ENGINEERING CONSULTANTS LIMITED

Development
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Tauranga

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The Building is Single story Building with tilt panels on one long side and steel framing for the remainder of the building.

- **Gravity Load**

  Cold Formed purlins spanning between the external concrete panel walls and mid span steel beams.

  Mid span steel beams supported on rafter. Rafter supported on vertical post on front & rear side of the building.

  Thus total loads will be transferred to the ground via the concrete panel walls and vertical post on periphery of the building.

- **Lateral Load**

  The Roof bracing will be provided in the form of X – Bracing to create a roof diaphragm for across direction lateral forces.

  Along direction

  Lateral load resistance will be provided by the tilt panel wall on the North East elevation between grid 1 to 5 and South West elevation between grid 1 & 2.

  Across direction

  Lateral Load resistance on across direction will be provided by portal frames on Grid 1 to 6.

  Analysis ofSteel Structures is done using STAAD-PRO 2007 Structural Analysis Software.

- **Design Code**

BS –PART	YEAR	TITLE
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	2011	Structural Design actions Part 2: Wind actions
AS/NZS 1170.5	2004	Structural Design actions Part 5: Earthquake actions
NZS 3101: Part 1: 2006	2006	Concrete Structures Standard Part 1: The Design of Concrete Structures
NZS 3404: Part 1& 2: 1997	1997	Steel Structures Standard Part 1 & Part 2: The Design of Steel Structures

- **FOUNDATION: -**

  S&L Consultants Limited (S& L) has Submitted geotechnical Investigation Report.

The Proposed foundations should bear at or below the level of adjacent foundations to prevent structural loading from influencing the neighboring building. If the proposed foundations are to bear at a level below adjacent foundations, proposed foundations and walls should be designed to resist loading from the neighboring building.

As per Geotechnical Investigation Report, **ultimate bearing capacity of 300 kPa with corresponding allowable bearing capacity of 100kPa will be adopted for the detailing of foundations**. Foundation excavations should be clean of any loose or excessively wet material and should be free of standing water at the time of concrete placement.

- **MATERIAL PROPERTIES**

  Structural Steel

  Design yield strength fy= 300 N/mm^2

  Connection Bolts

  All Bolts To be Grade 8.8 IN S300.

  Concrete Grade

  Compressive Strength of Concrete = 30 N/mm2

  Rebar Grade

  Yield stress of reinforcement = 300 N/mm2 D
  (For Small Members & Stirrups)
  Yield stress of reinforcement = 500 N/mm2 HD
  (For Longitudinal steel in ground Beams)

- **Roof Dead Load:-**

  Self-weight of Main steel Members = 0.07 kN/m2
  Purlin = 0.05 kN/m2
  Cladding = 0.05 kN/m2
  Suspended Celling = 0.07 kN/m2

Insulation = 0.02 kN/m2
Battens = 0.02 kN/m2
Services = 0.025 kN/m2
Total Roof Dead Load = 0.305kN/m2

- **Wall Load :-**
  Timber Framed wall = 0.35 kN/m2
  Glazed wall = 0.35 kN/m2
  150 mm Concrete = 0.15 x 24 = 3.6kN/m2

- **Roof Imposed Load :-**
  Roof Imposed Load = 0.25 kN/m2 (Table 3.2 NZS1170 Part 1)

Seismic Load as per NZS1170 Part 5 :-

Elastic Site Spectra for Horizontal Loading :

Elastic site hazard spectrum for Horizontal Loading C(T) :

C(T) = Ch (T) Z R N (T,D)

Where,

Ch (T) =	The spectral shape factor Cl. 3.1.2 NZS 1170 Part 5 (Figure 3.1)	
Subsoil Classification is C		
T < 0.4	Sec	
Ch (T) =	2.36	
Z =	The hazard factor determind from Clause 3.1.4 NZS 1170 Part 5 (Table 3.3)	
Z =	0.2	for (Tauranga)
Importance Level =	2	
Design Working Life =	50	years
Annual Probabality of Exceedance =	1/500	(Table 3.3 AS/NZS 1170.0:2002)
R =	The return period factor Rs and Ru for the appropriate limit state determined from clause 3.1.5 NZS 1170 Part 5 but limited such that ZRu does not exceed 0.7	
Rs =	1	(Serviceability Limit State)
Ru =	0.25	(Ultimate Limit State)
Near-Fault factor N (T,D) =	1	Clause 3.1.6 NZS1170 Part 5

Elastic site hazard spectrum for Horizontal Loading C(T) :

C(T) = Ch (T) Z R N (T,D)

C(T) (Serviceability Limit State) = 2.36 X 0.2 X 1 X 1

= 0.472

C(T) (Ultimate Limit State) = 2.36 X 0.2 X 1 X 0.25

= 0.118

Horizontal Design action coefficients and Design Spectra :-

Ultimate Limit State :-

μ (Ductility Factor) = 1.25 (Assumed)

For soil Classes A ,B ,C and D

	$K\mu =$	μ	for $T1 \geq$	0.7	s
	$=$	$((\mu - 1)T1 / 0.7) + 1$	for $T1 <$	0.7	s

$Cd(T1) = C(T1)Sp / K\mu$

$\geq (Z/20 + 0.02)Ru$ but not less than $0.03Ru$

Sp = Structural performance factor clause 12.2.2 NZS3404 Part2

= 0.9

$K\mu =$ (1.25- 1) x 0.4/0.7)+1

= 1.143

Cd (T1) = (0.472X0.9)/1.143

= 0.372000

<u>Serviceability Limit State :-</u>

μ (Ductility Factor) = 1 (Assume)

For soil Classes A ,B ,C and D

	$K\mu =$	μ	for $T1 \geq$	0.7	s
	$=$	$((\mu - 1)T1 / 0.7) + 1$	for $T1 <$	0.7	s

$Cd(T) = C(T)Sp / K\mu$

Sp = Structural performance factor clause 12.2.2 NZS3404 Part2

Sp = 0.7

$K\mu =$ (1- 1) x 0.4/0.7)+1

= 1

Cd (T1) = (0.118X0.7)/1

= 0.0826

<u>Horizontal Design Action Coefficients :</u>

Cds/Cdn = 0.0826/0.372

= 0.223

Wind Load as per NZS1170 Part 2 :-

Regional Wind Speede :

V500 =	45	m/s	Table 3.1 NZS1170 Part 2
V25 =	37	m/s	Table 3.1 NZS1170 Part 2
Md =	1	Assume Clause 3.3.2 NZS1170 Part 2	
Purlin Spacing	1.2	m	

Assume Terrain Category 2.5 Clause 4.3.1 NZS1170 Part 2

Height of building,h 5.8 m

Determination of terrain/height multiplier (Mz,ca 0.928+0.83/2

= 0.879

Table 4.1 NZS 1170 Part 2

Ms =	Shielding multiplie	1	Clausew 4.3.1 NZS1170 Part 2		
Mt=	Topographic multiplier =		1	=	Mh=Mle

Site Wind Speed

Vsit,β =	Vr X Md X (Mz,cal X Ms X Mt)
V500 =	45X1X0.879X1X1
=	39.555
V25 =	37X1X0.879X1X1
=	32.523

Wh =	0.6 X 1564.598025		
=	0.939	kN/m2	(ULS)
Ws =	0.6 X 1057.745529		
=	0.635	kN/m2	(SLS)

Internal Pressure Coefficients Cpi= 0.2 or -0.3

(Table 5.1(B) NZS1170 Part 2)

External Pressure Coefficients (Clasuse 5.4 NZS1170 Part 2)

d =	25.94	m
b =	10.43	m

Calading Wall Coefficients (Along)

d/b= 2.488

Windward Cpe =	0.7		Table 5.2 (A) NZS1170 Part 2
Leeward Cpe =	-0.275		Table 5.2 (B) NZS1170 Part 2
Side wall Cpe =	-0.65	(0 to 5.8m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.5	(5.8m to 11.6m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.3	(11.6m to 17.4m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.2	(> 17.4m)	Table 5.2 (C) NZS1170 Part 2

Calading Wall Coefficients (Across)

b/d =	0.403		
Windward Cpe =	0.7		Table 5.2 (A) NZS1170 Part 2
Leeward Cpe =	-0.5	(ϴ = 90◦)	Table 5.2 (B) NZS1170 Part 2
Side wall Cpe =	-0.65	(0 to 5.8m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.5	(5.8m to 11.6m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.3	(11.6m to 17.4m)	Table 5.2 (C) NZS1170 Part 2
Side wall Cpe =	-0.2	(> 17.4m)	Table 5.2 (C) NZS1170 Part 2

Roof (Table 5.3(A) NZS1170 Part 2

α <	10°		
d =	25.94	m	
b =	10.43	m	
h =	5.8	m	
Along			
h/d =	0.224	Upwind	Downwind
0 to 2.9 Cpi		-0.9	-0.4
2.9 to 5.8 Cpi		-0.9	-0.4
5.8 to 11.6 Cpi		-0.5	0
11.6 to 17.4 Cpi		-0.3	0.1
> 17.4 Cpi		-0.2	0.2

Across			
h/b =	0.556088	Gable Roof	
0 to 2.9 Cpi		-0.945	-0.4
2.9 to 5.8 Cpi		-0.945	-0.4
5.8 to 11.6 Cpi		-0.522	0
11.6 to 17.4 Cpi		-0.300	0.1
> 17.4 m Cpi =		-0.200	0.2

WIND + X (CPE + CPI) (ALONG)

Wall	-0.45		-0.3		-0.1		0	
Pressure	-0.42255		-0.2817		-0.0939		0	
	Roof							
0.9	-0.7		-0.3		-0.1		0	-0.075
0.8451	-0.66		-0.28		-0.09		0.00	-0.07043
Wind X			Slope					
	(0 to 5.8m		(5.8m to 11.6m)		(11.6m to 17.4m)		(> 17.4m)	

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	For: Design Philosophy Report				
By:	Date: 16/1/2016	Checked By:	Checked Date:	Page No:	Rev:

WIND + X (CPE - CPI) (ALONG)

Wall	-0.95	-0.8	-0.6	-0.5	
Pressure	-0.89205	-0.7512	-0.5634	-0.4695	
	Roof				
0.4	-1.2	-0.8	-0.6	-0.5	-0.575
0.3756	-1.13	-0.75	-0.56	-0.47	-0.53993
Wind X		Slope			
	(0 to 5.8m	(5.8m to 11.6m)	(11.6m to 17.4m)	(> 17.4m)	

WIND - X (CPE + CPI) (ALONG)

Wall	0	-0.1	-0.3	-0.45	
Pressure	0	-0.0939	-0.2817	-0.42255	
	Roof				
-0.075	0.4	0.3	0.2	-0.2	0.9
-0.07043	0.38	0.28	0.19	-0.19	0.8451
		Slope			Wind -X
	(> 17.4m)	(11.6m to 17.4m)	(5.8m to 11.6m)	(0 to 5.8m)	

WIND-X (CPE - CPI) (ALONG)

Wall	-0.5	-0.6	-0.8	-0.95	
Pressure	-0.4695	-0.5634	-0.7512	-0.89205	
	Roof				
-0.575	-0.1	-0.2	-0.3	-0.7	0.4
-0.53993	-0.09	-0.19	-0.28	-0.66	0.3756
		Slope			Wind -X
	(> 17.4m)	(11.6m to 17.4m)	(5.8m to 11.6m)	(0 to 5.8m)	

WIND Z (CPE + CPI)(ACROSS)

Wall	-0.45	-0.3	
Pressure	-0.42255	-0.2817	
	Roof		
0.9	-0.74487	-0.32244	-0.3
0.8451	-0.70	-0.30	-0.2817
Wind Z			
	(0 to 5.8m	(5.8m to 11.6m)	

WIND Z (CPE - CPI)(ACROSS)

Wall	-0.95	-0.8	
Pressure	-0.89205	-0.7512	
	Roof		
0.4	-1.24487	-0.822	-0.8
0.3756	-1.17	-0.77	-0.7512
Wind Z			
	(0 to 5.8m	(5.8m to 11.6m)	

WIND -Z (CPE + CPI)(ACROSS)

Wall	-0.3		-0.45	
Pressure	-0.2817		-0.42255	
	Roof			
-0.3	-0.32244		-0.74487	0.9
-0.2817	-0.30		-0.70	0.8451

Wind -Z

(5.8m to 11.6m) (0 to 5.8m)

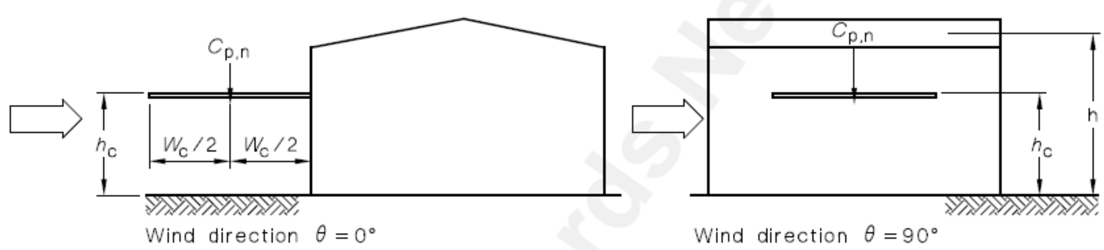
WIND -Z (CPE - CPI)(ACROSS)

Wall	-0.8		-0.95	
Pressure	-0.7512		-0.89205	
	Roof			
-0.8	-0.82244		-1.24487	0.4
-0.7512	-0.77		-1.17	0.3756

Wind -Z

(5.8m to 11.6m) (0 to 5.8m)

Wind Load Calculation (Table D8) (AS/NZS 1170.2:2011)



(a) Open canopy or awning

h_c = 3.5 m
h = 5.25 m
hc/h = 0.666

For ϴ = 0° (Table D8) (AS/NZS 1170.2:2011)

For hc/h = 0.5 Net pressure coefficients ($C_{p,n}$) = 0.5 , -0.3

For hc/h = 0.75 Net pressure coefficients ($C_{p,n}$) = 0.4 , -0.3 – 0.2(3.5/2.0) = -0.65

For hc/h = 0.666 Net pressure coefficients ($C_{p,n}$) = 0.433 , -0.532

For ϴ = 90° (Table D4A& D4B) (AS/NZS 1170.2:2011)

Net pressure coefficients ($C_{p,n}$) = 0.4 , -0.3 for (0 to 10.5 m)

Net pressure coefficients (Cp,n) = 0.2, -0.2 for (> 10.5 m)

The Aerodynamic Shape Factor = C_{fig} = $C_{p,n}K_a K_l$

Wh =	**0.6 X 1564.598025**		
	0.939	**kN/m2**	**(ULS)**
Ws =	**0.6 X 1057.745529**		
	0.635	**kN/m2**	**(SLS)**

- **<u>Seismic Load:</u>**

Dead Load of Roof:

Part – A:-

Width of Building, B = 10.010 m

Length of Building, D = 25.275 m

Total Dead load of Roof = 0.305 kN/m2

Total Dead load of Roof = 0.305 x 10.01 x 25.275

= 77.165 kN

Part – B:-

Length of Building= 25.275 m + 5.69 m

Height of building = 5.0 m

150 mm Concrete tilt wall

Considering half weight of wall half for Seismic Contribution =

Seismic Weight = 0.15 x 5.0 x (25.275 + 5.69) x 24 (density of Concrete) / 2 = 278.685kN

Part – C:-

Glazed wall load at canopy Level = 2.5m (Height) x 0.35kN/m2 (Unit Weight) x 20.7 m (Length) Glazed wall load at canopy Level = 18.11kN

Part – D:-

Weight of Canopy = 2m (Height) x 0.305kN/m2 (Unit Weight) x 20.7 m (Length)

Weight of Canopy = 12.627kN

Total Vertical Load on Roof Level = Part A +Part B +Part C +Part D

Total Vertical Load on Roof Level = 77.165 + 278.685 + 12.627+18.11 = 386.587Kn

Elastic site hazard spectrum for horizontal loading:

Cd (T1) = 0.372 (Ultimate Limit State)

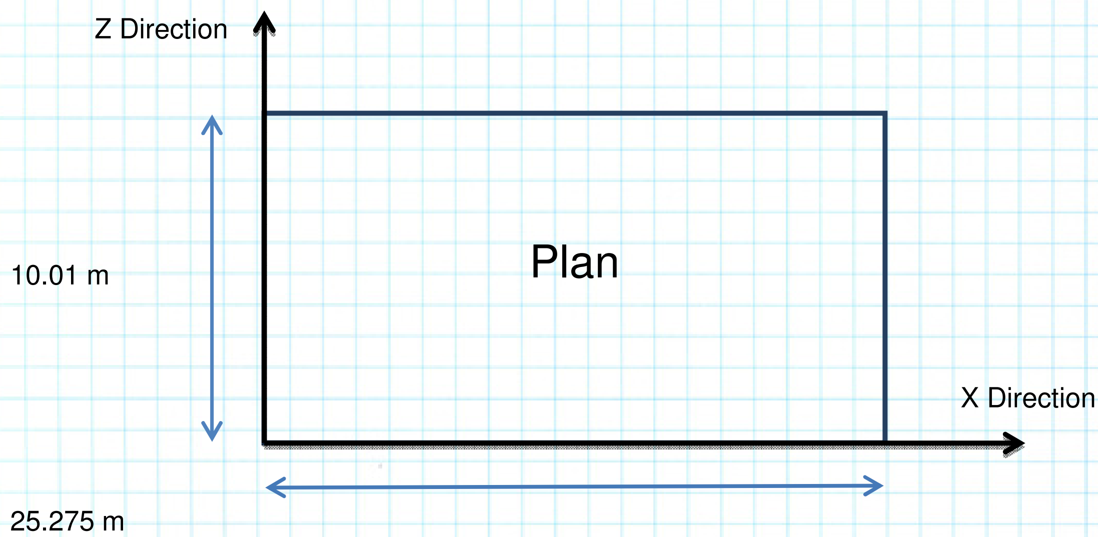
Cd (T1) = 0.0826 (Serviceability Limit States)

Total seismic Force on Frame (Ultimate Limit State) = 386.587 x 0.372 / (6) = 23.97kN

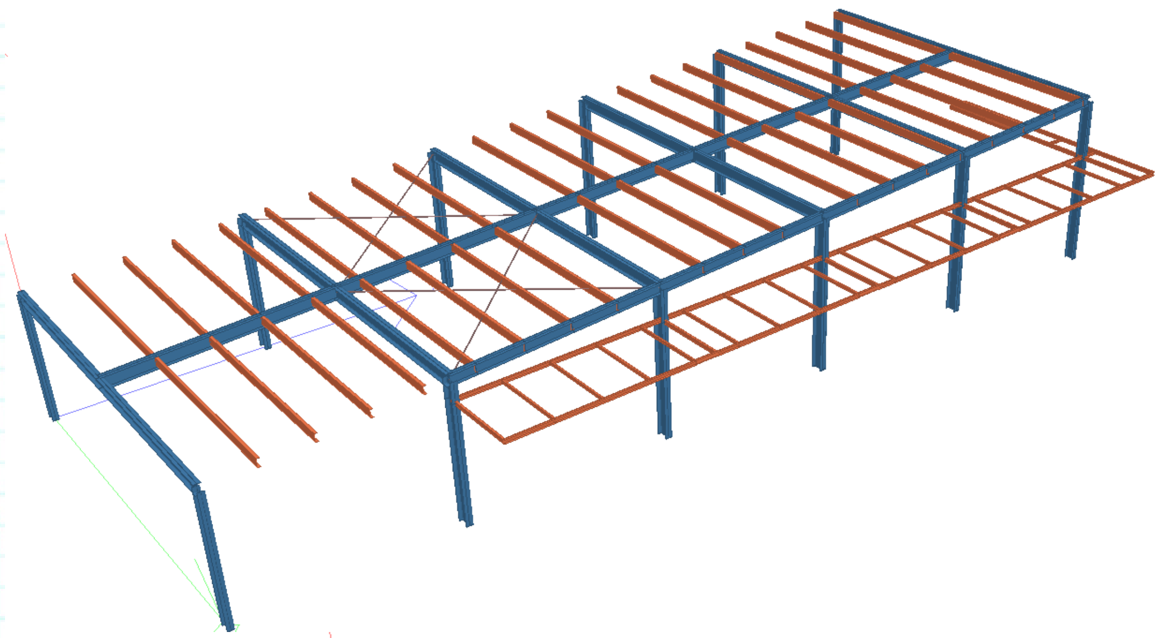
Total seismic Force on Frame (Serviceability Limit States) = 386.587x0.0826 / (6) = 5.322kN

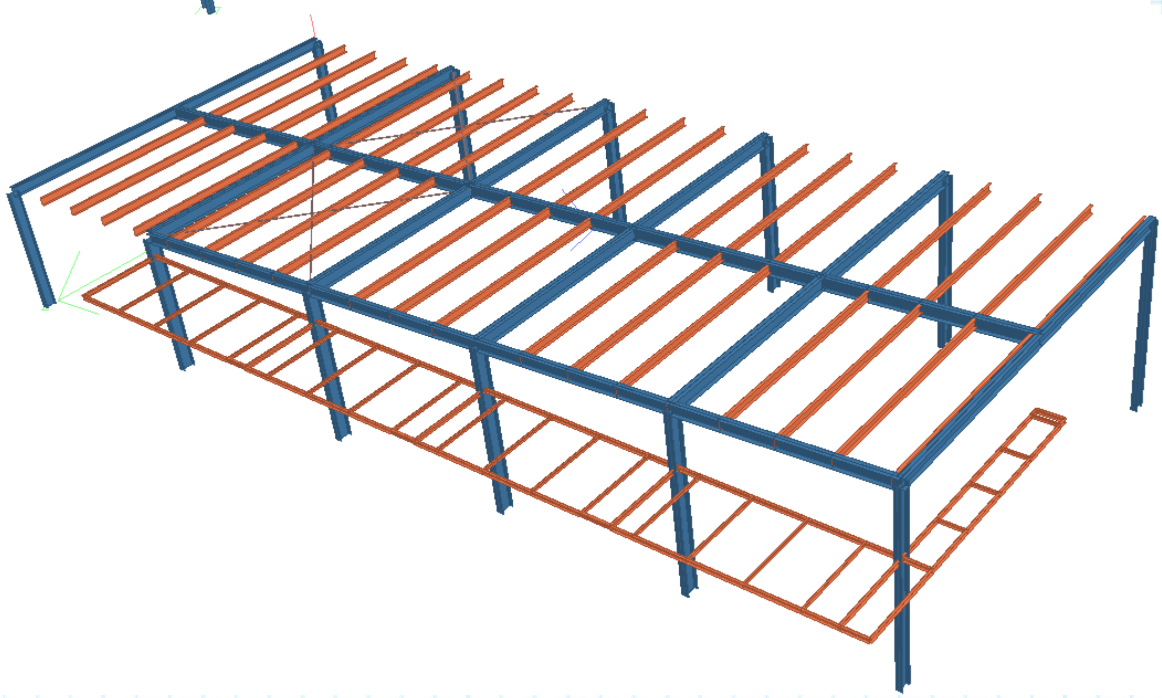
Note: - Seismic Load from X & -X Direction will be taken care by tilt Panels.

