y) / Zx x weld thickness
Here, Z_x = (b+d)^3/6 for unit weld length
= (1.03) x 10^6 / [(100+100)^3/6 * 4.242]
= 0.18 N/mm^2

Check for combined bending and shear stress in the Fillet weld,

$f_e = [(R_x+R_b)^2 + 3(R_{yz})^2]^{1/2}$
$= [(15.13+0.18)^2 + 3(0.44)^2]^{1/2}$
= 15.33 N/mm^2 < 233 N/mm² (Hence, OK)

Check for Anchor



Node number 352 reactions for anchor design is:
Fx = 1.48 kN
Fy = 0.00 kN
Fz = 29.25 kN

Moment due to eccentricity = c/c distance of anchor to member center x Fy
= 0.140 x 29.25 = 4.1 kN.m

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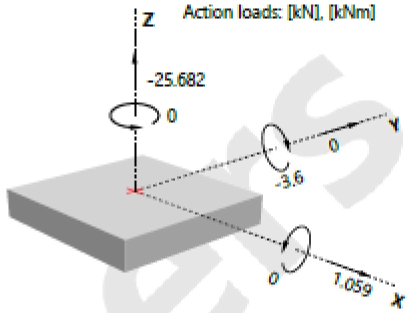
1. Input Data

Selected anchors:

- Allfasteners VF22PRO+ & Threaded Rod Zn 8.8 M16
  Injection anchor Vinylester
  Zinc plated
  Design based on AS 5216
- Assessment ETA-20/0584
  Issued by ZUS, on 8/17/2021
- Effective anchorage depth h_{ef} = 130 mm
- Drilled hole Φ x h_0 = 18.0 x 130 mm



Base material:

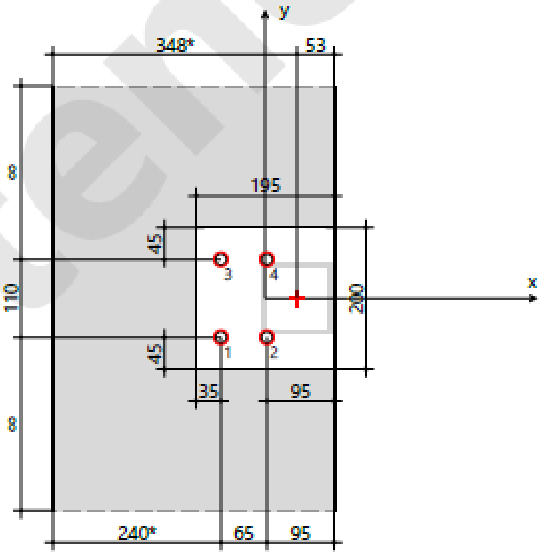
- Cracked concrete, Thickness of base material h=200mm
  Strength class 40MPa, f'_c=40.0N/mm²
- Wide concrete reinforcement
  Rebar spacing a≥150mm for all Ø or a≥100mm for Ø≤10mm
- No edge and stirrup reinforcement
- Long-term temperature 24°C, Short-term temperature 40°C
- Hammer drilled, dry hole

Action loads:

- Predominantly static and quasi-static design loads, α_{sus}=0.6

Installation:

- Stand-off with grouting
  Mortar compressive strength must be higher than 30N/mm².
  Distance=15.0mm, rotational restraint grade=2.0
- With gap filling



Base plate:

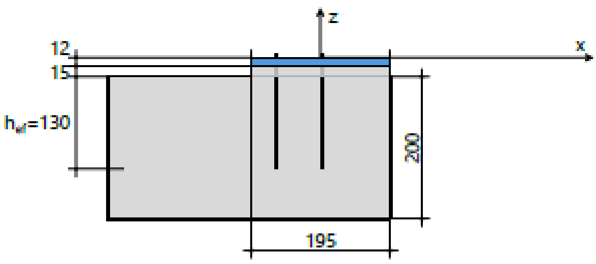
- G250, E=200000N/mm²
  f_y=250N/mm², ϕ_s=0.741, $f_{yd} = \phi_s \cdot f_y$
- Assumed: rigid plate
- Current thickness: 12.0mm
- Required thickness is not calculated.
- Rectangle
  Side length: 195 x 200 mm

Profile:

- Square Hollow Section: 100x5.0 SHS
  H x W x T x FT [mm]: 100 x 100 x 5.0 x 0.0
  Action point [mm]: [45, 0]
  Rotation counterclockwise: 0°