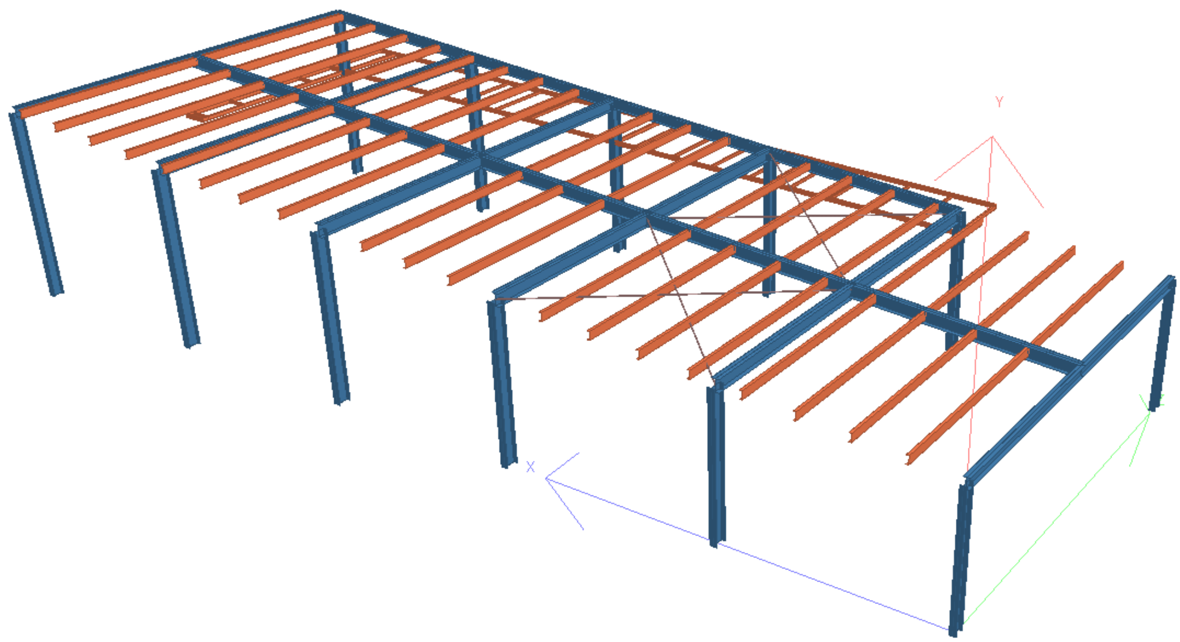
Seismic Load from Z & -Z Direction will be taken care by Moment Frames.

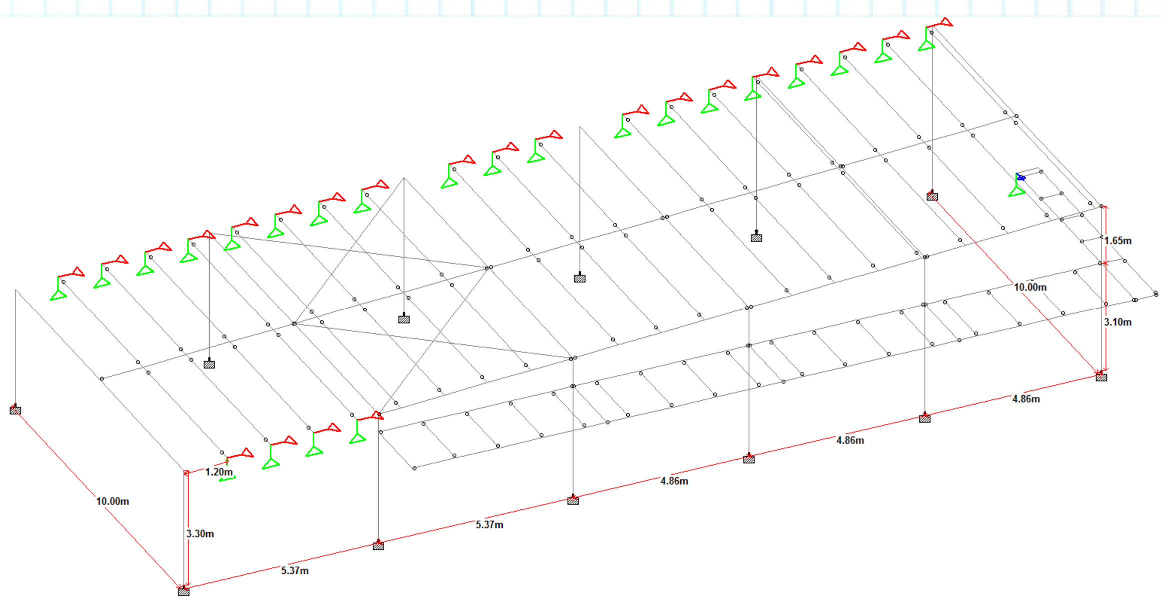


DESIGN OF THE MAIN STRUCTURE
GEOMETRICAL DATA (3D VIEW)

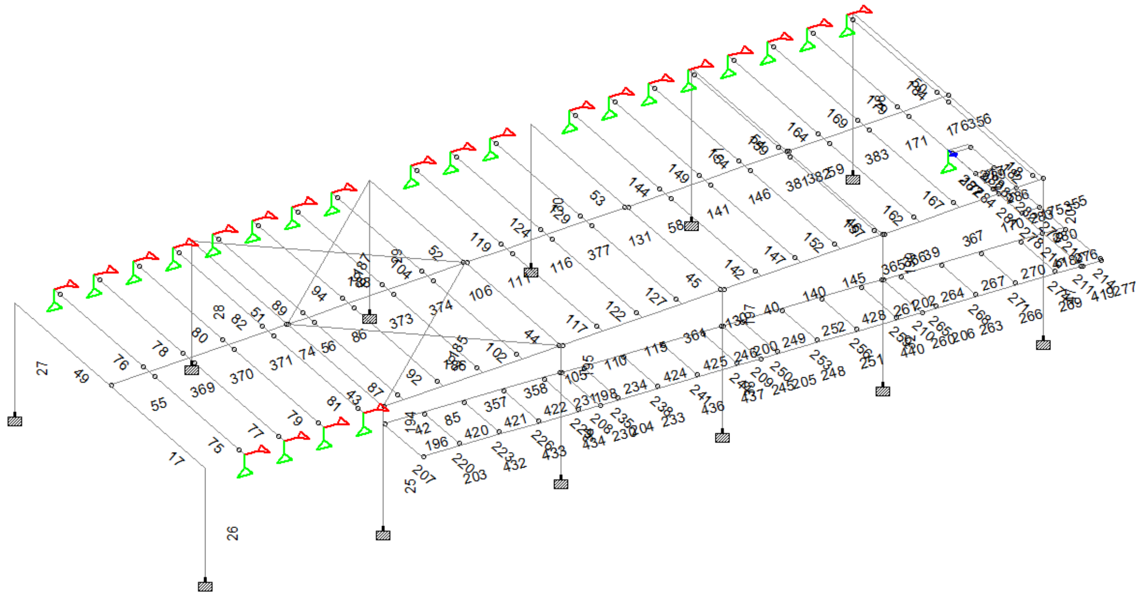








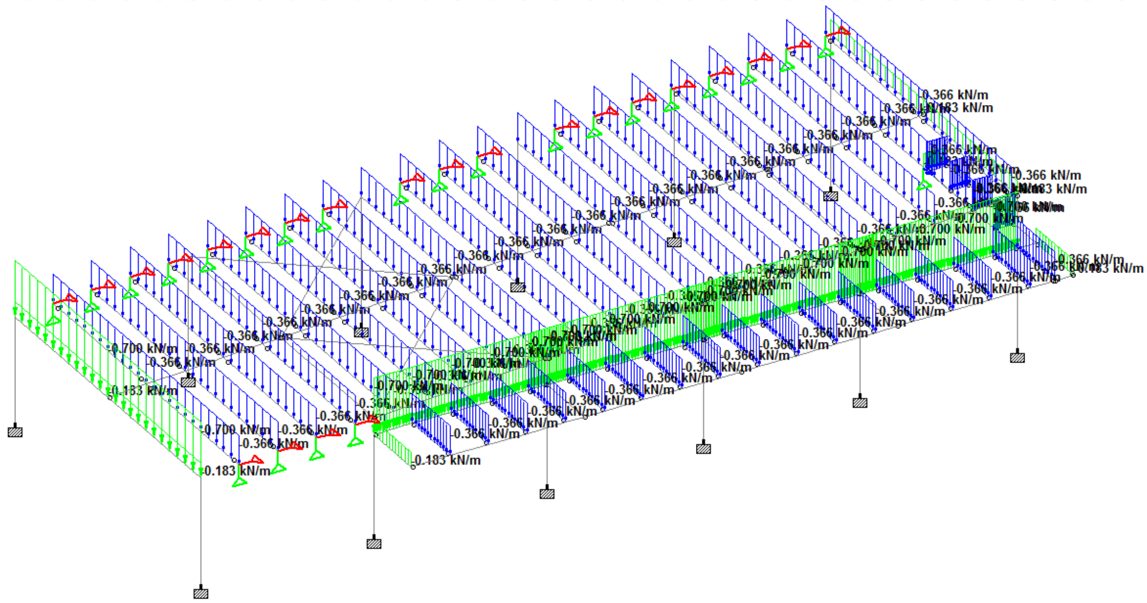
Beam No. & Node No. :-



Support Condition
All column supports are considered as Fixed.

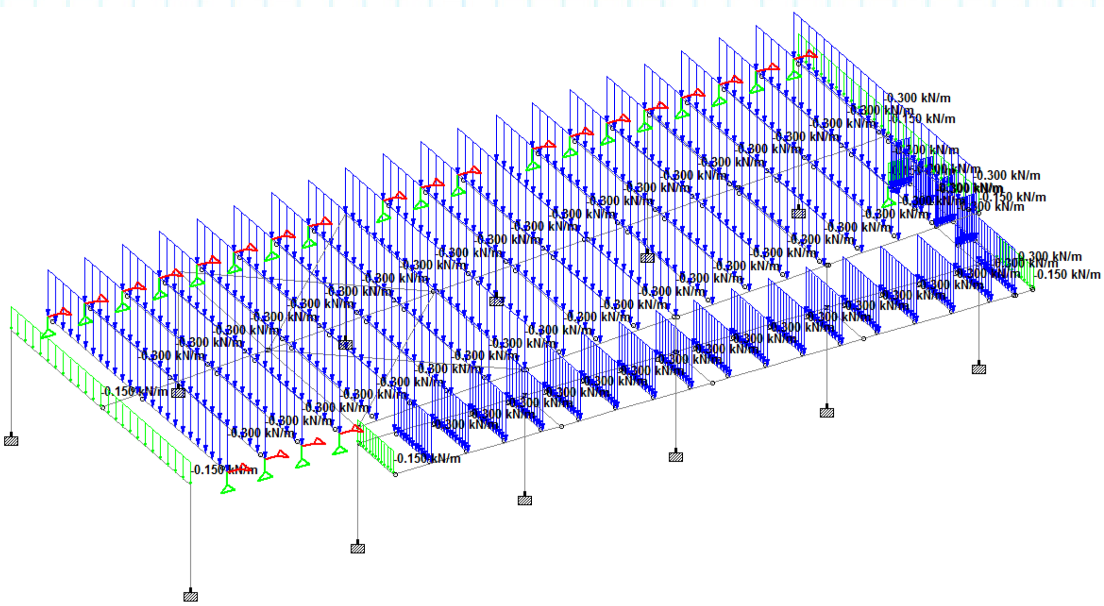
Loading Data

Dead load



Total dead load is assigned to members

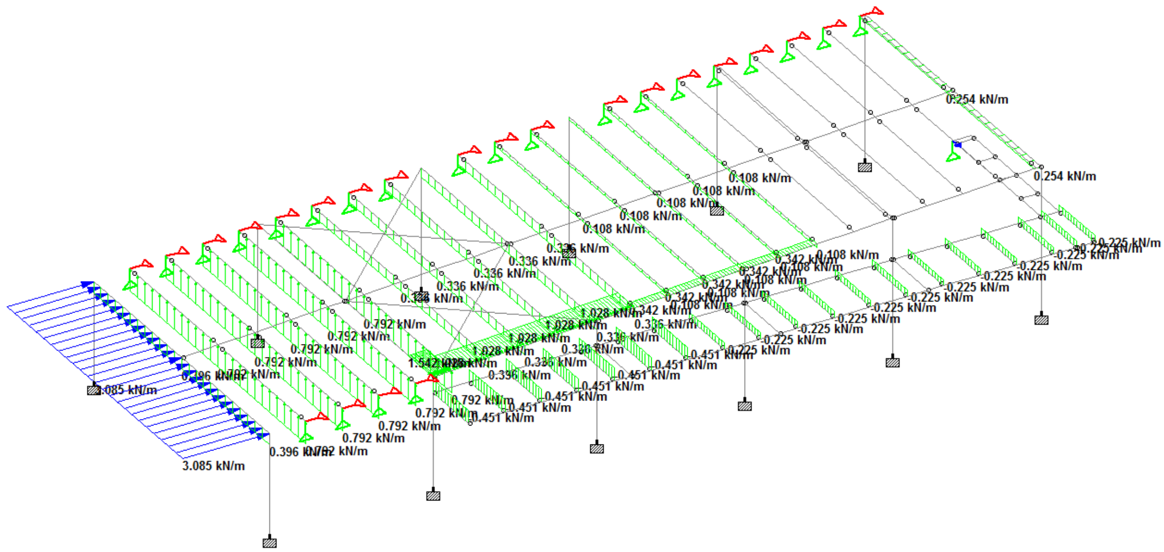
Live loads



Live load on sheeting is assigned to members

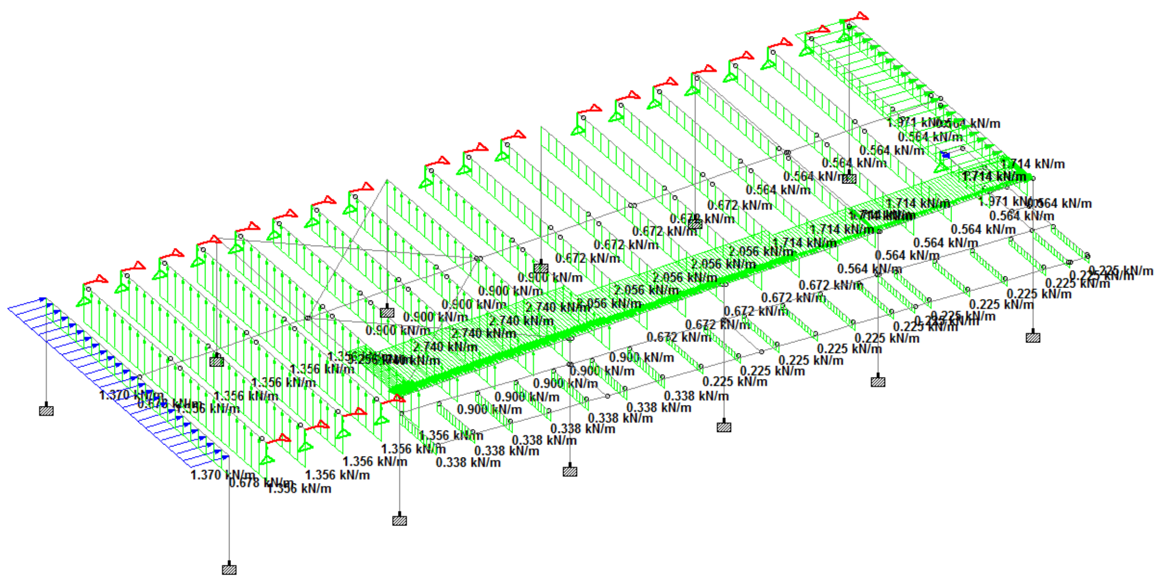
Wind Load

Wind + X (CPE + CPI)



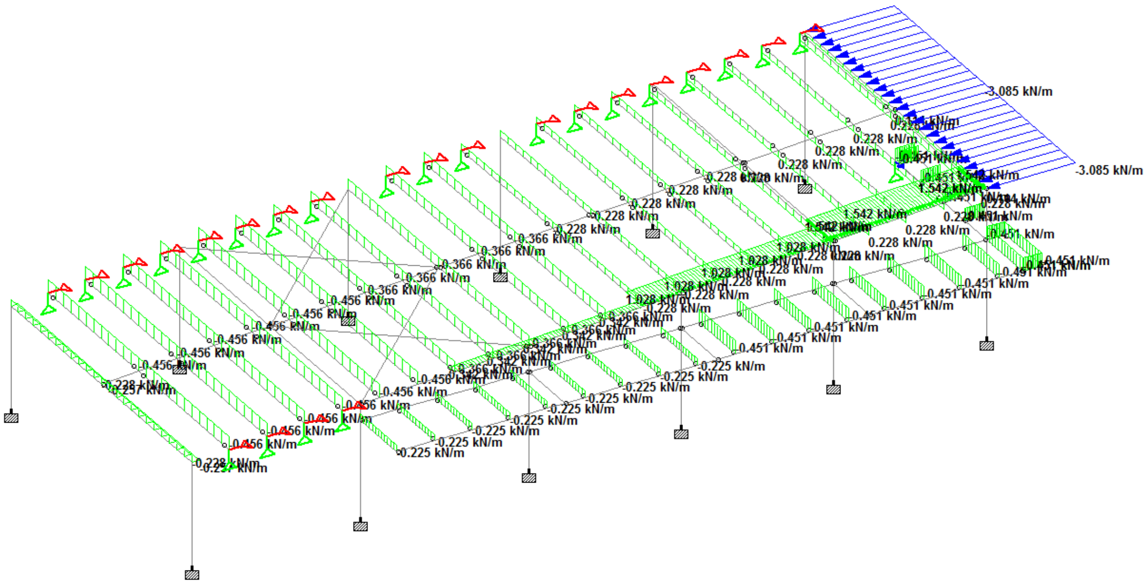
Wind load on members for the case of Wind + X (CPE + CPI) Direction

Wind + X (CPE - CPI)



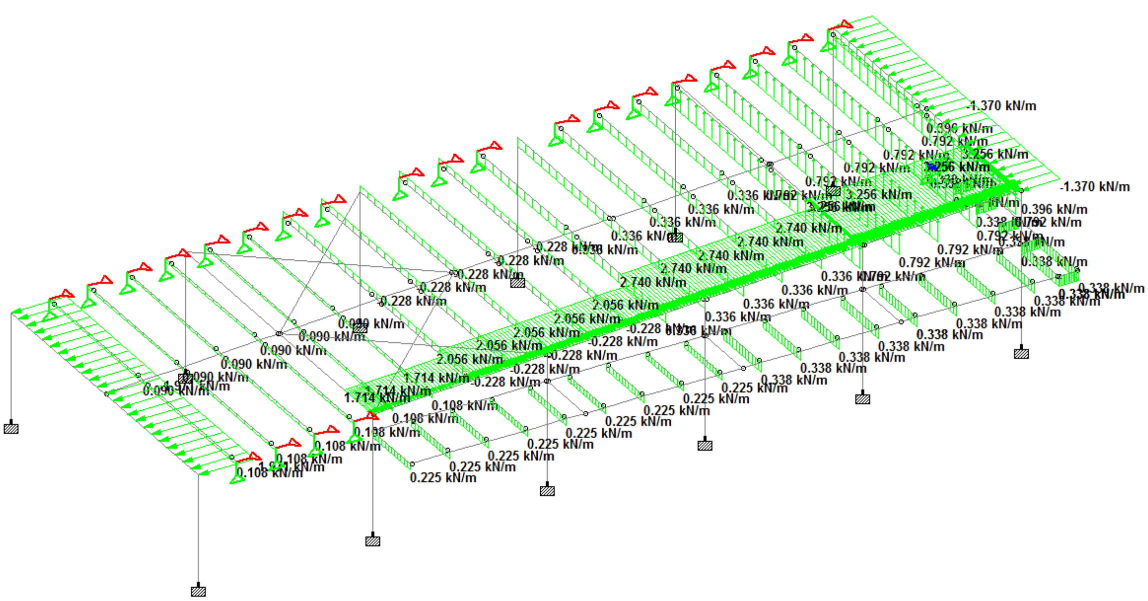
Wind load members for the case of Wind + X (CPE - CPI) Direction

Wind -X (CPE + CPI)



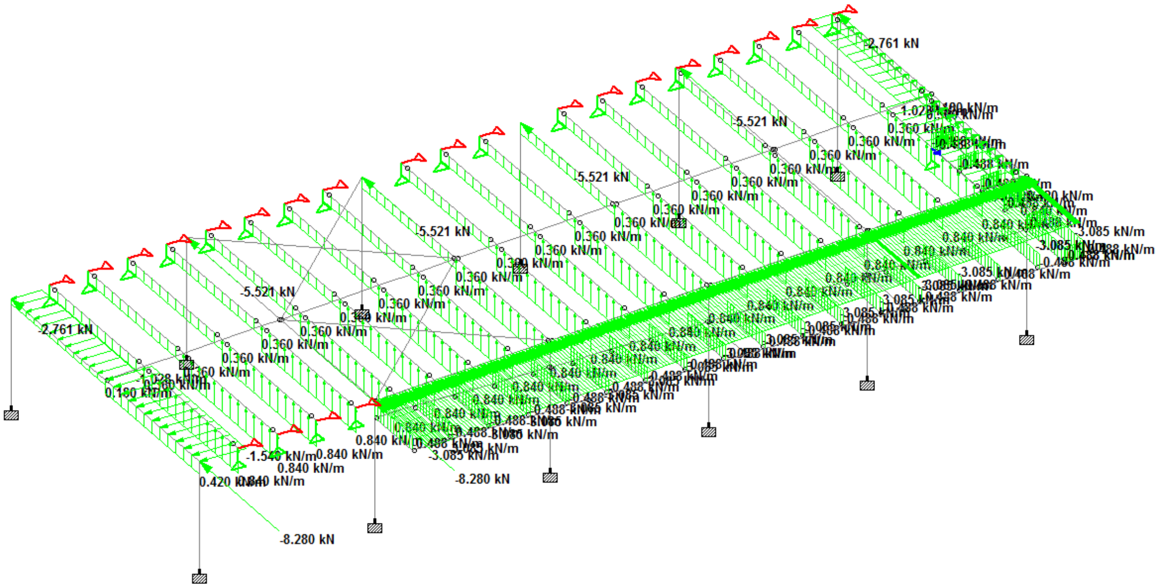
Wind load on members for the case of Wind -X (CPE + CPI) Direction

Wind - X (CPE - CPI)



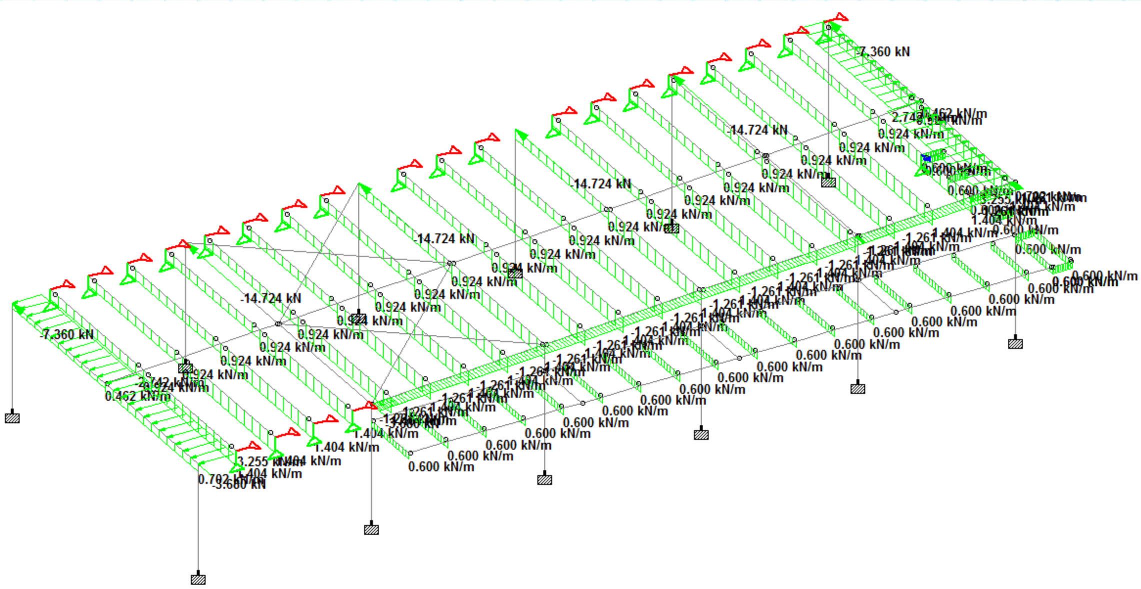
Wind load on members for the case of Wind -X (CPE - CPI) Direction

Wind + Z (CPE + CPI)



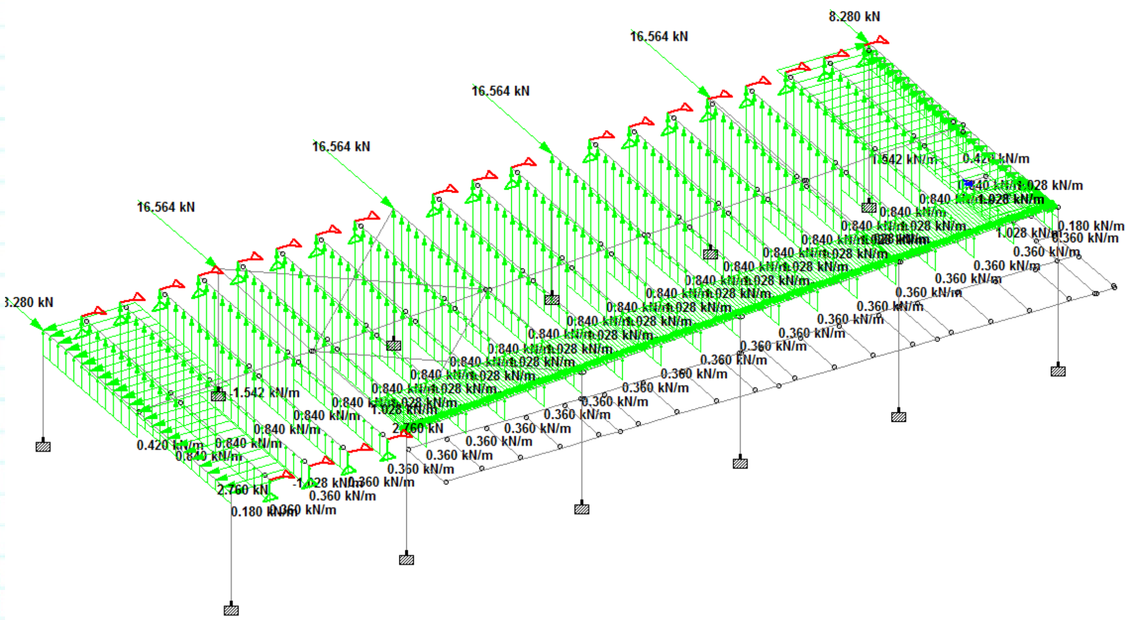
Wind load on members for the case of Wind + Z (CPE + CPI) Direction

Wind + Z (CPE - CPI)



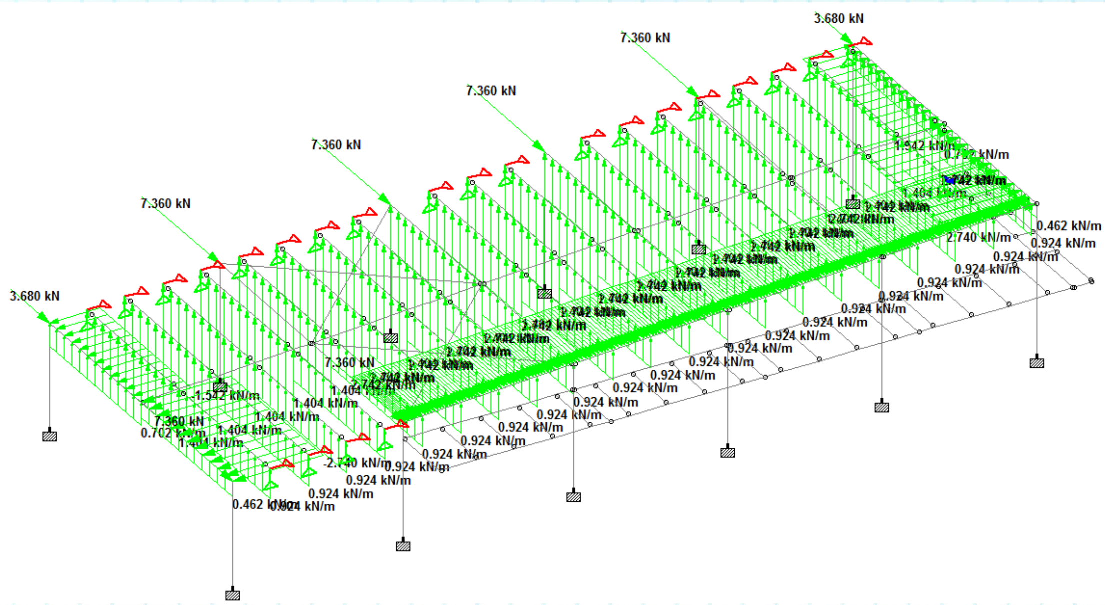
Wind load on members for the case of Wind +Z (CPE - CPI) Direction

Wind - Z (CPE + CPI)



Wind load on members for the case of Wind -Z (CPE + CPI) Direction

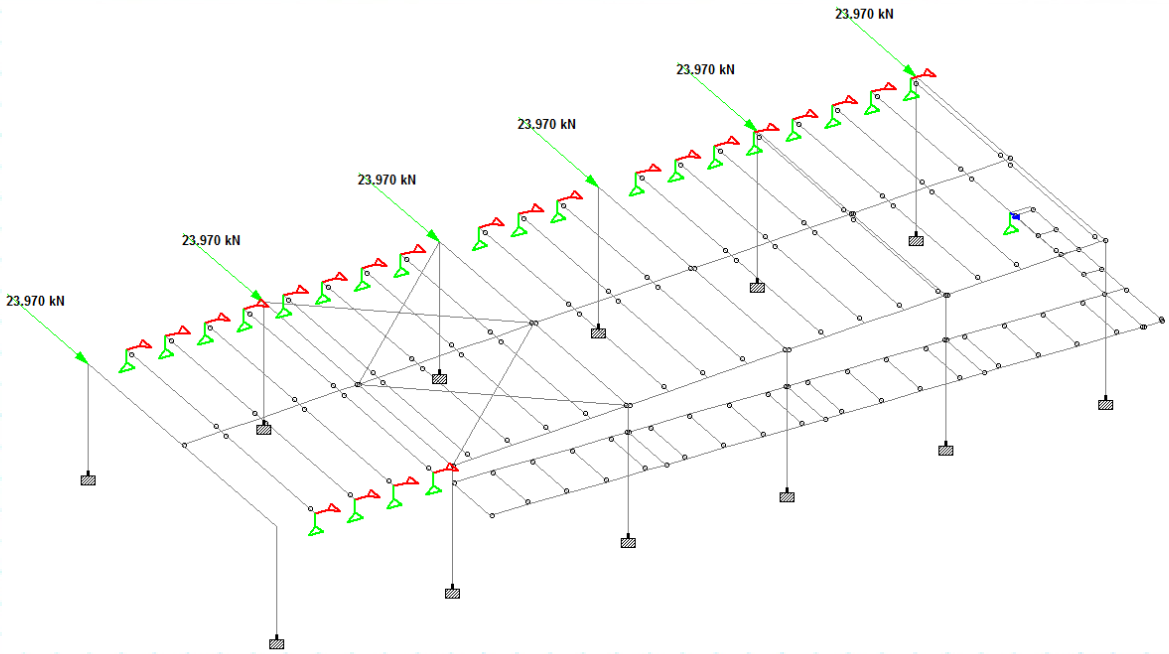
Wind - Z (CPE - CPI)



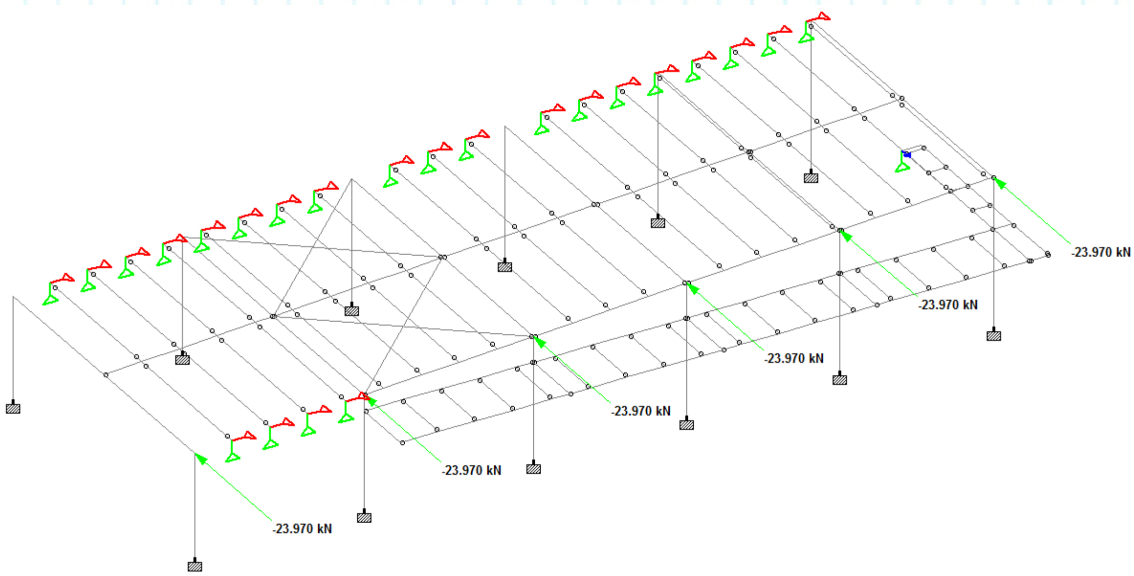
Wind load on members for the case of Wind -Z (CPE - CPI) Direction

EARTHQUAKE + Z

For load value refer Page 13.



EARTHQUAKE - Z



- **Design Load Cases**

  <u>For Steel Roof structure</u>

  LOAD 1 DEAD LOAD

  LOAD 2 LIVE LOAD

  LOAD 3WIND + X (CPE + CPI)

LOAD 4WIND+ X (CPE - CPI)
LOAD 5WIND-X (CPE + CPI)
LOAD 6WIND - X (CPE - CPI)
LOAD 7WIND + Z (CPE + CPI)
LOAD 8WIND+Z (CPE - CPI)
LOAD 9WIND- Z (CPE +CPI)
LOAD 10WIND - Z (CPE - CPI)
LOAD 11 EARTHQUAKE + Z
LOAD 12 EARTHQUAKE - Z

<u>Design Load Combinations</u>(Clause 4.2.2 NZS1170.0:2002)
<u>For Steel Roof structure design</u>

1) LOAD COMB 101 1.35DL
2) LOAD COMB 102 1.2DL + 1.5LL
3) LOAD COMB 103 1.2DL +1.0 (WL+ X (CPE + CPI))
4) LOAD COMB 104 1.2DL + 1.0 (WL- X (CPE + CPI))
5) LOAD COMB 105 1.2DL + 1.0 (WL+ Z (CPE + CPI))
6) LOAD COMB 106 1.2DL + 1.0 (WL-Z (CPE + CPI))
7) LOAD COMB 107 1.2DL + 1.0 (WL+ X (CPE - CPI))
8) LOAD COMB 108 1.2DL + 1.0 (WL- X (CPE - CPI))
9) LOAD COMB 109 1.2DL + 1.0 (WL+ Z (CPE - CPI))
10) LOAD COMB 110 1.2DL + 1.0 (WL-Z (CPE - CPI))
11) LOAD COMB 111 0.9DL + 1.0 (WL+ X (CPE + CPI))
12) LOAD COMB 112 0.9DL + 1.0 (WL- X (CPE + CPI))
13) LOAD COMB 113 0.9DL + 1.0 (WL+ Z (CPE + CPI))
14) LOAD COMB 114 0.9DL + 1.0 (WL-Z (CPE + CPI))
15) LOAD COMB 115 0.9DL + 1.0 (WL+ X (CPE - CPI))
16) LOAD COMB 116 0.9DL + 1.0 (WL- X (CPE - CPI))
17) LOAD COMB 117 0.9DL + 1.0 (WL+ Z (CPE - CPI))
18) LOAD COMB 118 0.9DL + 1.0 (WL-Z (CPE - CPI))
19) LOAD COMB 119 1.0DL + 1.0 EARTHQUAKE + Z
20) LOAD COMB 120 1.0DL + 1.0 EARTHQUAKE - Z

<u>For Foundation design</u>

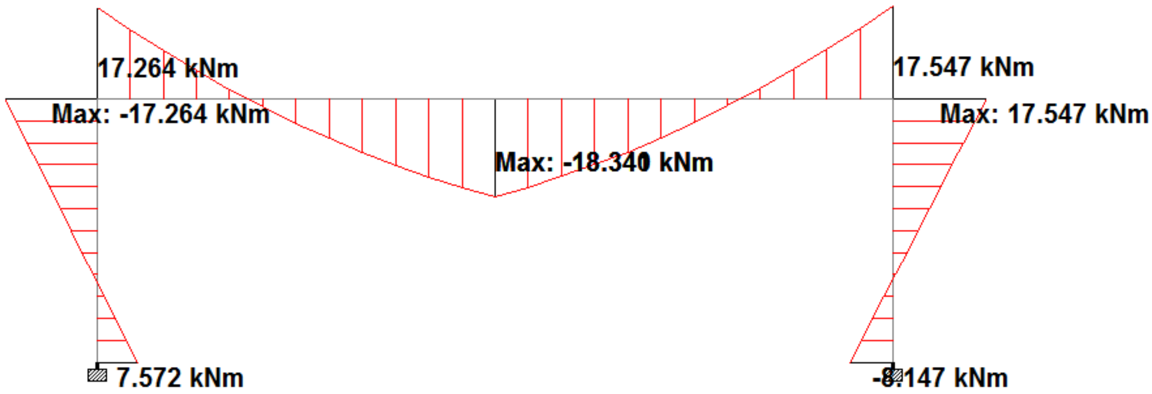
21) LOAD COMB 201 1.0DL + 1.0LL
22) LOAD COMB 202 1.0DL + 1.0 (WL+ X (CPE + CPI))
23) LOAD COMB 203 1.0DL + 1.0 (WL- X (CPE + CPI))
24) LOAD COMB 204 1.0DL + 1.0 (WL+ Z (CPE + CPI))
25) LOAD COMB 205 1.0DL + 1.0 (WL-Z (CPE + CPI))
26) LOAD COMB 206 1.0DL + 1.0 (WL+ X (CPE - CPI))
27) LOAD COMB 207 1.0DL + 1.0 (WL- X (CPE - CPI))

28) LOAD COMB 208 1.0DL + 1.0 (WL+ Z (CPE - CPI))
29) LOAD COMB 209 1.0DL + 1.0 (WL-Z (CPE - CPI))
30) LOAD COMB 210 1.0DL + 1.0 EARTHQUAKE + Z
31) LOAD COMB 211 1.0DL + 1.0 EARTHQUAKE – Z

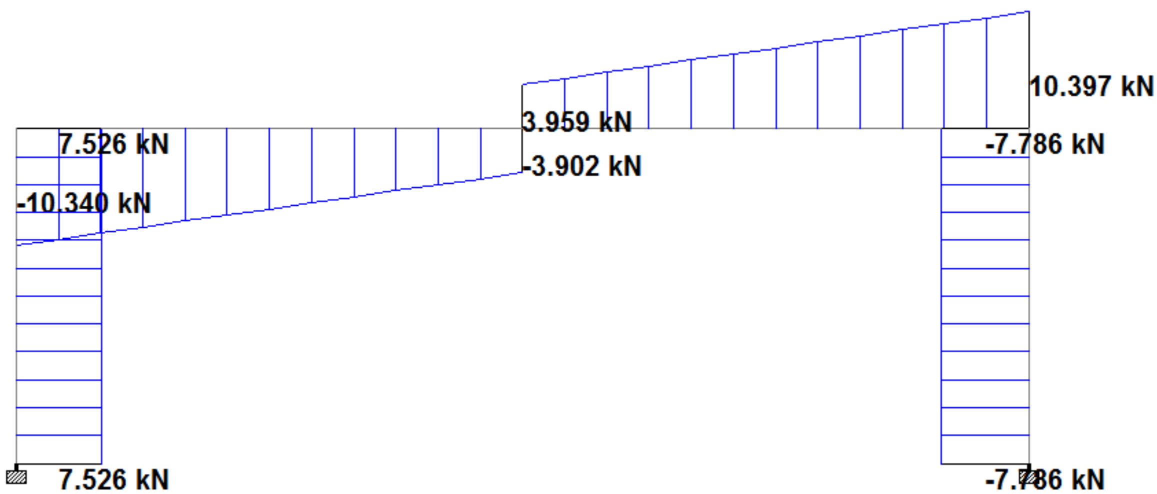
(For Frame on Grid 1)

Value of B.M. & S.F. for DL + WL+Z(Cpe + Cpi)