Coordinates of anchors [mm]:

No.	x	y	Slotted hole L-x	Slotted hole L-y
1	-62.5	-55.0		
2	2.5	-55.0		
3	-62.5	55.0		
4	2.5	55.0		



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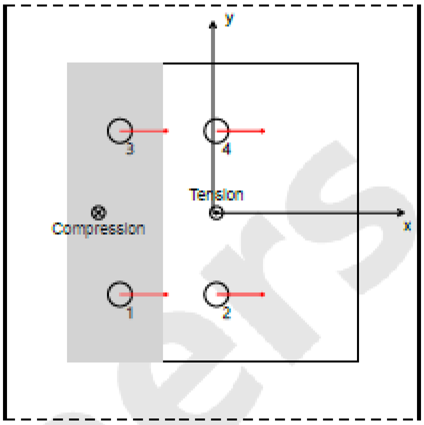
2. Anchor internal forces and verification of base plate bending stiffness

Anchor internal forces [kN]

Anchor No.	Tension N_i	Shear V_i	Shear x	Shear y
1	0.000	0.265	0.265	0.000
2	2.961	0.265	0.265	0.000
3	0.000	0.265	0.265	0.000
4	2.961	0.265	0.265	0.000



Maximum concrete compressive strain [‰]: 0.1586
Maximum concrete compressive stress: 4.76 [N/mm²]
Resultant tension force in (x/y=2.5/0.0): 5.922 [kN]
Resultant compression force in (x/y=-76.9/0.0): 31.604 [kN]
Remark: The edge distance is not to scale.

Conditions of verification:

a) $\sigma \leq f_{yd}$

b) $N^h_r \approx N^h_e$

N^h_r : highest anchor tension force on flexurally rigid base plate

N^h_e : highest anchor tension force on elastic base plate

The proof of the base plate bending stiffness was not carried out.

3. Verification at ultimate limit state based on AS 5216

3.1 Tension load

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure	2,4	2.961	84.000	3.5	√
Combined failure	2,4	5.922	23.506	**25.2**	√
Concrete cone failure	2,4	5.922	32.348	18.3	√
Splitting failure	-	-	-	-	not applicable

Steel failure

$N_{Rd,s} = N_{Rk,s} \cdot \phi_{s,N}$ $\beta_{N,s} = N^* / N_{Rd,s}$

$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}$ [kN]	N^* [kN]	$\beta_{N,s}$
126.0	0.667	84.000	2.961	0.035

Combined pull-out and concrete cone failure

$N_{Rk,Np}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{ec,Np} \cdot \psi_{re,Np}$ $\quad N^0_{Rk,p} = \psi_{sus} \cdot \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $\quad \psi_{A,Np} = A_{p,N}/A^0_{p,N}$ $\quad N_{Rd,Np} = N_{Rk,Np} \cdot \phi_{p,N}$

$s_{cr,Np} = 7.3 \cdot d \cdot (\psi_{sus} \cdot \tau_{Rk,ucr})^{0.5} \leq 3 \cdot l_b$ $\quad \psi_{g,Np} = \psi^0_{g,Np} - (s_m / s_{cr,Np})^{0.5} \cdot (\psi^0_{g,Np} - 1) \geq 1.0$

$\psi^0_{g,Np} = n^{0.5} - (n^{0.5} - 1) \cdot (\tau_{Rk} / \tau_{Rk,c})^{1.5} \geq 1.0$ $\quad \tau_{Rk,c} = k_3 \cdot (h_{ef} \cdot f'_c)^{0.5} / (\pi \cdot d)$ $\quad \psi^0_{sus} = 0.73$ $\quad \alpha_{sus} = 0.6$ $\quad \psi_{sus} = 1.0$

τ_{Rk} [N/mm²]	$\tau_{Rk,ucr}$ [N/mm²]	ψ_c	d [mm]	k_3	f'_c [N/mm²]	h_{ef} [mm]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	l_b [mm]	$\phi_{p,N}$	$\tau_{Rk,c}$ [N/mm²]
5.5	9.0	1.231	16.0	7.7	40	130.0	350.4	175.2	130.0	0.556	11.046

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi_{s,Np}$	c_{min} [mm]
44.242	124335	122780	1.013	0.863	95.0

n	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	$\psi_{re,Np}$	$e_{Np,x}$ [mm]	$e_{Np,y}$ [mm]	$\psi_{ec,Np,x}$	$\psi_{ec,Np,y}$	$\psi_{ec,Np}$	$N_{Rk,Np}$ [kN]	$N_{Rd,Np}$ [kN]	N* [kN]	$\beta_{N,p}$
2	1.215	110.0	1.095	1.0	0.0	0.0	1.000	1.000	1.000	42.311	23.506	5.922	0.252

Concrete cone failure

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}$ $\quad N^0_{Rk,c} = k_1 \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$ [N] $\quad \psi_{A,N} = A_{c,N}/A^0_{c,N}$ $\quad N_{Rd,c} = N_{Rk,c} \cdot \phi_{c,N}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	k_1	$\phi_{c,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
72.183	145000	152100	0.953	7.7	0.556	130.0	390.0	195.0

$\psi_{s,N}$	$\psi_{re,N}$	$e_{N,x}$ [mm]	$e_{N,y}$ [mm]	$\psi_{ec,N,x}$	$\psi_{ec,N,y}$	$\psi_{ec,N}$	$\psi_{M,N}$	$N_{Rk,c}$ [kN]	$N_{Rd,c}$ [kN]	N* [kN]	$\beta_{N,c}$
0.846	1.0	0.0	0.0	1.0	1.0	1.0	1.0	58.227	32.348	5.922	0.183

Splitting

Verification of splitting failure is not necessary, because:

- The calculations of resistances at concrete cone failure and pull-out failure were conducted for cracked concrete.
- The crack width is limited to 0.3mm.

3.2 Shear

	Related anchor	Action [kN]	Resistance [kN]	Utilization [%]	Status
Steel failure (with l. arm)	2,4	0.265	14.159	1.9	√
Pry-out	1,2,3,4	1.059	60.605	1.7	√
Concrete edge failure (x+)	1,2,3,4	1.059	21.370	**5.0**	√

Steel failure with lever arm

$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ $\quad M_{Rk,s}=M^0_{Rk,s}\,(1-|N^*|/N_{Rd,s})$ $\quad V_{Rd,s}=V_{Rk,s} \cdot \phi_{s,V}$ $\quad \beta_{V,s}=V^*/V_{Rd,s}$

$M^0_{Rk,s}$ [Nm]	$N_{Rk,s}$ [kN]	$\phi_{s,N}$	$N_{Rd,s}=N_{Rk,s} \cdot \phi_{s,N}$ [kN]	α_M	e_1 [mm]	a_3 [mm]	$l=a_3+e_1$ [mm]	$\phi_{s,V}$
266.0	126.0	0.667	84.000	2.0	21.0	8.0	29.0	0.8

N* [kN]	$M_{Rk,s}=M^0_{Rk,s}\,(1-\|N^*\|/N_{Rd,s})$ [Nm]	$V_{Rk,s}=\alpha_M \cdot M_{Rk,s}/l$ [kN]	$V_{Rd,s}$ [kN]	V* [kN]	$\beta_{V,s}$
2.961	256.624	17.698	14.159	0.265	0.019

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Pry-out failure ($N_{Rk,p}$ Decisive)

$N_{Rk,p}=N^0_{Rk,p} \cdot \psi_{A,Np} \cdot \psi_{s,Np} \cdot \psi_{g,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,p} = \pi \cdot d \cdot l_b \cdot \tau_{Rk} \cdot \psi_c$ [N] $V_{Rk,cp}=k_8 \cdot N_{Rk,p}$ $V_{Rd,cp} =V_{Rk,cp} \cdot \phi_{cp,V}$

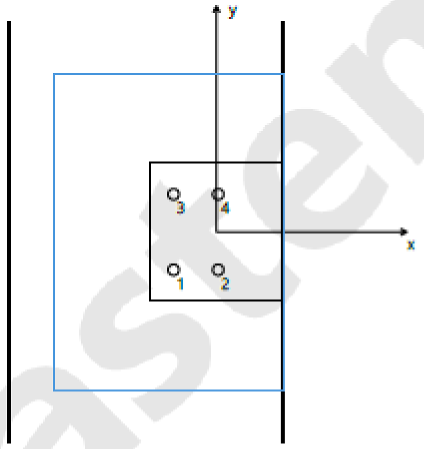
For stand-off installation (overturning moment): $V_{Rd,cp} =V_{Rk,cp} \cdot \alpha_h \cdot \phi_{cp,V}$ $\alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	$\tau_{Rk,ucr}$ [N/mm²]	$s_{cr,Np}$ [mm]	$c_{cr,Np}$ [mm]	d [mm]	l_b [mm]	τ_{Rk} [N/mm²]	ψ_c	k_8	$\phi_{cp,V}$
130.0	9.0	350.4	175.2	16.0	130.0	5.5	1.231	2.0	0.667