Bending Moment Diagram



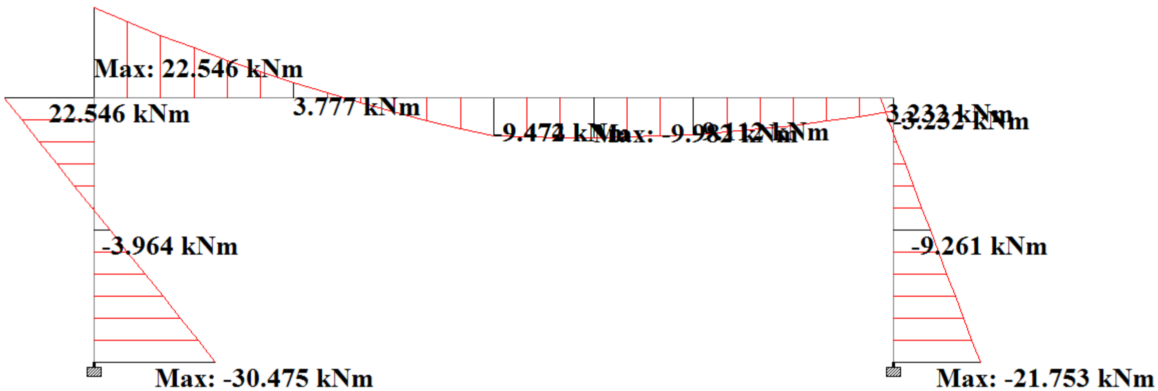
Shear Force Diagram



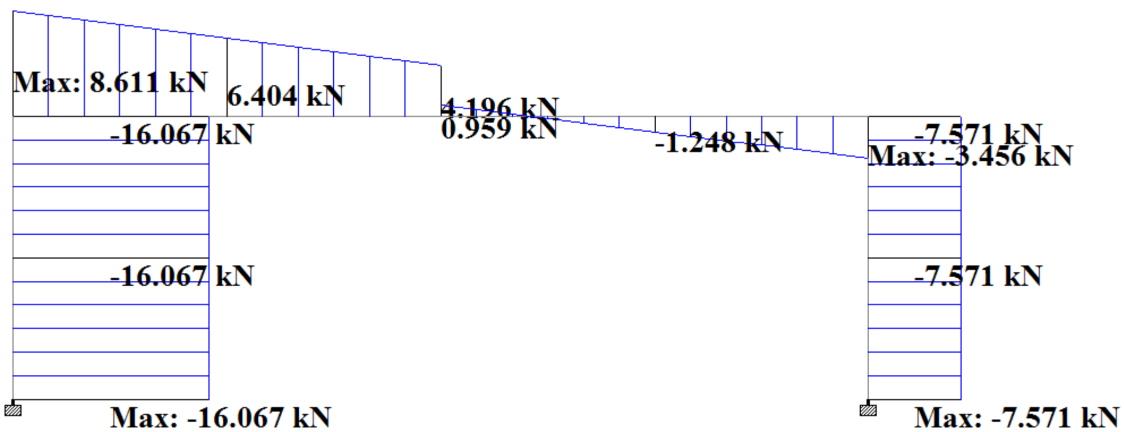
(For Frame on Grid 1)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



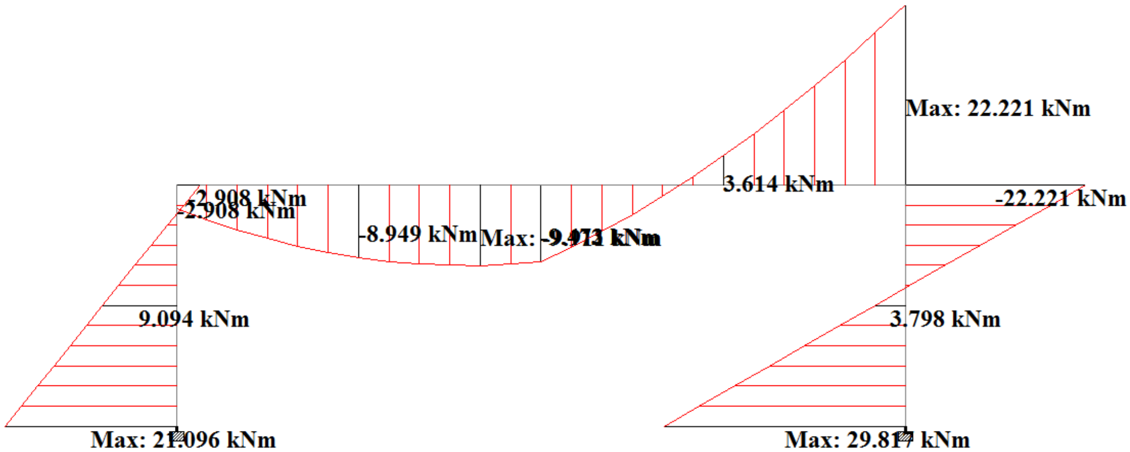
Shear Force Diagram



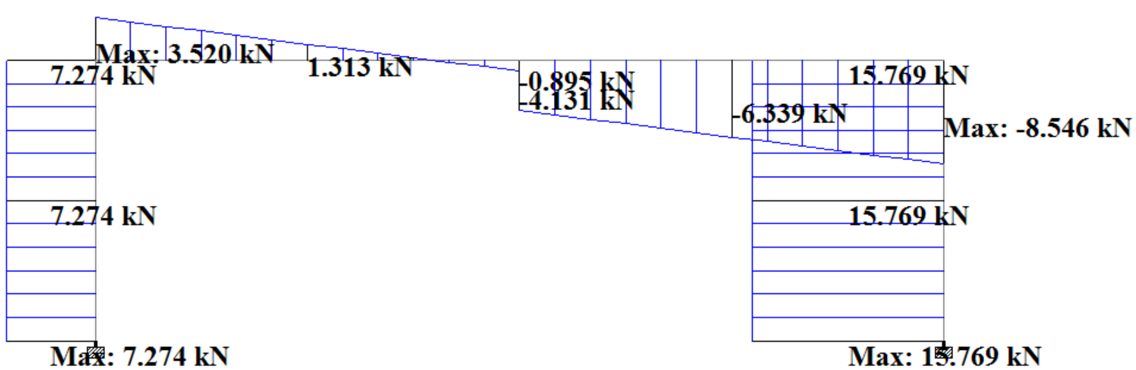
(For Frame on Grid 1)

Value of B.M. & S.F. for DL + EQ - Z

Bending Moment Diagram



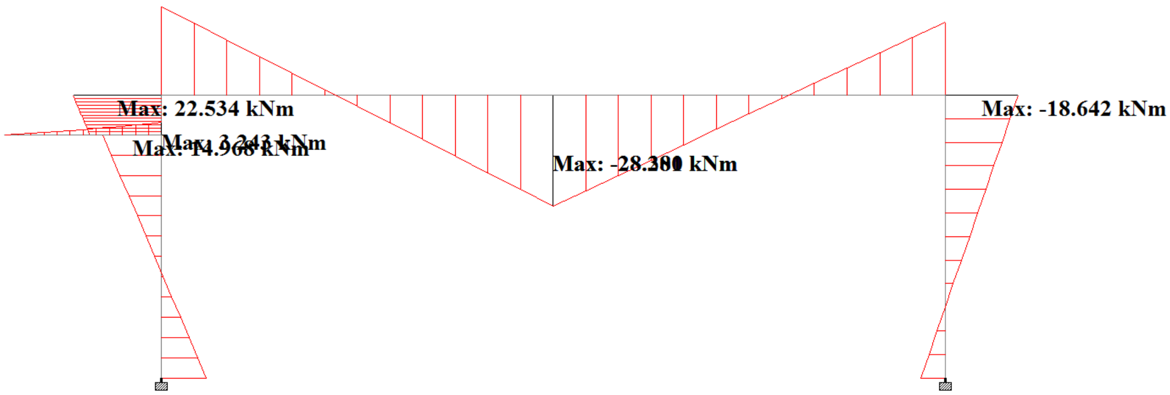
Shear Force Diagram



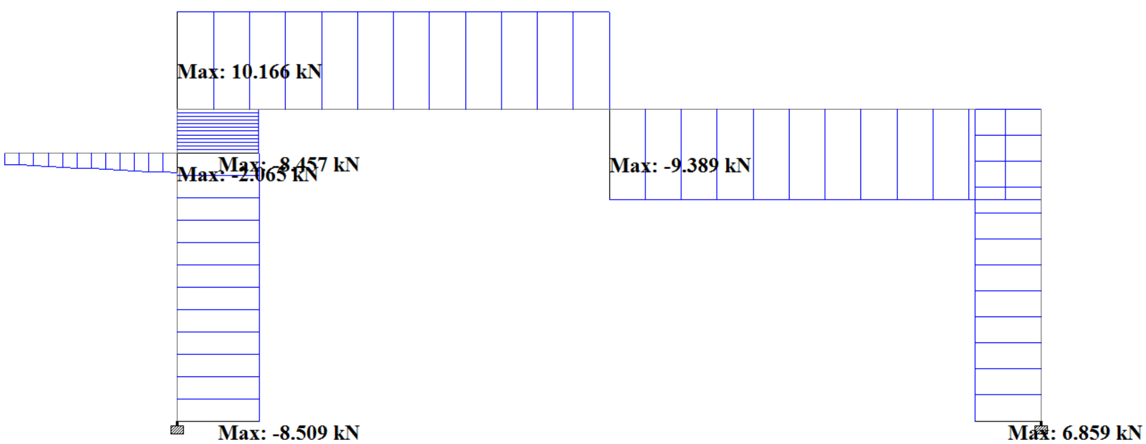
(For Frame on Grid 2)

Value of B.M. & S.F. for DL + WL+Z (Cpe + Cpi)

Bending Moment Diagram



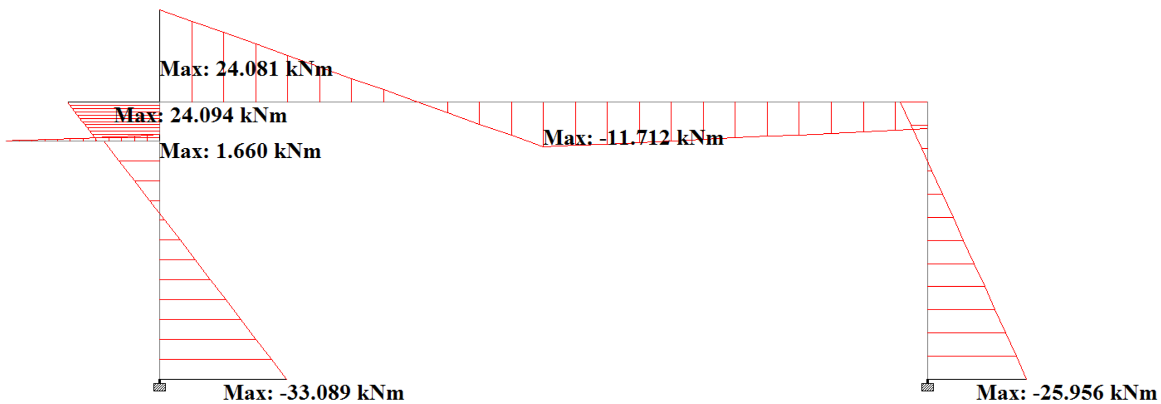
Shear Force Diagram



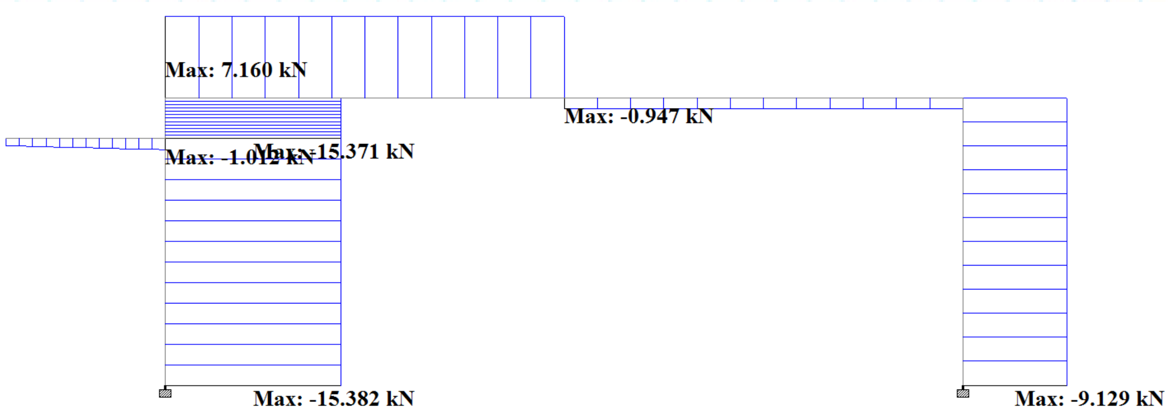
(For Frame on Grid 2)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



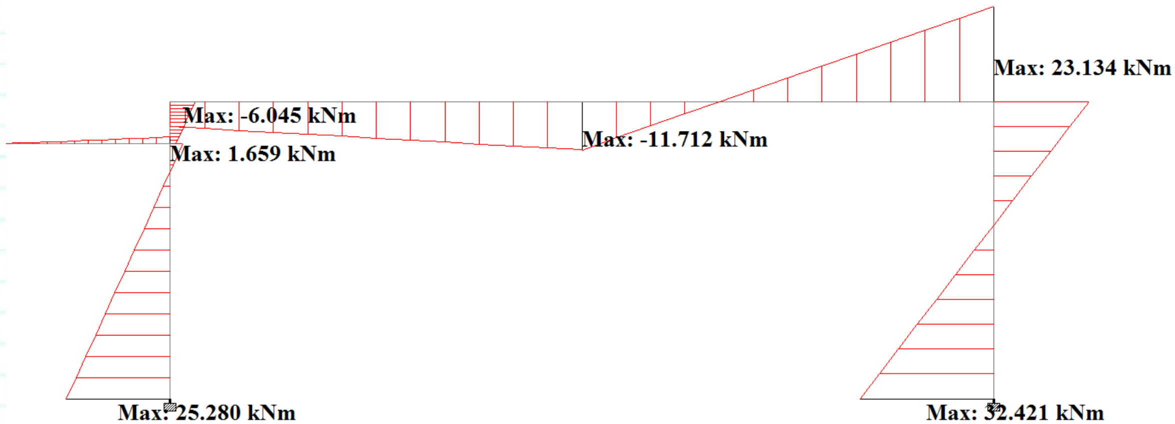
Shear Force Diagram



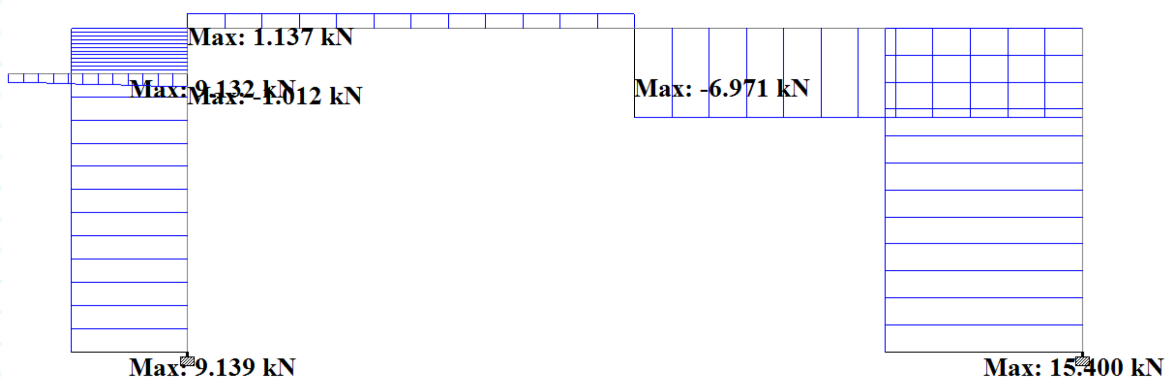
(For Frame on Grid 2)

Value of B.M. & S.F. for DL + EQ - Z

Bending Moment Diagram



Shear Force Diagram

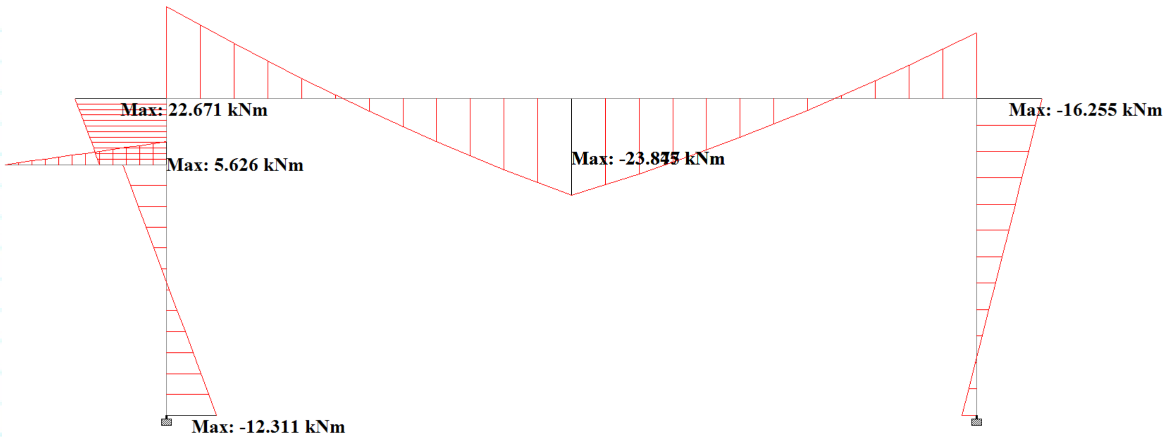


(For Frame on Grid 3)

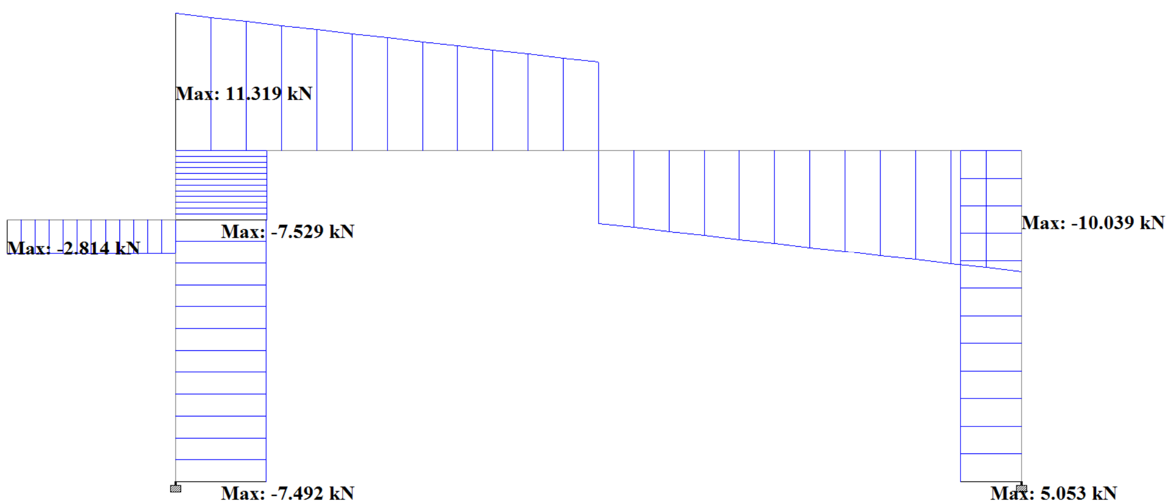
Value of B.M. & S.F. for DL + WL+Z (Cpe + Cpi)

Bending Moment Diagram

ΩMEGA ENGINEERING CONSULTANTS LIMITED	Project Name: 1333 Cameron Road Tauranga				Project No:
	For: Design Philosophy Report				
By:	Date: 16/1/2016	Checked By:	Checked Date:	Page No:	Rev:



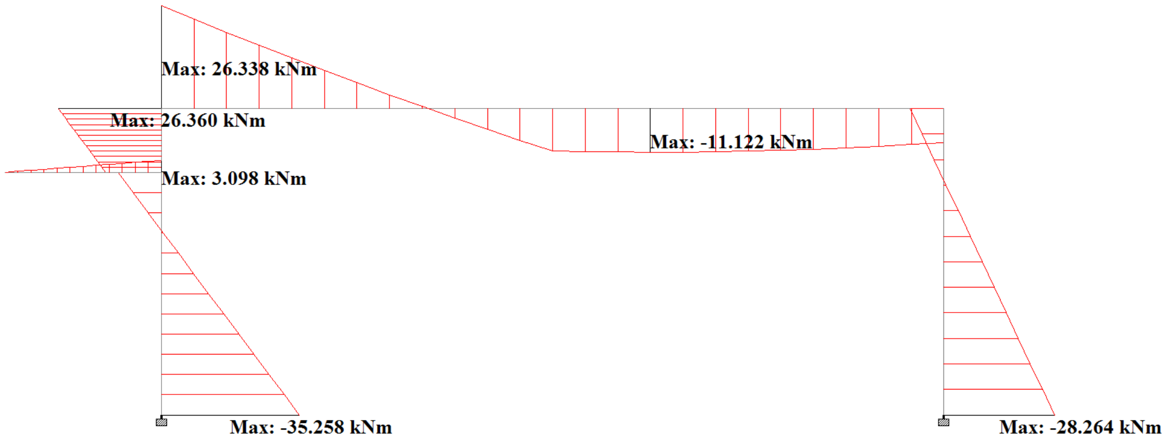
Shear Force Diagram



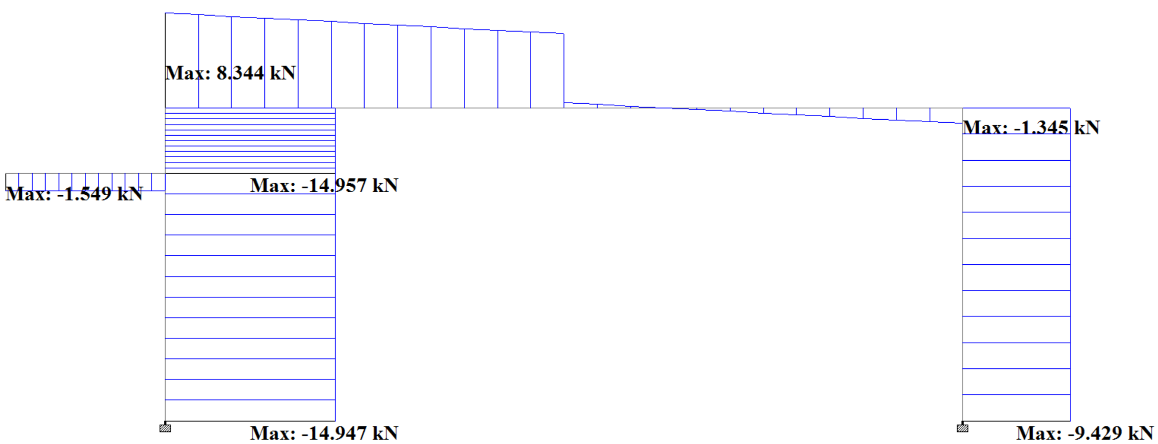
(For Frame on Grid 3)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



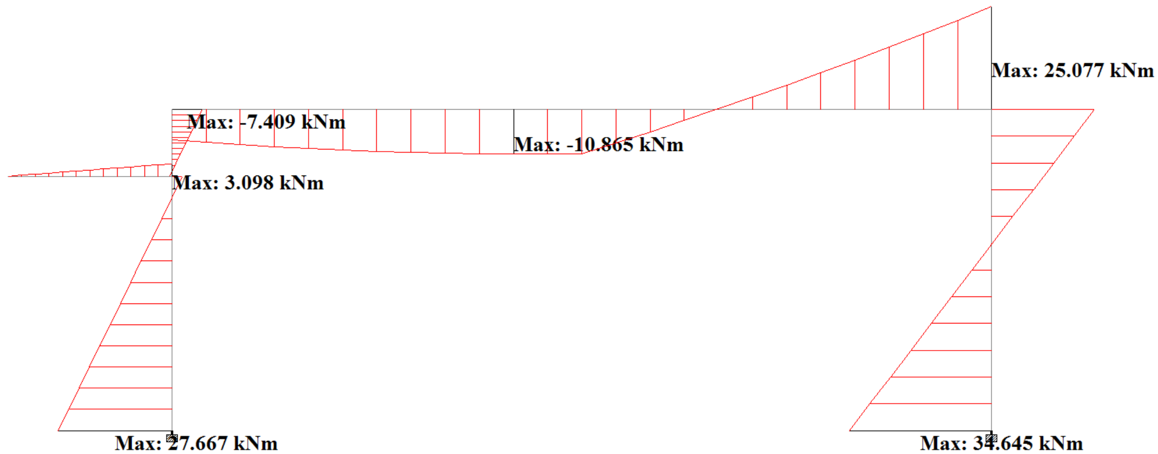
Shear Force Diagram



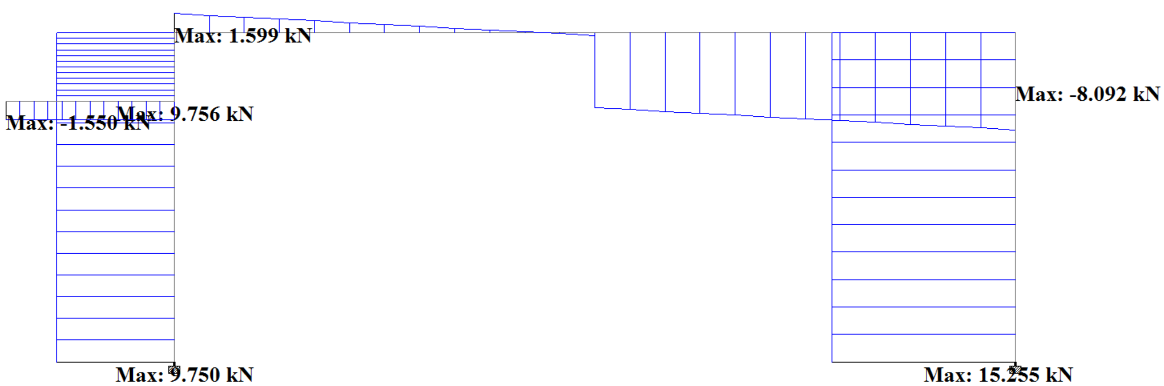
(For Frame on Grid 3)

Value of B.M. & S.F. for DL + EQ - Z

Bending Moment Diagram



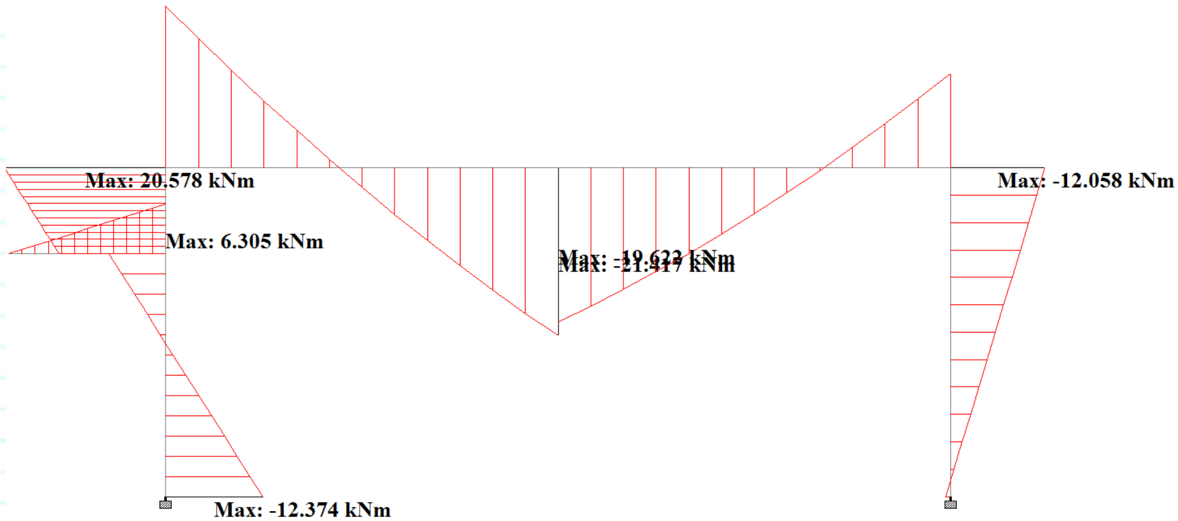
Shear Force Diagram



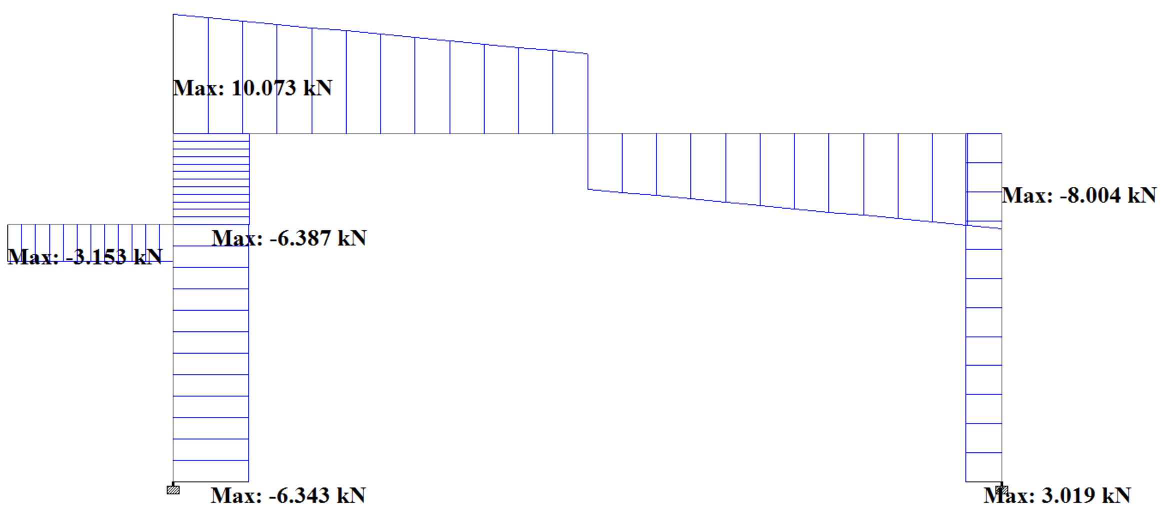
(For Frame on Grid 4)

Value of B.M. & S.F. for DL + WL+Z (Cpe + Cpi)

Bending Moment Diagram



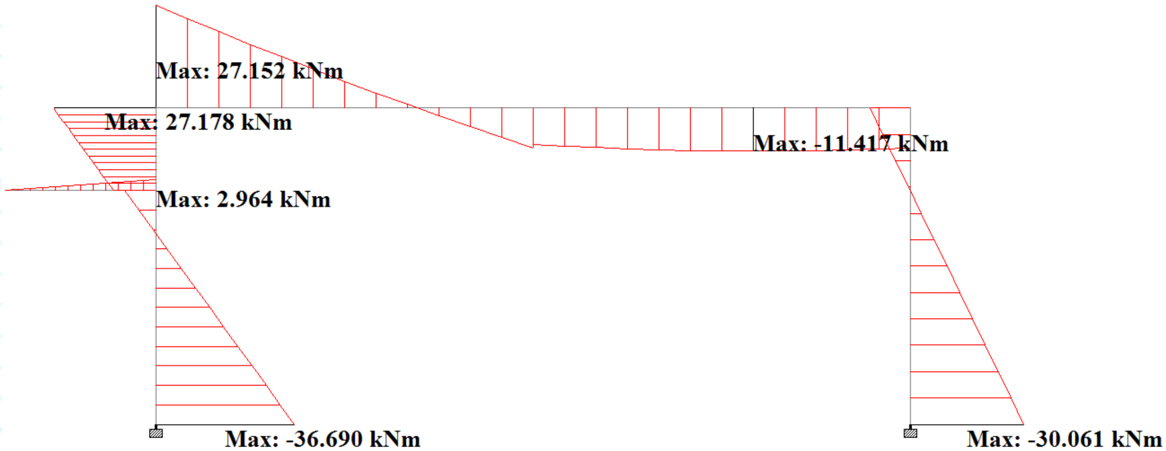
Shear Force Diagram



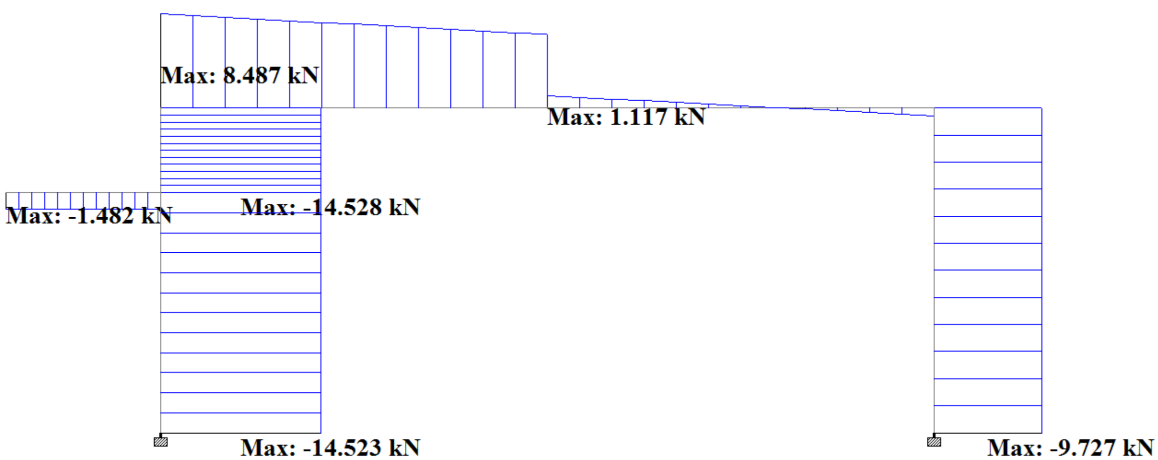
(For Frame on Grid 4)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



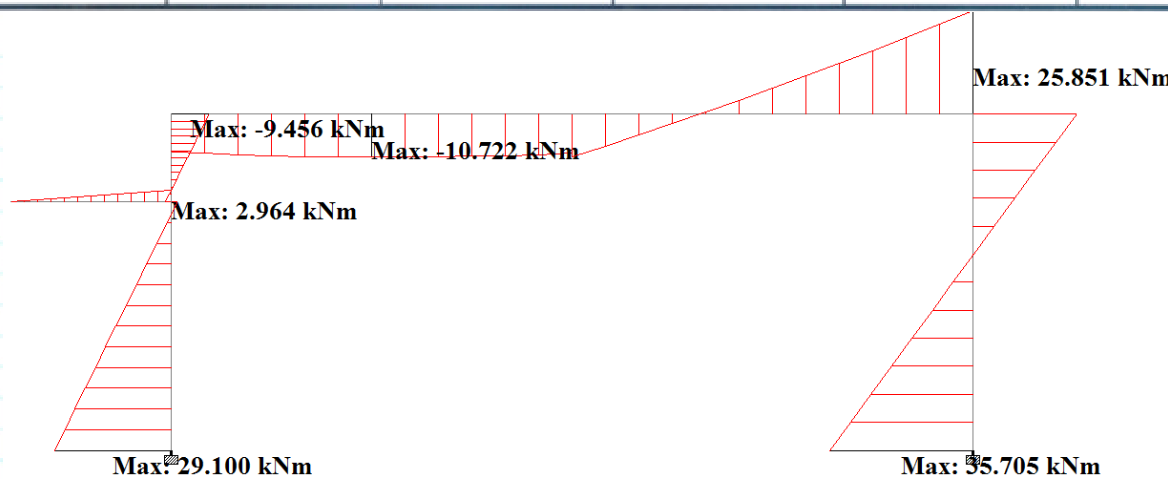
Shear Force Diagram



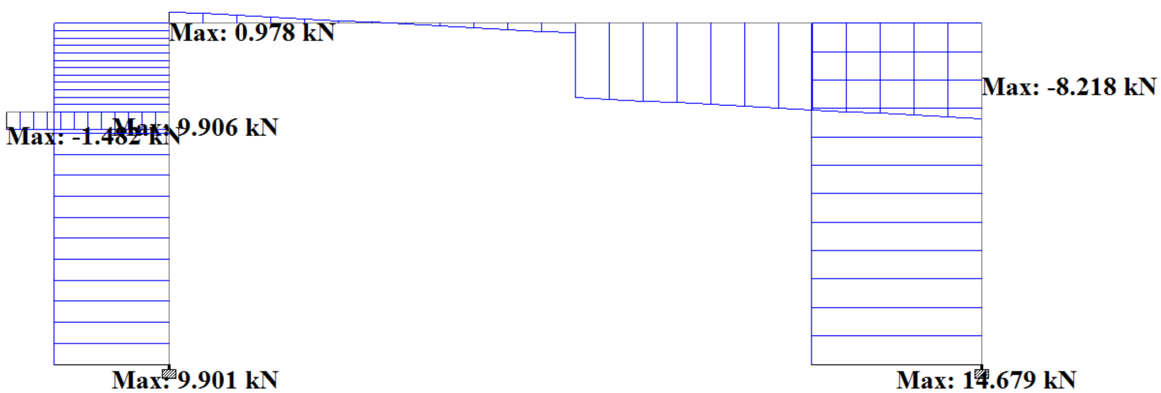
<u>(For Frame on Grid 4)</u>

<u>Value of B.M. & S.F. for DL + EQ - Z</u>

Bending Moment Diagram



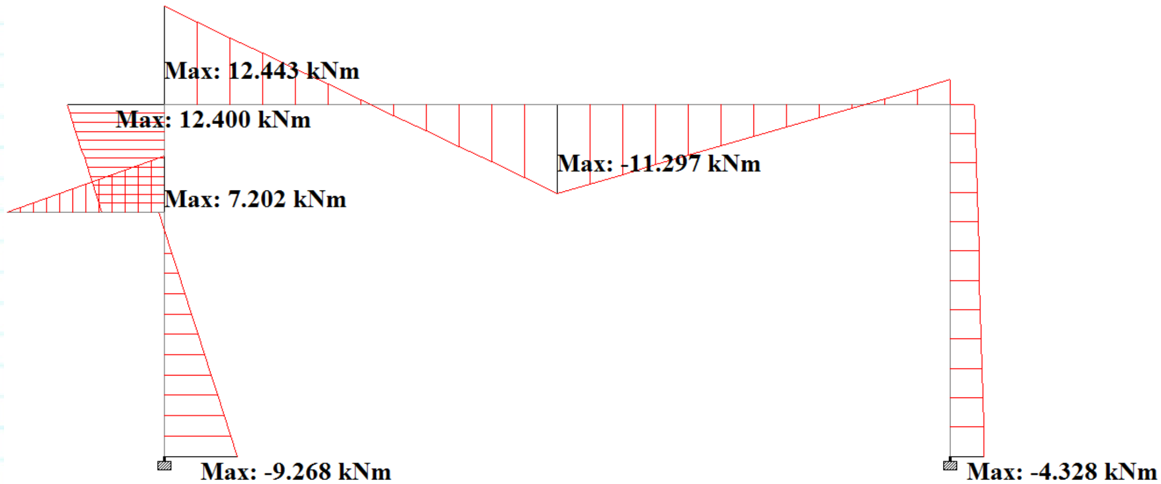
Shear Force Diagram



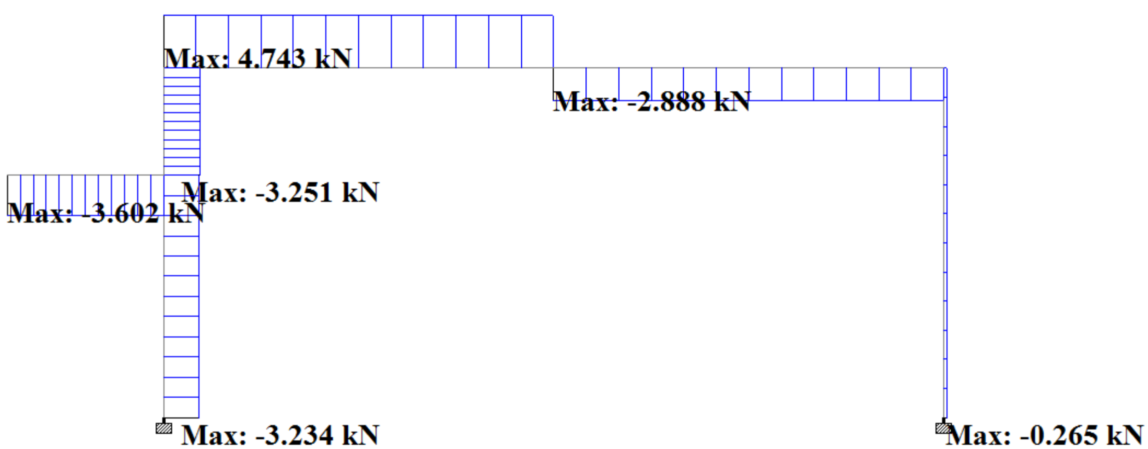
<u>(For Frame on Grid 5)</u>

<u>Value of B.M. & S.F. for DL + WL+Z (Cpe + Cpi)</u>

Bending Moment Diagram



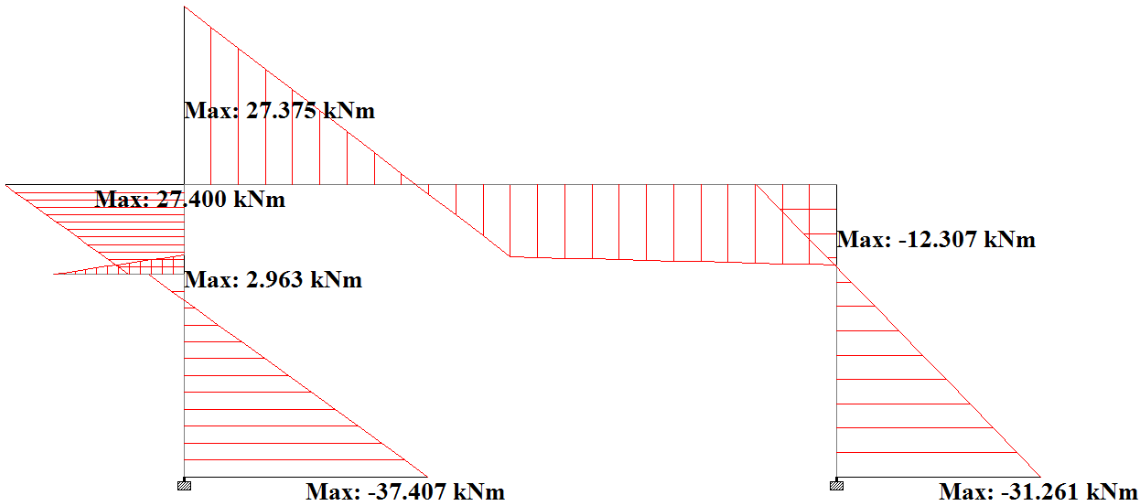
Shear Force Diagram



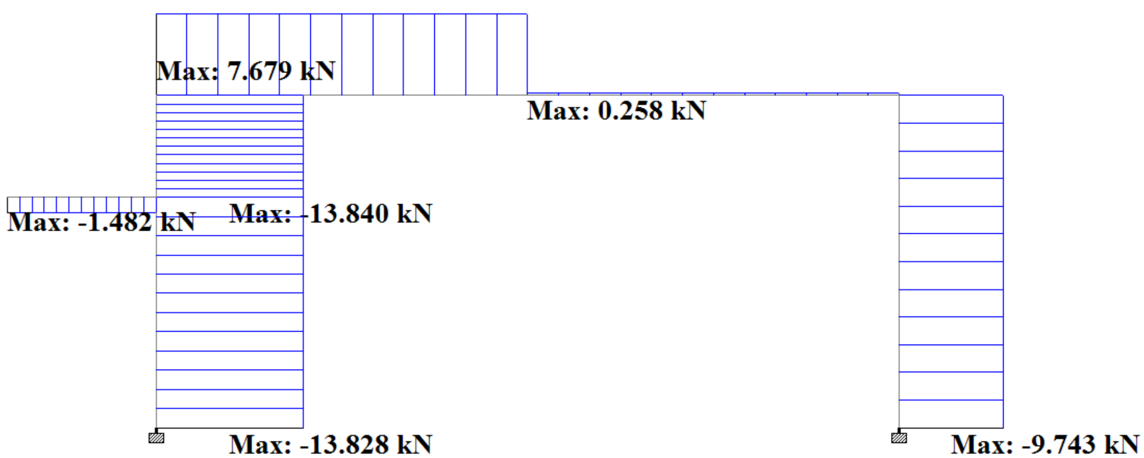
(For Frame on Grid 5)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