$N^0_{Rk,p}$ [kN]	$A_{p,N}$ [mm²]	$A^0_{p,N}$ [mm²]	$\psi_{A,Np}$	$\psi^0_{g,Np}$	s_m [mm]	$\psi_{g,Np}$	ψ_{sus}
44.242	154268	122780	1.256	1.52	87.5	1.26	

$\psi_{s,Np}$	$\psi_{re,Np}$	$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,p}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V* [kN]	$\beta_{V,cp}$
0.863	1.0	0.0	0.0	1.0	1.0	1.0	60.433	120.866	60.605	1.059	0.017

Related area for calculation of pry-out failure $A_{p,N}$:



Concrete edge failure, direction x+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} =k_9 \cdot d^{\alpha} \cdot l_f^{\beta} \cdot (f'_c)^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} =A_{c,V}/A^0_{c,V}$ $V_{Rd,c} =V_{Rk,c} \cdot \phi_{c,V}$

$l_f = \min(h_{ef}, 12d)$ $\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

For stand-off installation (overturning moment): $V_{Rd,c} =V_{Rk,c} \cdot \alpha_h \cdot \phi_{c,V}$ $\alpha_h =(h_h - a_3) / (e_1 + h_h)= 0.752$ $h_h = \min(h_{ef}, 6d)$

h_{ef} [mm]	k_9	f'_c [N/mm²]	$\phi_{c,V}$	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d [mm]	l_f [mm]
130.0	1.7	40	0.667	160.0	-	0.090	0.063	37.982	1.000	16.0	130.0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V* [kN]	$\beta_{V,c}$
118000	115200	1.024	1.095	1.000	0.0	1.000	1.000	42.618	21.370	1.059	0.050

3.3 Combined tension and shear

	Anchor	Tension(β_N)	Shear(β_V)	Condition	Utilization [%]	Status
Steel	-	-	-	$\beta^2_N + \beta^2_V \leq 1.0$	-	not applicable
Concrete	2,4	0.252	0.050	$\beta^{1.5}_N + \beta^{1.5}_V \leq 1.0$	13.7	√

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Anchor-related utilization

A-No.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,sp}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{N,c,max,t}$	$\beta_{V,c,max,t}$	$\beta_{combi,c,t}$	$\beta_{combi,s,t}$
1	0.000	0.000	0.000	0.000	0.018	0.017	0.050	0.000	0.050	0.011	-
2	0.035	0.252	0.183	0.000	0.019	0.017	0.050	0.252	0.050	0.137	-
3	0.000	0.000	0.000	0.000	0.018	0.017	0.050	0.000	0.050	0.011	-
4	0.035	0.252	0.183	0.000	0.019	0.017	0.050	0.252	0.050	0.137	-

$\beta_{N,c,max,t}$: Highest utilization of individual anchors under tension loading except steel failure
$\beta_{V,c,max,t}$: Highest utilization of individual anchors under shear loading except steel failure
$\beta_{combi,c,t}$: Utilization of individual anchors under combined tension and shear loading except steel failure
$\beta_{combi,s,t}$: Utilization of individual anchors under combined tension and shear loading at steel failure

4. Displacement

Tension loading: $\tau^{*h} = N^{*h} / (\pi \cdot d \cdot l_b)$

Short-term displacement: $\delta_N^{\,0} = (\delta_{N0} \cdot \tau^{*h}) / 1.4$

Long-term displacement: $\delta_N^{\,\infty} = (\delta_{N\infty} \cdot \tau^{*h}) / 1.4$

Shear loading: $V_k^{\,h} = V^{*h} / 1.4$

Short-term displacement: $\delta_V^{\,0} = V_k^{\,h} \cdot \delta_{V0}$

Long-term displacement: $\delta_V^{\,\infty} = V_k^{\,h} \cdot \delta_{V\infty}$

N^{*h} [kN]	τ^{*h} [N/mm²]	δ_{N0} [mm³/N]	$\delta_{N\infty}$ [mm³/N]	$\delta_N^{\,0}$ [mm]	$\delta_N^{\,\infty}$ [mm]
2.961	0.453	0.050	0.180	**0.016**	**0.058**