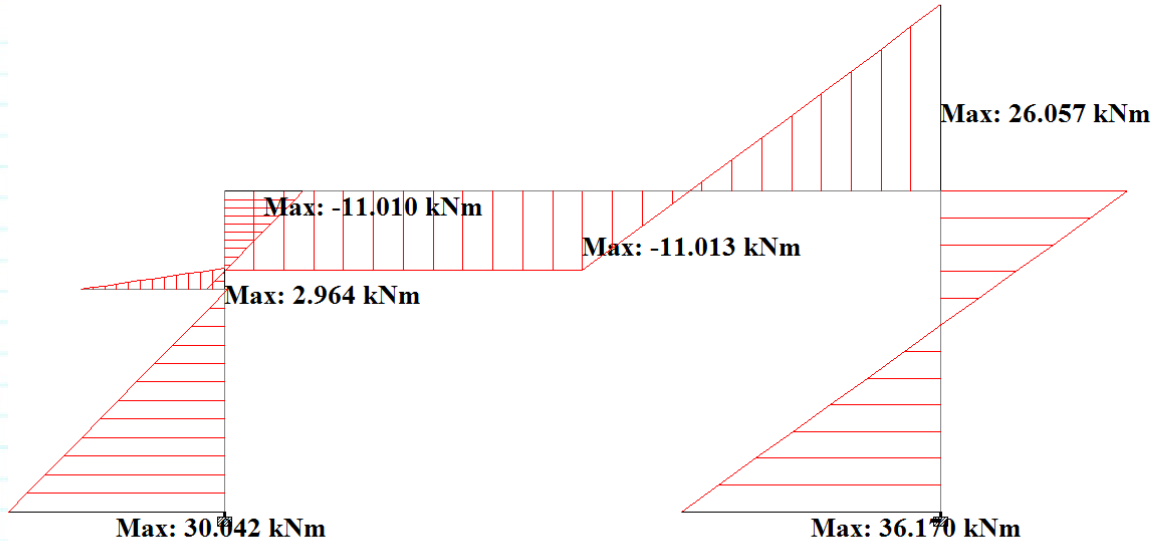
Shear Force Diagram



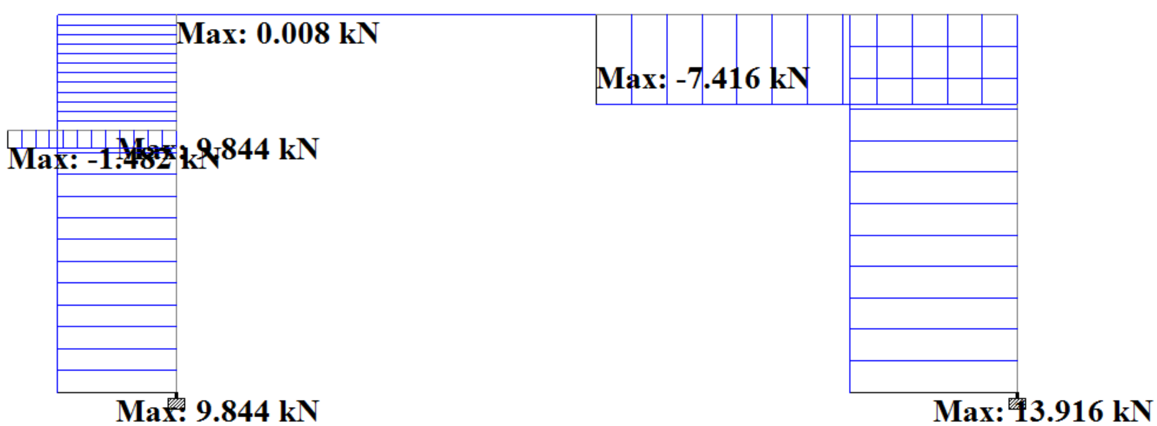
(For Frame on Grid 5)

Value of B.M. & S.F. for DL + EQ - Z

Bending Moment Diagram



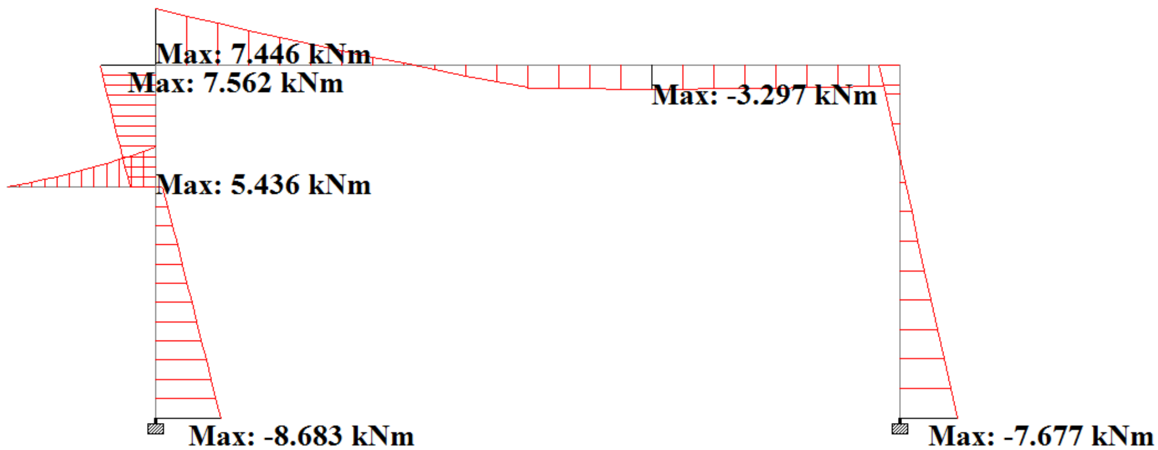
Shear Force Diagram



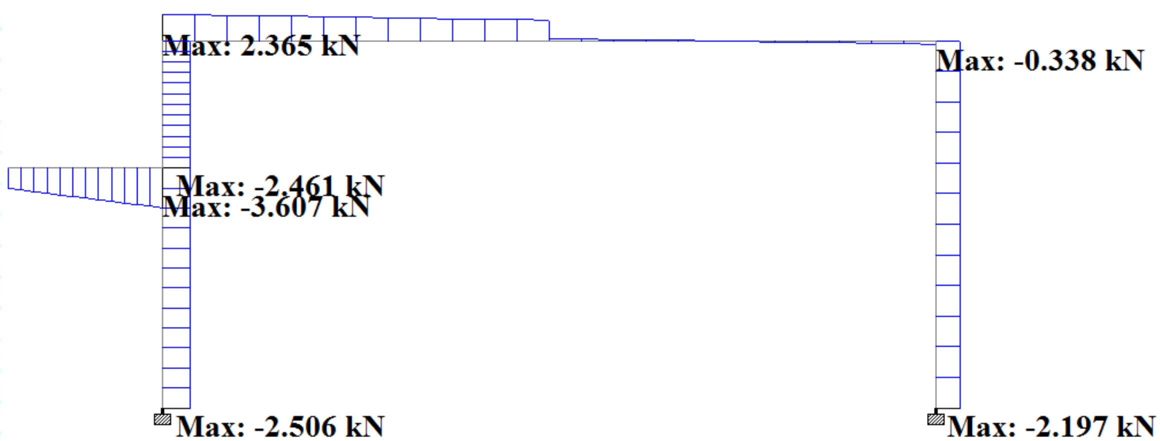
(For Frame on Grid 6)

Value of B.M. & S.F. for DL + WL+Z (Cpe + Cpi)

Bending Moment Diagram



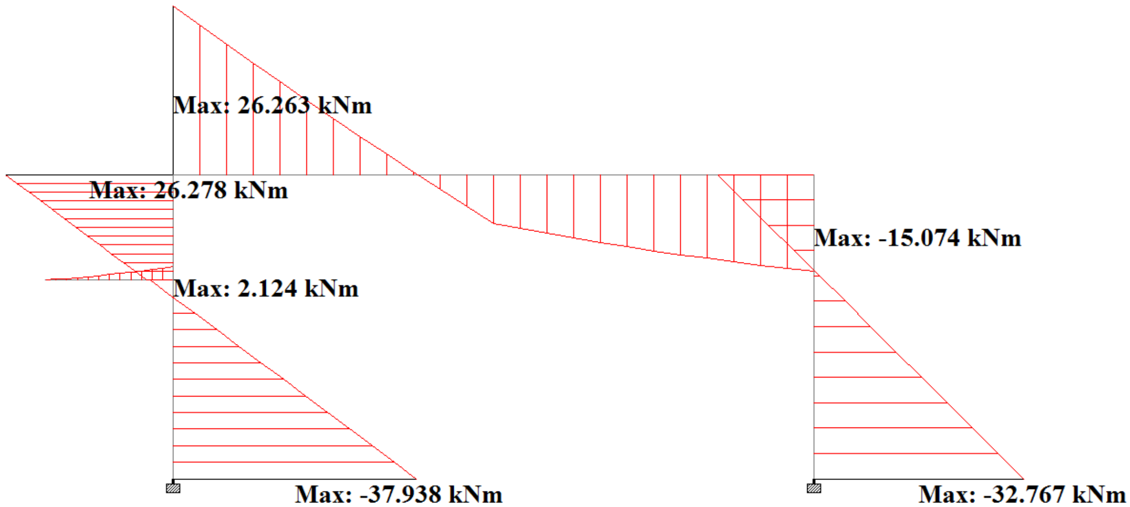
Shear Force Diagram



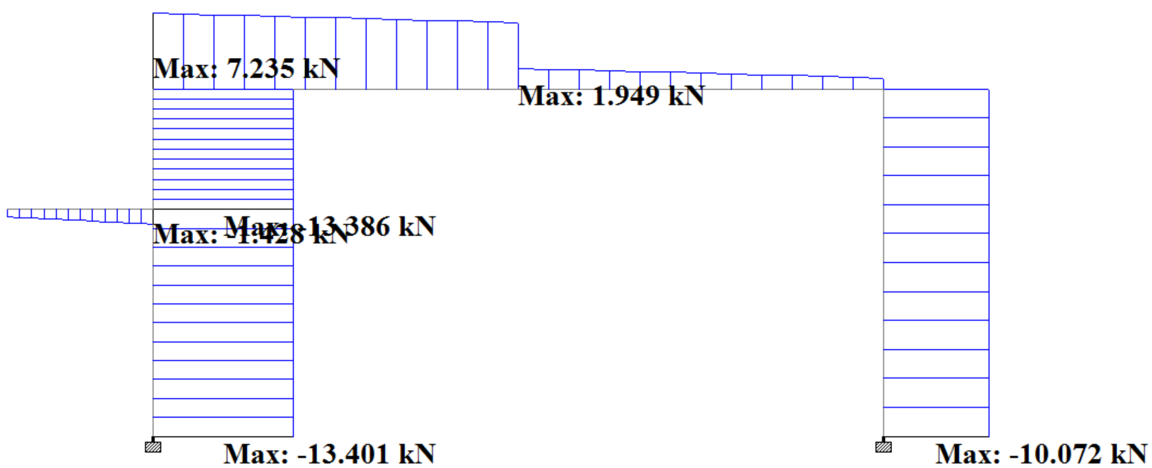
(For Frame on Grid 6)

Value of B.M. & S.F. for DL + EQ+Z

Bending Moment Diagram



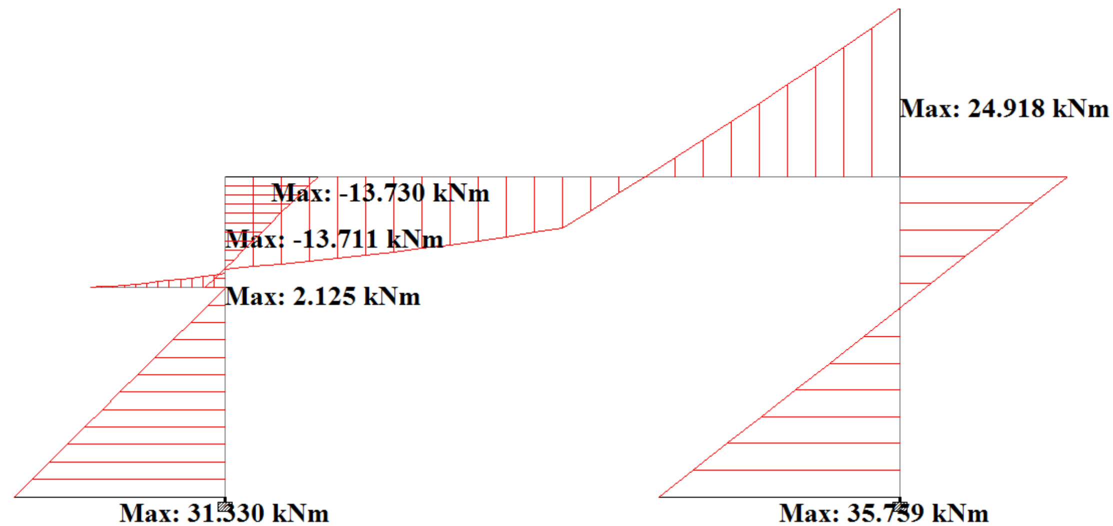
Shear Force Diagram



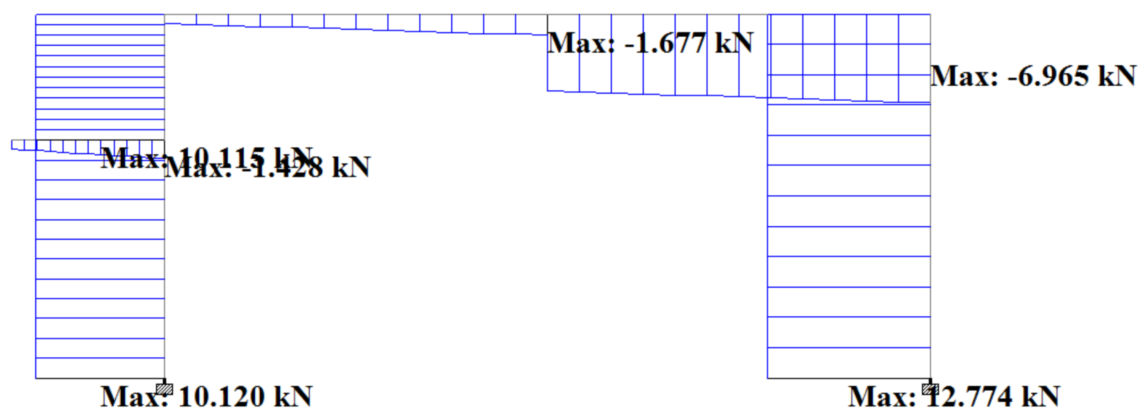
(For Frame on Grid 6)

Value of B.M. & S.F. for DL + EQ - Z

Bending Moment Diagram



Shear Force Diagram



<u>Purlin Design:-</u>

Dead Load = 0.305 kN/m2

Live load = 0.25 kN/m2

Wind load = - 1.13 x 1.5 kN/m2 = - 1.695kN/m2

<u>Critical Design Load Combinations for the Ultimate Limit State</u>

1.2 DL + 1.5 LL = (1.2 x 0.305) + (1.5 x 0.25) = 0.741 kN/m2

0.9 DL + WL = (0.9 x 0.305) - (1.695) = -1.4205 kN/m2

<u>Critical Design Load Combinations for the Serviceability Limit State</u>

$W_{SLS} = L_p / 150$ under WL = - 1.13 kN/m2

For 5.5 m single Spacing DHS 200 / 12 Purlin required as per below load table.

2.3.7 DHS LOAD SPAN TABLES – SINGLE SPANS

Uniformly loaded bending capacities (kN/m) $\phi_b W_{bx}$

Span (m)	DHS 150/12 1B	DHS 150/12 2B	DHS 150/12 3B	DHS 150/12 FR	DHS 150/12 W_s	DHS 150/15 1B	DHS 150/15 2B	DHS 150/15 3B	DHS 150/15 FR	DHS 150/15 W_s
3.0	5.17	5.17	5.17	5.17	4.73					
3.5	3.80	3.80	3.80	3.80	3.02	5.18	5.18	5.18	5.18	3.92
4.0	2.91	2.91	2.91	2.91	2.05	3.96	3.96	3.96	3.96	2.65
4.5	2.30	2.30	2.30	2.30	1.45	3.09	3.13	3.13	3.13	1.86
5.0	1.73	1.86	1.86	1.86	1.06	2.29	2.53	2.53	2.53	1.36
5.5	1.26	1.54	1.54	1.54	0.80	1.67	2.09	2.09	2.09	1.02
6.0	0.94	1.29	1.29	1.29	0.62	1.24	1.76	1.76	1.76	0.78
6.5	0.71	1.10	1.10	1.10	0.49	0.94	1.50	1.50	1.50	0.62
7.0	0.55	0.94	0.95	0.95	0.39	0.72	1.26	1.29	1.29	0.49
7.5	0.43	0.78	0.82	0.82	0.32	0.56	1.03	1.12	1.12	0.40
8.0						0.44	0.84	0.99	0.99	0.33
8.5										
9.0										
9.5										
10.0										
10.5										
11.0										
11.5										
12.0										
12.5										
13.0										
13.5										
14.0										
14.5										
15.0										
15.5										
16.0										
16.5										
17.0										
17.5										
18.0										

Span (m)	DHS 200/12 1B	DHS 200/12 2B	DHS 200/12 3B	DHS 200/12 FR	DHS 200/12 W_s	DHS 200/15 1B	DHS 200/15 2B	DHS 200/15 3B	DHS 200/15 FR	DHS 200/15 W_s
3.0										
3.5	5.63	5.63	5.63	5.63	5.86					
4.0	4.31	4.31	4.31	4.31	4.03	5.91	5.91	5.91	5.91	5.51
4.5	3.40	3.40	3.40	3.40	2.90	4.67	4.67	4.67	4.67	3.91
5.0	2.69	2.75	2.75	2.75	2.16	3.78	3.78	3.78	3.78	2.87
5.5	2.09	2.28	2.28	2.28	1.65	3.02	3.12	3.12	3.12	2.17
6.0	1.63	1.91	1.91	1.91	1.29	2.35	2.62	2.62	2.62	1.68
6.5	1.27	1.63	1.63	1.63	1.02	1.79	2.23	2.23	2.23	1.33
7.0	1.00	1.40	1.40	1.40	0.82	1.39	1.93	1.93	1.93	1.07
7.5	0.81	1.21	1.22	1.22	0.67	1.09	1.68	1.68	1.68	0.87
8.0	0.65	1.02	1.07	1.07	0.56	0.87	1.47	1.47	1.47	0.72
8.5	0.53	0.86	0.95	0.95	0.47	0.70	1.25	1.30	1.30	0.60
9.0	0.43	0.74	0.85	0.85	0.39	0.57	1.07	1.16	1.16	0.50
9.5	0.35	0.62	0.76	0.76	0.34	0.47	0.89	1.04	1.04	0.43
10.0	0.29	0.53	0.67	0.69	0.29	0.38	0.75	0.94	0.94	0.37
10.5						0.32	0.63	0.85	0.85	0.32
11.0										
11.5										
12.0										
12.5										
13.0										
13.5										
14.0										
14.5										
15.0										
15.5										
16.0										
16.5										
17.0										
17.5										
18.0										

Span (m)	DHS 200/18 1B	DHS 200/18 2B	DHS 200/18 3B	DHS 200/18 FR	DHS 200/18 W_s	DHS 250/13 1B	DHS 250/13 2B	DHS 250/13 3B	DHS 250/13 FR	DHS 250/13 W_s
3.0										
3.5										
4.0	7.60	7.60	7.60	7.60	6.80	5.37	5.37	5.37	5.37	7.48
4.5	6.00	6.00	6.00	6.00	4.82	4.77	4.77	4.77	4.77	5.37
5.0	4.86	4.86	4.86	4.86	3.54	4.27	4.27	4.27	4.27	3.99
5.5	3.85	4.02	4.02	4.02	2.66	3.43	3.53	3.53	3.53	3.05
6.0	2.94	3.38	3.38	3.38	2.05	2.73	2.96	2.96	2.96	2.39
6.5	2.24	2.88	2.88	2.88	1.61	2.20	2.53	2.53	2.53	1.91
7.0	1.73	2.48	2.48	2.48	1.29	1.75	2.18	2.18	2.18	1.55
7.5	1.36	2.16	2.16	2.16	1.05	1.41	1.90	1.90	1.90	1.28
8.0	1.07	1.90	1.90	1.90	0.86	1.15	1.66	1.67	1.67	1.07
8.5	0.85	1.60	1.68	1.68	0.72	0.94	1.43	1.48	1.48	0.90
9.0	0.69	1.34	1.50	1.50	0.60	0.79	1.23	1.32	1.32	0.77
9.5	0.56	1.11	1.34	1.34	0.51	0.66	1.06	1.18	1.18	0.66
10.0	0.46	0.93	1.21	1.21	0.44	0.56	0.92	1.06	1.06	0.57
10.5	0.38	0.79	1.10	1.10	0.38	0.48	0.79	0.97	0.97	0.49
11.0	0.32	0.67	0.97	1.00	0.33	0.40	0.68	0.86	0.88	0.43
11.5	0.27	0.57	0.85	0.92	0.29	0.34	0.59	0.77	0.80	0.38
12.0						0.29	0.52	0.68	0.74	0.33
12.5						0.25	0.45	0.61	0.68	0.29
13.0										
13.5										
14.0										
14.5										
15.0										
15.5										
16.0										
16.5										
17.0										
17.5										
18.0										

1. 1B, 2B & 3B: Load Capacity for 1, 2 and 3 rows of bracing. 2. FR: Load Capacity for fully restrained compression flange. 3. W_s: Load at a deflection of span/150.