V^{*h} [kN]	$V_k^{\,h}$ [kN]	δ_{V0} [mm/kN]	$\delta_{V\infty}$ [mm/kN]	$\delta_V^{\,0}$ [mm]	$\delta_V^{\,\infty}$ [mm]
0.265	0.189	0.110	0.170	**0.021**	**0.032**

5. Remarks

- Capacity verifications of Section 3 are in accordance with AS 5216. For more complex cases which are outside of AS 5216, the same principles of AS 5216 are still used.
- For connections with a flexurally rigid base plate, it is assumed that the base plate is sufficiently rigid. However, the current anchor design methods (ETAG, Eurocode, AS 5216, ACI 318, CSA A23.3) do not provide any usable guidance to check for rigidity. In the realistically elastic (flexible) base plate, the tension load distribution between anchors may be different to that in the assumed rigid base plate. The plate prying effects could further increase anchor tension loading. To verify the sufficient base plate bending rigidity, the stiffness condition according to the publication "Required Thickness of Flexurally Rigid Base plate for Anchor Fastenings" (fib Symposium 2017 Maastricht) is used in this software.
- For connections with an elastic base plate, the anchor tension forces are calculated with the finite element method with consideration of deformations of base plate, anchors and concrete. Background for design with elastic base plates is described in the paper "Design of Anchor Fastenings with Elastic Base Plates Subjected to Tension and Bending". This paper was published in "Stahlbau 88 (2019), Heft 8" and "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017".
  Anchor shear forces are calculated with the assumption of a rigid base plate. Attention should be paid to a narrow base plate with a width to length ratio of less than 1/3.
- Verification for the ultimate limit state and the calculated displacement under service working load are valid only if the anchors are installed properly according to ETA.
- For design in cracked concrete, anchor design standards/codes assume that the crack width is limited to ≤ 0.3mm by reinforcement. Splitting failure in cracked concrete is prevented by this reinforcing. The user needs to verify that this reinforcing is present in cracked concrete. Generally, concrete structures design standards/codes (e.g. AS 3600) meet this crack width requirement for most structures. Particular caution must be taken at close edge distances where the location of reinforcing is not clearly known.
- Verification of strength of concrete elements to loads applied by fasteners is to be done in accordance with AS 5216.
- All information in this report is for use of Allfasteners products only. It is the responsibility of the user to ensure that the latest version of the software is used, and in accordance with AFOS licensing agreement. This software serves only as an aid to interpret the standards and approvals without any guarantee to the absence of errors. The results of the software should be checked by a suitably qualified person for correctness and relevance of the results for the application.

The load-bearing capacity of the anchorage is: **verified !**

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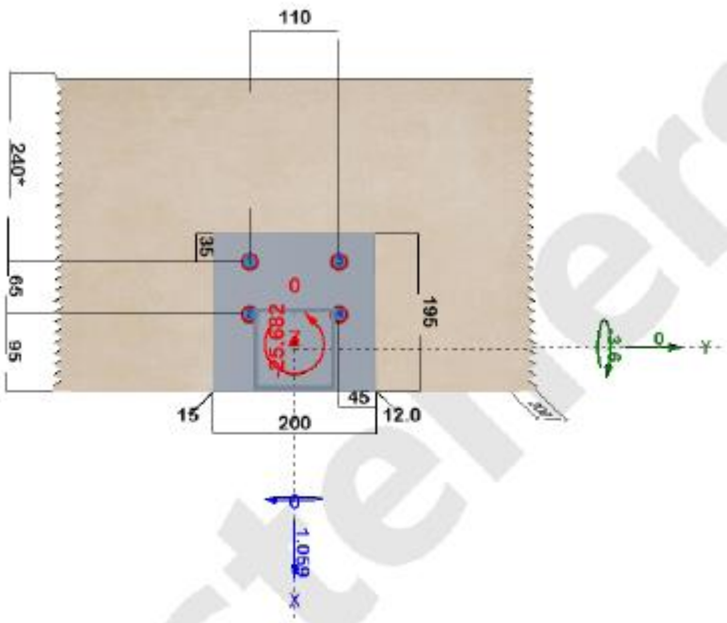
Anchorage figure in 3D:



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Anchor: **VF22PRO+ & Threaded Rod Zn 8.8 M16**

Drilled hole: $d_0 \times h_0$ = 18 x 130 mm
Embedment depth: h_{nom} = 130 mm
Effective anchorage depth: h_{ef} = 130 mm
Installation torque: T_{inst} = 80 Nm

Base plate: **G250**

Thickness: t = 12 mm
Clearance hole: d_f = 18 mm