May 2004

Dimond

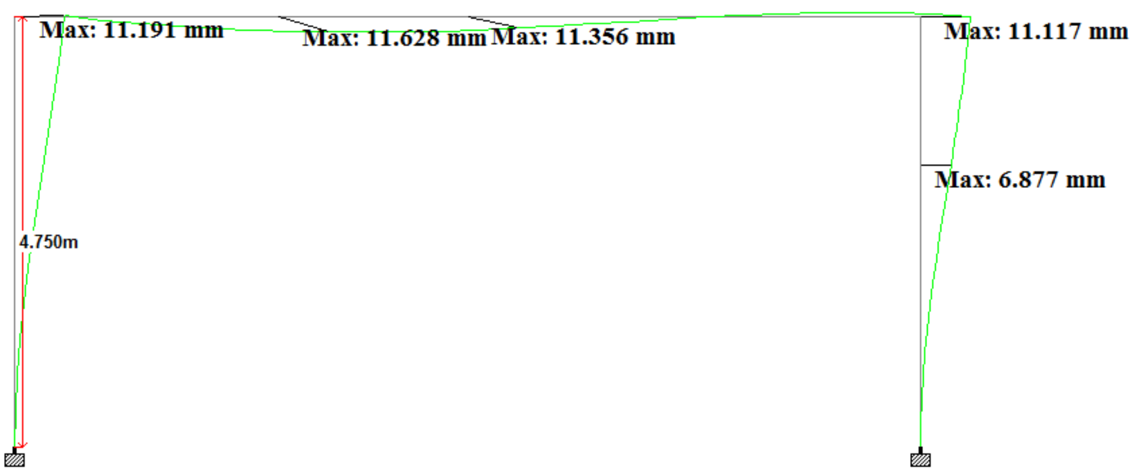
Deflection Limits:- (Table C1 NZS1170.0:2002)

Lateral Deflection:

Lateral Deflection for Earthquake.

Lateral Deflection for servicibility and ultimate.

Grid – 6 (1.0 DL + 1.0 Eq+Z)



Maximum Allowable Deflectuion for servicibility = Height /200 = 4750 / 200 = 23.75 mm

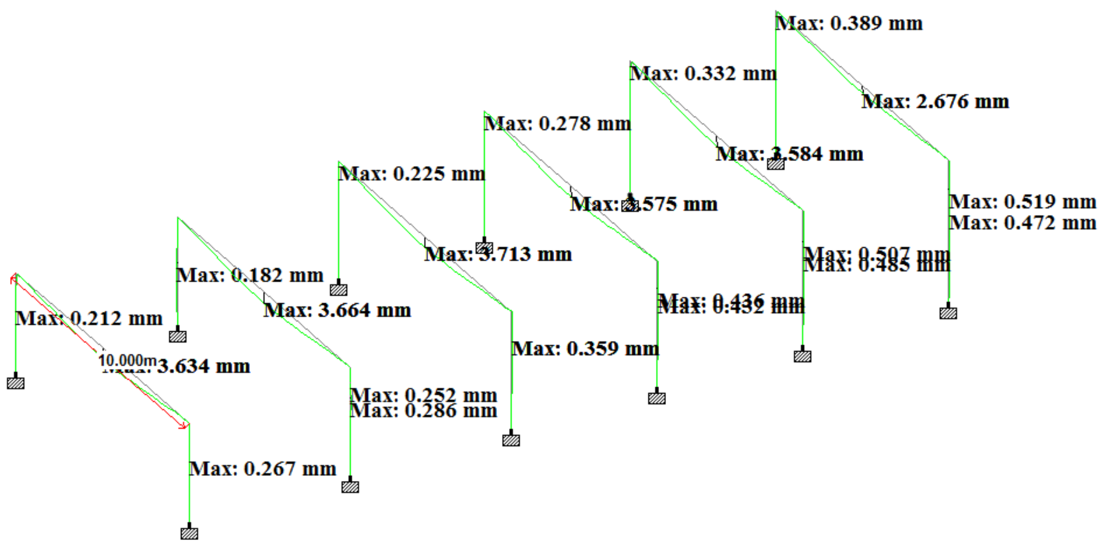
Maximum Allowable Deflectuion for ultimate = 2.5 % of Height = 4750 x 2.5 / 100 = 118.75 mm

Safe In Deflection.

Vertical Deflection:

For rafter beam for 1.0 DL case:

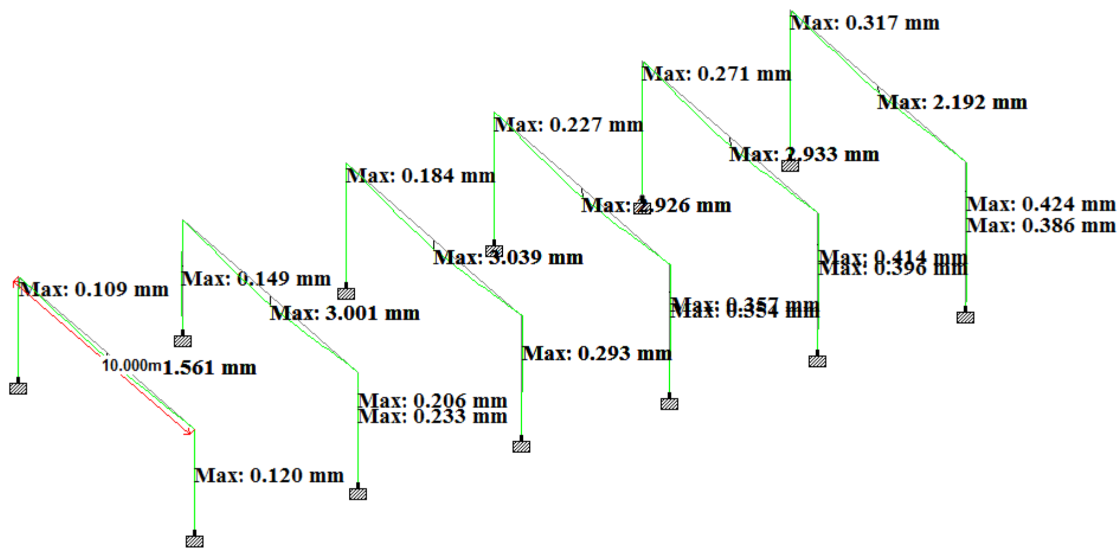
Maximum vertical deflection is = 3.713 mm



Maximum Allowable Deflectuion for 1.0 DL = Span /300 = 10000 / 300 = 33.33 mm (Hence safe)

For rafter beam for 1.0 LL case:

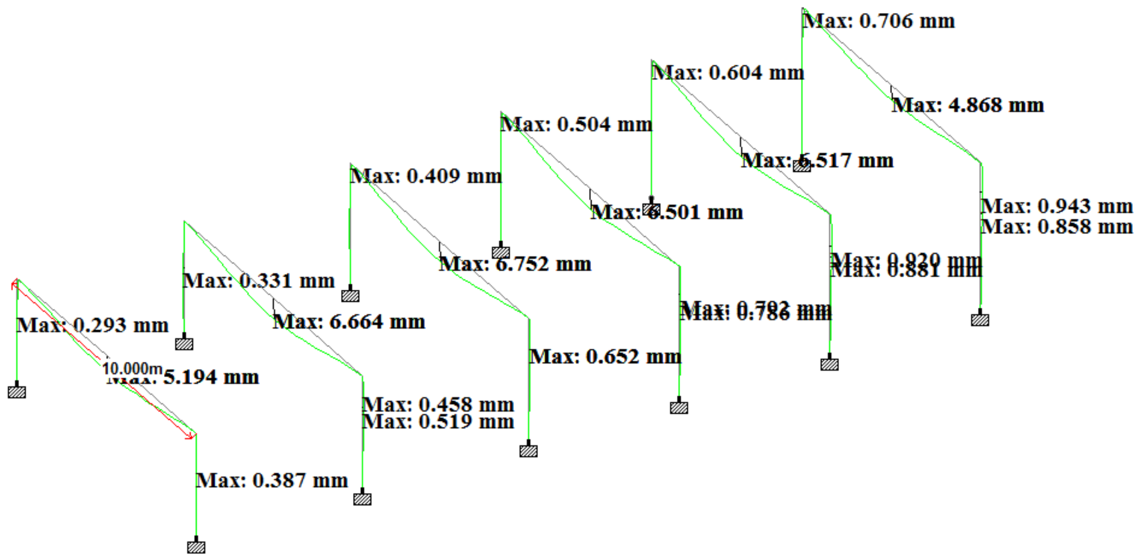
Maximum vertical deflection is = 3.039 mm



Maximum Allowable Deflectuion for 1.0 LL = Span /300 = 10000 / 300 = 33.33 mm (Hence safe)

For rafter beam for (1.0DL + 1.0 LL) case:

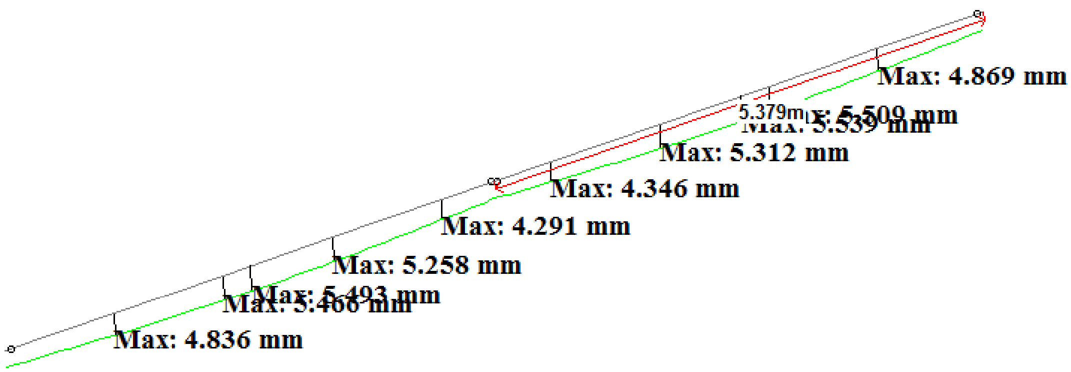
Maximum vertical deflection is = 6.752 mm



Maximum Allowable Deflectuion for 1.0DL + 1.0 LL = Span /300 = 10000 / 300 = 33.33 mm (Hence safe)

For Central beam for 1.0 DL case:

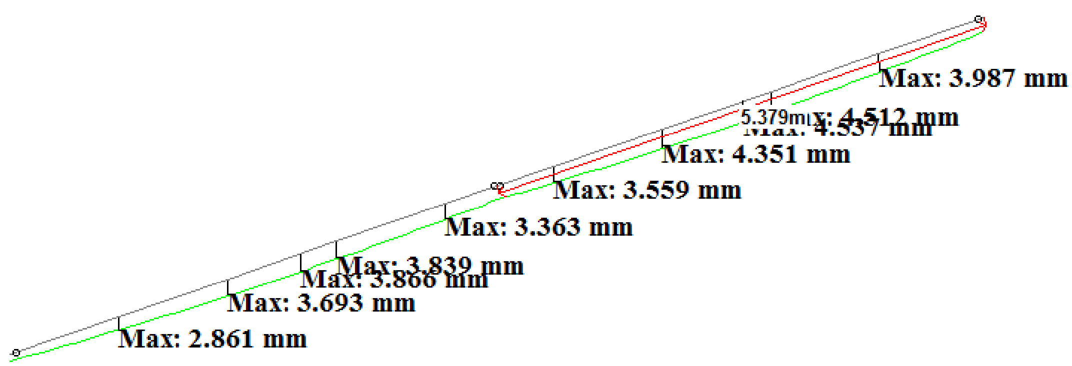
Maximum vertical deflection is = 5.539 mm



Maximum Allowable Deflectuion for 1.0 DL = Span /300 = 5379 / 300 = 17.93 mm (Hence safe)

For Central beam for 1.0 LL case:

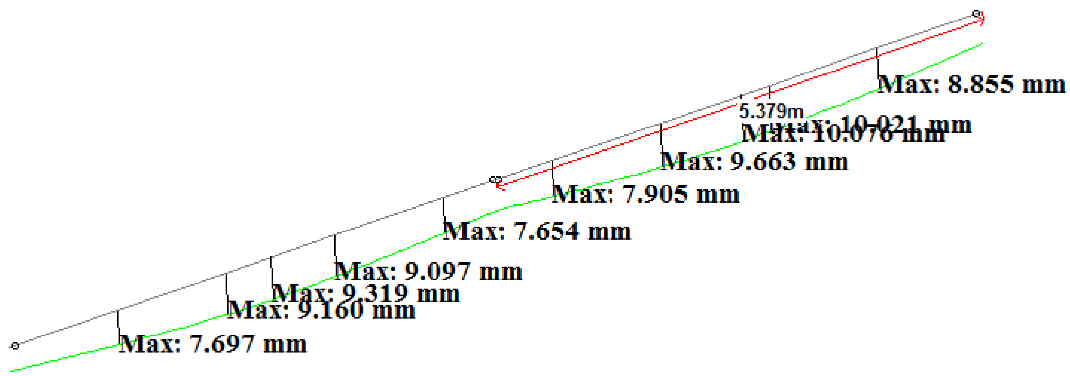
Maximum vertical deflection is = 4.512 mm



Maximum Allowable Deflectuion for 1.0 LL = Span /300 = 5379 / 300 = 17.93 mm (Hence safe)

For Central beam for (1.0DL + 1.0 LL) case:

Maximum vertical deflection is = 10.076 mm

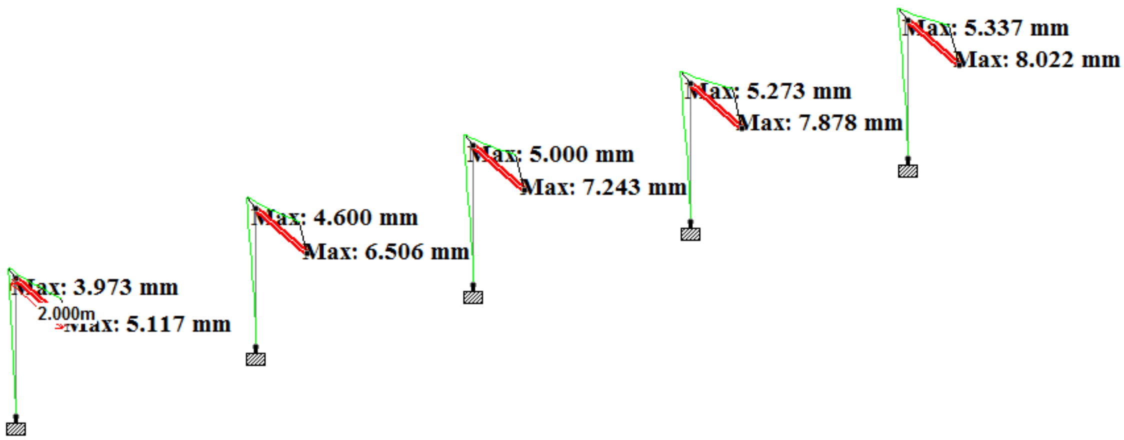


Maximum Allowable Deflectuion for 1.0DL + 1.0 LL = Span /300 = 5379 / 300 = 17.93 mm (Hence safe)

Uplift Deflection:

Uplift deflection for cantiliver canopy beam for (1.0DL + 1.0 WL - X (CPE-CPI)) case:

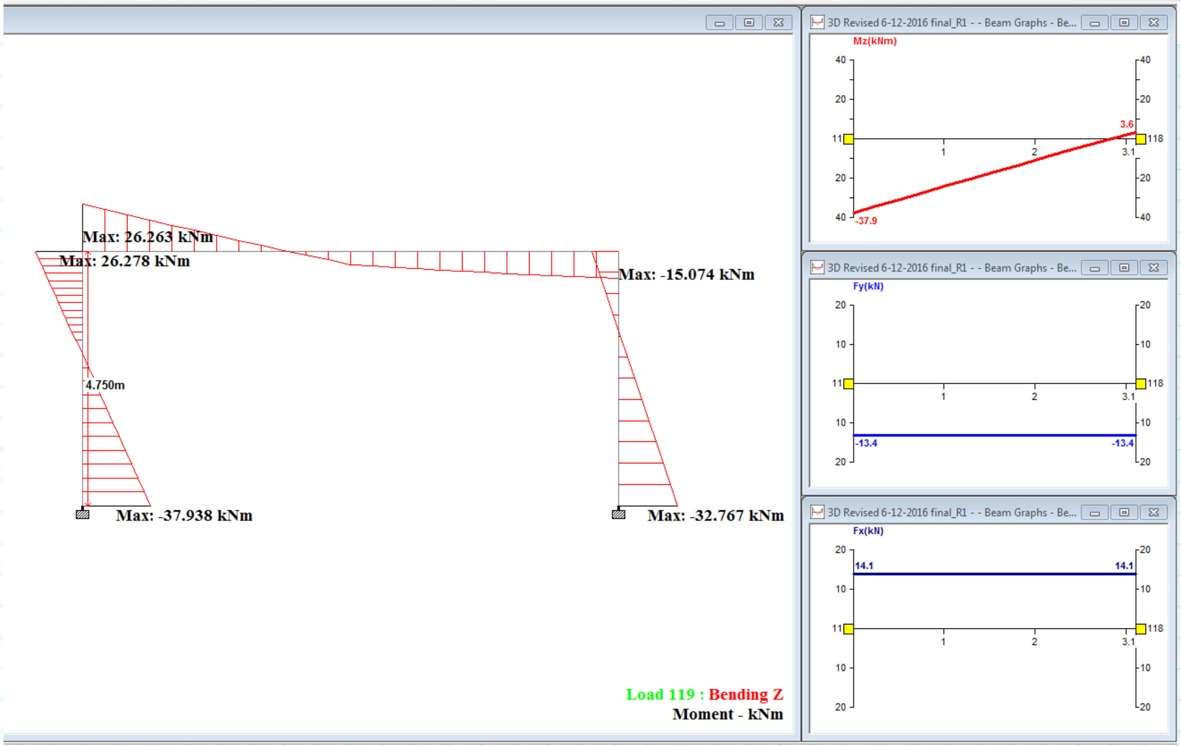
Maximum vertical deflection is = 8.022 mm



Maximum Allowable Deflectuion for (1.0DL + 1.0 WL - X(CPE+CPI)) = Span /100 = 2000 / 100 = 20 mm (Hence safe)

Design of Column 6B:-

B.M. , S.F. & Axial Force diagram for 119 LOAD CASE 1.0 DL + 1.0 EARTHQUAKE (+Z)



MemDes Calculations @ 17:25:23 06-12-2016
==
Project : 1333 Cameron Road Tauranga
Description :**COLUMN B6**

Section :**310UB40 Grade 300+**
d = 304 mm b = 165 mm tf = 10.2 mm tw= 6.1 mm
Area = 5210 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa
Ixx = 86.40 E6mm4 Sx = 633.00 E3mm3 Zx = 569.00 E3mm3 rx = 129.00 mm
Iyy= 7.65 E6mm4 Sy = 142.00 E3mm3 Zy = 92.70 E3mm3 ry = 38.30 mm
Iw =165.00 E9mm6 J =157.00 E3mm4 kf = 0.952 Manufact. Type = HR
Compactness(x,y) = C, C
Zex = 633.00 E3mm3 Zey = 139.00 E3mm3

Check of Section Category Requirements
Specified Minimum Section Category = 3
Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
**** MAJOR axis bending ****
Flange :Lambdae : Actual = 8.81, Allowable = 10.00 [OK]
[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.079 / 0.010 * 1.131 = 8.812]
Web : Lambdae : Actual = 52.60, Allowable = 101.00 [OK]
[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.284 / 0.006 * 1.131 = 52.599]
Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs

Torsional End Restraint Conds (TERC) are given as FL
=>kt = 1.00
User provided value of kl = 1.00
With End Restraints of FL, =>kr = 1.00
Eff. Len Le = L * kt * kl * kr = 4.75 * 1.00 * 1.00 * 1.00 = 4.75m

Major Axis Bending
Design Action M*x = 38.0kNm
Section Bending Capacity Msx = fyfZex = 202.56 kNm
User provided value for Alpha-m = 1.00
Reference Buckling Moment Calculation : Mo
Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 4.7502 = 89.7 E06
Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
Mo = 137.0 kNm(Eqtn 5.6.1.1(4))
Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.48 [(Ms/Mo) = 202.6 / 137.0 = 1.479]
Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained
Mbx = 1.00 * 0.48 * 202.6 = 97.1
Major axis capacity Ratio = M*x / Phi Mbx
= 0.43, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V*x = 13.4kN
Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.304 * 0.006
Nominal Shear Yield capacity Vw = 356.0 kN
Alphav = 2.43 >= 1.0 => full web shear capacity
Vu = Vw = 356.0 kN
Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
Shear capacity ratio = V*x / Phi Vu
= 0.04, ---- OK ----

Axial Calculations
Design Action Nd= 14.1kN [Comp], LeAxx = 4.75 m, LeAxy = 4.75 m
Sect. Compression Capacity Ns = kf An fycomb
= 0.952 * 5.2.E-3 * 320
= 1587.2 kN
(where An = Ag, &fy combined = 320.0 MPa
Major axis buckling : Alpha-c = 0.9022
Alpha-cx < 1.0 =>Ncx = Alpha-cx Ns = 1431.9
Minor axis buckling : Alpha-c = 0.3419
Alpha-cy < 1.0 =>Ncy = Alpha-cy Ns = 542.6
Minimum Capac.Ncmin = 542.6
Axial buckling capac. Ratio = Nd / Phi Ncmin
= 0.029, ---- OK ----

Combined Actions Checks
Cl 8.1.1.1 : Have axial load, in conjunction with flexure, => check Cl 8.1.4
Significant Axial Load test (Cl. 8.1.4(a)) NOT Satisfied **** NOK ****
Significant Axial Load test (Cl. 8.1.4(b) IS Satisfied ---- OK ----
Loading PASSES Cl 8.1.4, => Combined Actions Checks are not required

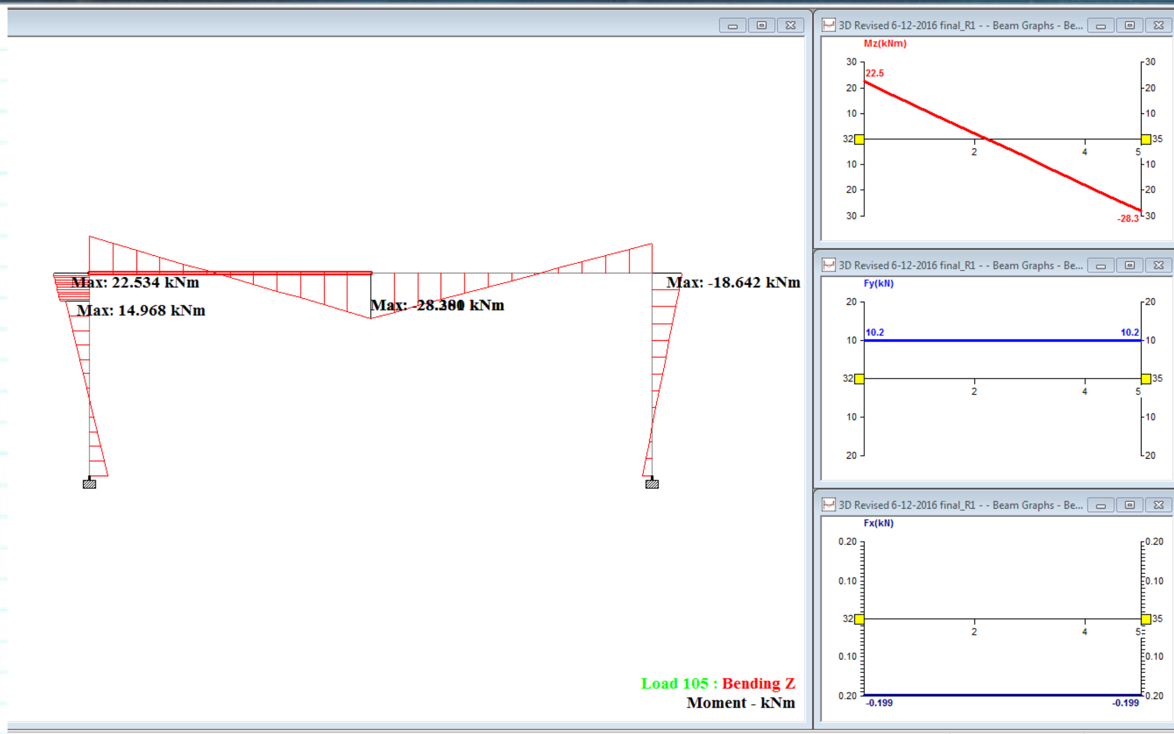
======================= SUMMARY ====================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.43 ---- OK ----

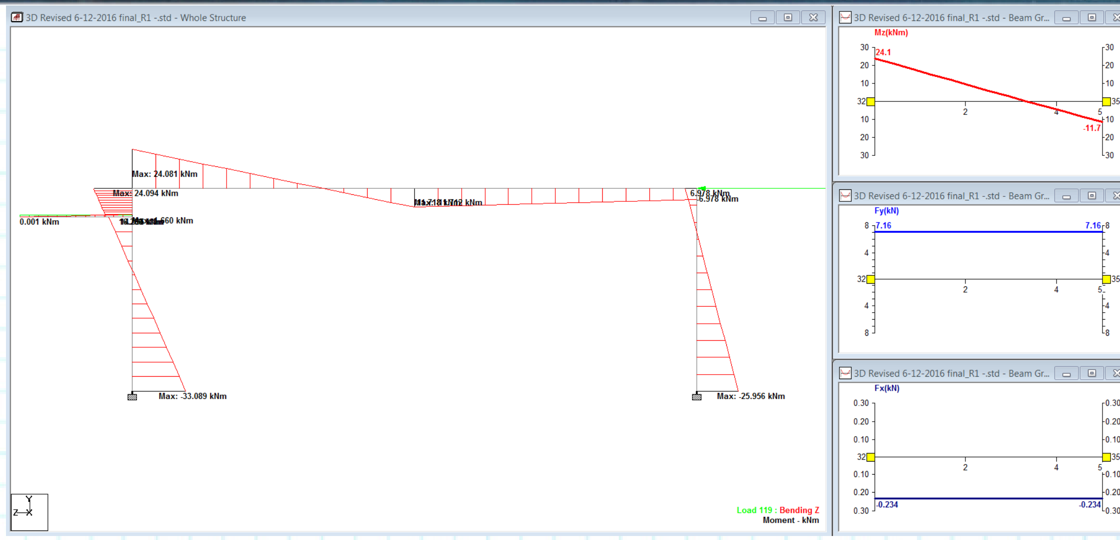
ΩMEGA ENGINEERING CONSULTANTS LIMITED	Project Name: 1333 Cameron Road Tauranga				Project No:
	For: Design Philosophy Report				
By:	Date: 16/1/2016	Checked By:	Checked Date:	Page No:	Rev:

Design of Rafter on Grid 2:-

B.M. , S.F. & Axial Force diagram for 105 (1.2 DL + 1.0 WL + Z (CPE+CPI)).



B.M. , S.F. & Axial Force diagram for 119 (1.0DL + 1.0 EARTHQUAKE + Z)



MemDes Calculations @ 17:37:18 06-12-2016

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Project : 1333 Cameron Road Tauranga

Description :**RAFTER ON GRID 2**

Section :**310UB40 Grade 300+**

d = 304 mm b = 165 mm tf = 10.2 mm tw= 6.1 mm
Area = 5210 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa
Ixx = 86.40 E6mm4 Sx = 633.00 E3mm3 Zx = 569.00 E3mm3 rx = 129.00 mm
Iyy= 7.65 E6mm4 Sy = 142.00 E3mm3 Zy = 92.70 E3mm3 ry = 38.30 mm
Iw =165.00 E9mm6 J =157.00 E3mm4 kf = 0.952 Manufact. Type = HR
Compactness(x,y) = C, C
Zex = 633.00 E3mm3 Zey = 139.00 E3mm3

Check of Section Category Requirements
Specified Minimum Section Category = 3
Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
**** MAJOR axis bending ****
Flange :Lambdae : Actual = 8.81, Allowable = 10.00 [OK]
[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.079 / 0.010 * 1.131 = 8.812]
Web : Lambdae : Actual = 52.60, Allowable = 101.00 [OK]
[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.284 / 0.006 * 1.131 = 52.599]
Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs
Torsional End Restraint Conds (TERC) are given as LL
=>kt = 1.00
User provided value of kl = 1.00
With End Restraints of LL, =>kr = 1.00
Eff. Len Le = L * kt * kl * kr = 10.00 * 1.00 * 1.00 * 1.00 = 10.00m

Major Axis Bending
Design Action M*x = 28.4kNm
Section Bending Capacity Msx = fyfZex = 202.56 kNm

User provided value for Alpha-m = 1.00
Reference Buckling Moment Calculation : Mo
Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 10.0002 = 20.2 E06
Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
Mo = 49.6kNm (Eqtn 5.6.1.1(4))
Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.21 [(Ms/Mo) = 202.6 / 49.6 = 4.083]
Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained
Mbx = 1.00 * 0.21 * 202.6 = 42.8
Major axis capacity Ratio = M*x / Phi Mbx
= 0.74, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V*x = 10.2kN
Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.304 * 0.006
Nominal Shear Yield capacity Vw = 356.0 kN
Alphav = 2.43 >= 1.0 => full web shear capacity
Vu = Vw = 356.0 kN
Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
Shear capacity ratio = V*x / Phi Vu
= 0.03, ---- OK ----
Axial Calculations
Design Action Nd = 0.2 kN [Comp], LeAxx = 10.00 m, LeAxy = 10.00 m
Sect. Compression Capacity Ns = kf An fycomb
= 0.952 * 5.2.E-3 * 320
= 1587.2 kN
(where An = Ag, &fy combined = 320.0 MPa
Major axis buckling : Alpha-c = 0.6414
Alpha-cx < 1.0 =>Ncx = Alpha-cx Ns = 1018.0
Minor axis buckling : Alpha-c = 0.0890
Alpha-cy < 1.0 =>Ncy = Alpha-cy Ns = 141.2
Minimum Capac.Ncmin = 141.2
Axial buckling capac. Ratio = Nd / Phi Ncmin
= 0.002, ---- OK ----

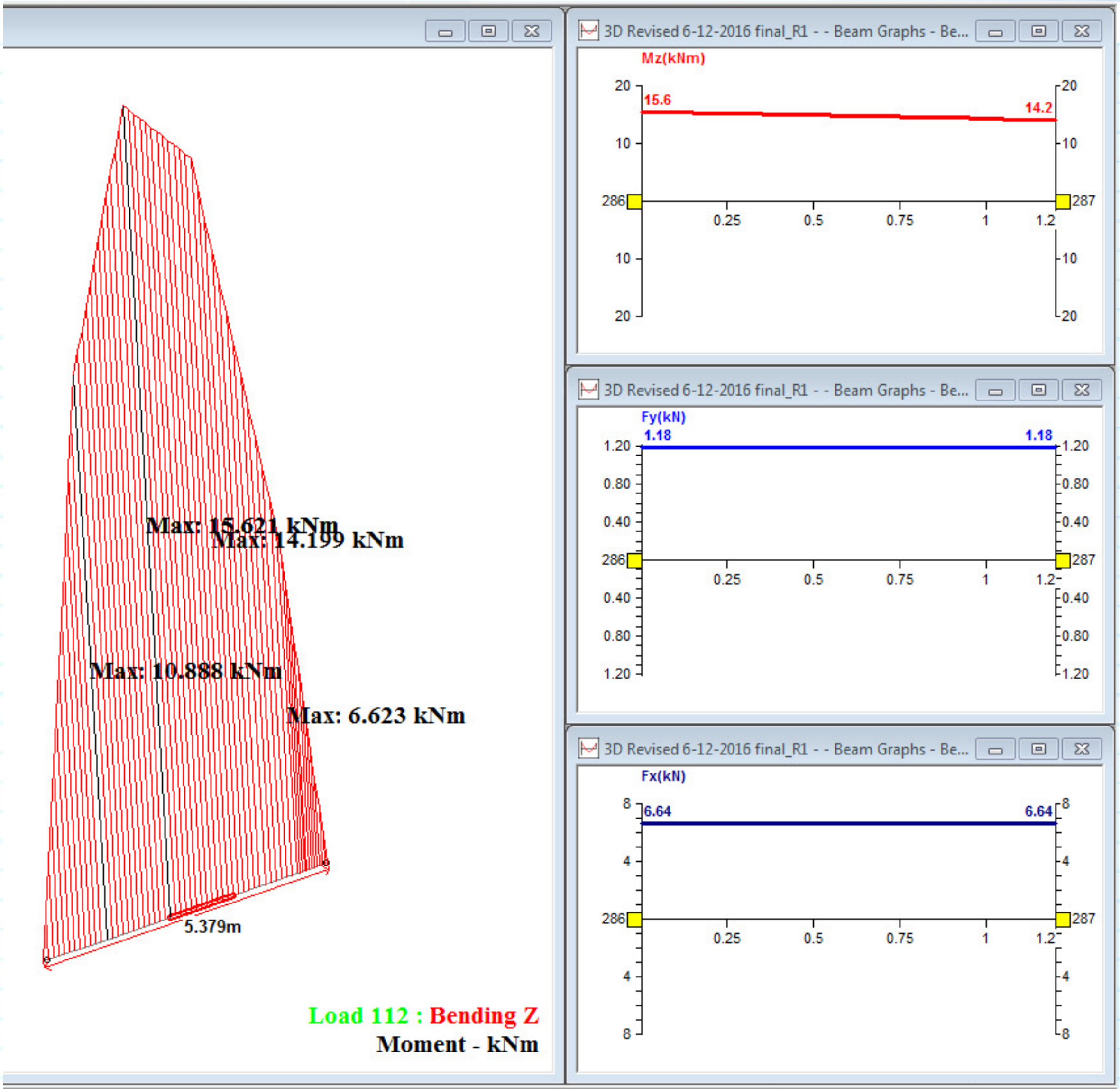
Combined Actions Checks
Cl 8.1.1.1 : Have axial load, in conjunction with flexure, => check Cl 8.1.4
Significant Axial Load test (Cl. 8.1.4(a)) NOT Satisfied **** NOK ****
Significant Axial Load test (Cl. 8.1.4(b) IS Satisfied ---- OK ----
Loading PASSES Cl 8.1.4, => Combined Actions Checks are not required

======================== SUMMARY =====================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.74 ---- OK ----

<u>Design of Central Beam At Rafter Level:-</u>

B.M. , S.F. & Axial Force diagram for LOAD CASE 112(0.9 DL + 1.0 WL – X (CPE + CPI)).



MemDes Calculations @ 17:45:42 06-12-2016
==
Project : 1333 Cameron Road Tauranga
Description :**CENTRAL BEAM AT RAFTER LVL.**

Section :**250UB31 Grade 300+**
d = 252 mm b = 146 mm tf= 8.6 mm tw = 6.1 mm
Area = 4010 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa
Ixx = 44.50 E6mm4 Sx = 397.00 E3mm3 Zx = 354.00 E3mm3 rx = 105.00 mm
Iyy= 4.47 E6mm4 Sy = 94.20 E3mm3 Zy = 61.20 E3mm3 ry = 33.40 mm

Iw = 65.90 E9mm6 J = 89.30 E3mm4 kf = 1.000 Manufact. Type = HR
Compactness(x,y) = N, N
Zex = 395.00 E3mm3 Zey = 91.40 E3mm3

Check of Section Category Requirements
Specified Minimum Section Category = 3
Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
**** MAJOR axis bending ****
Flange :Lambdae : Actual = 9.20, Allowable = 10.00 [OK]
[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.070 / 0.009 * 1.131 = 9.202]
Web : Lambdae : Actual = 43.55, Allowable = 101.00 [OK]
[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.235 / 0.006 * 1.131 = 43.549]
Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs
Torsional End Restraint Conds (TERC) are given as LL
=>kt = 1.00
User provided value of kl = 1.00
With End Restraints of LL, =>kr = 1.00
Eff. Len Le = L * kt * kl * kr = 5.38 * 1.00 * 1.00 * 1.00 = 5.38m

Major Axis Bending
Design Action M*x = 15.6kNm
Section Bending Capacity Msx = fyfZex = 126.40 kNm
User provided value for Alpha-m = 1.00
Reference Buckling Moment Calculation : Mo
Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 5.3792 = 69.9 E06
Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
Mo = 60.6kNm (Eqtn 5.6.1.1(4))
Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.38 [(Ms/Mo) = 126.4 / 60.6 = 2.085]
Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained
Mbx = 1.00 * 0.38 * 126.4 = 47.4
Major axis capacity Ratio = M*x / Phi Mbx
= 0.37, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V*x = 1.2 kN
Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.252 * 0.006
Nominal Shear Yield capacity Vw = 295.1 kN
Alphav = 3.55 >= 1.0 => full web shear capacity
Vu = Vw = 295.1 kN
Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
Shear capacity ratio = V*x / Phi Vu
= 0.00, ---- OK ----

Axial Calculations
Design Action Nd = 6.7 kN [Comp], LeAxx = 5.38 m, LeAxy = 5.38 m
Sect. Compression Capacity Ns = kf An fycomb
= 1.000 * 4.0.E-3 * 320
= 1283.2 kN
(where An = Ag, &fy combined = 320.0 MPa
Major axis buckling : Alpha-c = 0.8199
Alpha-cx < 1.0 =>Ncx = Alpha-cx Ns = 1052.1

Minor axis buckling : Alpha-c = 0.2086
Alpha-cy < 1.0 =>Ncy = Alpha-cy Ns = 267.7
Minimum Capac.Ncmin = 267.7
Axial buckling capac. Ratio = Nd / Phi Ncmin
= 0.028, ---- OK ----

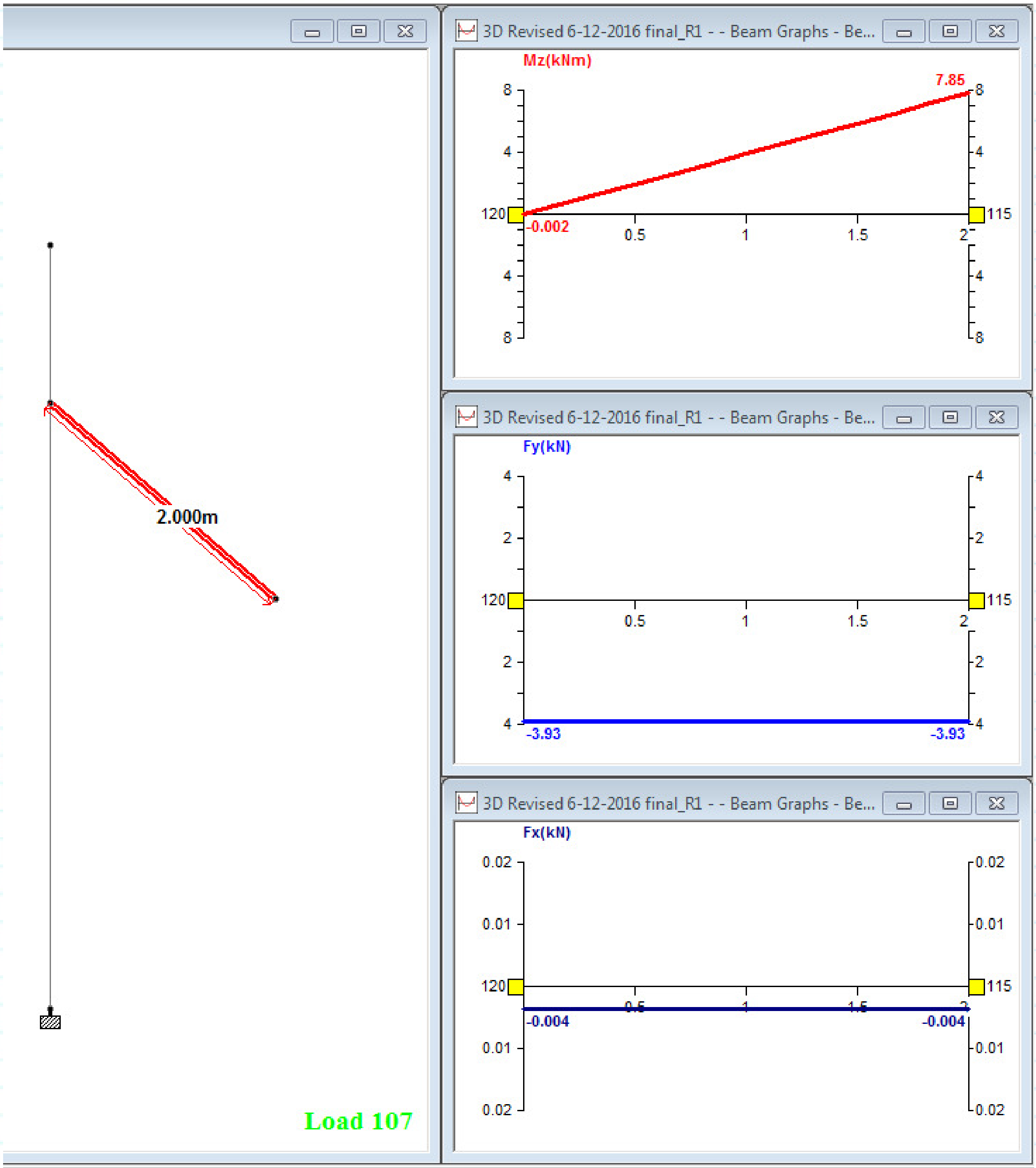
Combined Actions Checks
Cl 8.1.1.1 : Have axial load, in conjunction with flexure, => check Cl 8.1.4
Significant Axial Load test (Cl. 8.1.4(a)) NOT Satisfied **** NOK ****
Significant Axial Load test (Cl. 8.1.4(b) IS Satisfied ---- OK ----
Loading PASSES Cl 8.1.4, => Combined Actions Checks are not required

======================= SUMMARY ====================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.37 ---- OK ----

<u>Canopy Main Beam (Grid -3)</u>

B.M. , S.F. & Axial Force diagram for load Case 107 (1.2 DL + 1.0 WL + X (CPE - CPI)).



MemDes Calculations @ 17:55:22 06-12-2016
==
Project : 1333 Cameron Road Tauranga
Description **CANOPY MAIN BEAM**

Section :**150UB14 Grade 300+**
d = 150 mm b = 75 mm tf = 7.0 mm tw = 5.0 mm
Area = 1780 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa
Ixx= 6.66 E6mm4 Sx = 102.00 E3mm3 Zx = 88.80 E3mm3 rx = 61.10 mm
Iyy= 0.49 E6mm4 Sy = 20.80 E3mm3 Zy = 13.20 E3mm3 ry = 16.60 mm
Iw= 2.53 E9mm6 J = 28.10 E3mm4 kf = 1.000 Manufact. Type = HR
Compactness(x,y) = C, C
Zex = 102.00 E3mm3 Zey = 19.80 E3mm3

Check of Section Category Requirements
Specified Minimum Section Category = 3
Maximum Yield Stress Requirement Satisfied (Tbl 12.4)
NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***
**** MAJOR axis bending ****
Flange :Lambdae : Actual = 5.66, Allowable = 10.00 [OK]
[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.035 / 0.007 * 1.131 = 5.657]
Web : Lambdae : Actual = 30.77, Allowable = 101.00 [OK]
[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.136 / 0.005 * 1.131 = 30.773]
Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs
Torsional End Restraint Conds (TERC) are given as FL
=>kt = 1.00
User provided value of kl = 1.00
With End Restraints of FL, =>kr = 1.00
Eff. Len Le = L * kt * kl * kr = 2.00 * 1.00 * 1.00 * 1.00 = 2.00m

Major Axis Bending
Design Action M*x = 7.9 kNm
Section Bending Capacity Msx = fyfZex = 32.64 kNm
User provided value for Alpha-m = 1.00
Reference Buckling Moment Calculation : Mo
Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 2.0002 = 505.8 E06
Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5
Mo = 29.7kNm (Eqtn 5.6.1.1(4))
Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.57 [(Ms/Mo) = 32.6 / 29.7 = 1.098]
Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained
Mbx = 1.00 * 0.57 * 32.6 = 18.7
Major axis capacity Ratio = M*x / Phi Mbx
= 0.47, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V*x = 4.0 kN
Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.150 * 0.005
Nominal Shear Yield capacity Vw = 144.0 kN
Alphav = 7.10 >= 1.0 => full web shear capacity
Vu = Vw = 144.0 kN
Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A
Shear capacity ratio = V*x / Phi Vu
= 0.03, ---- OK ----

======================= SUMMARY ====================

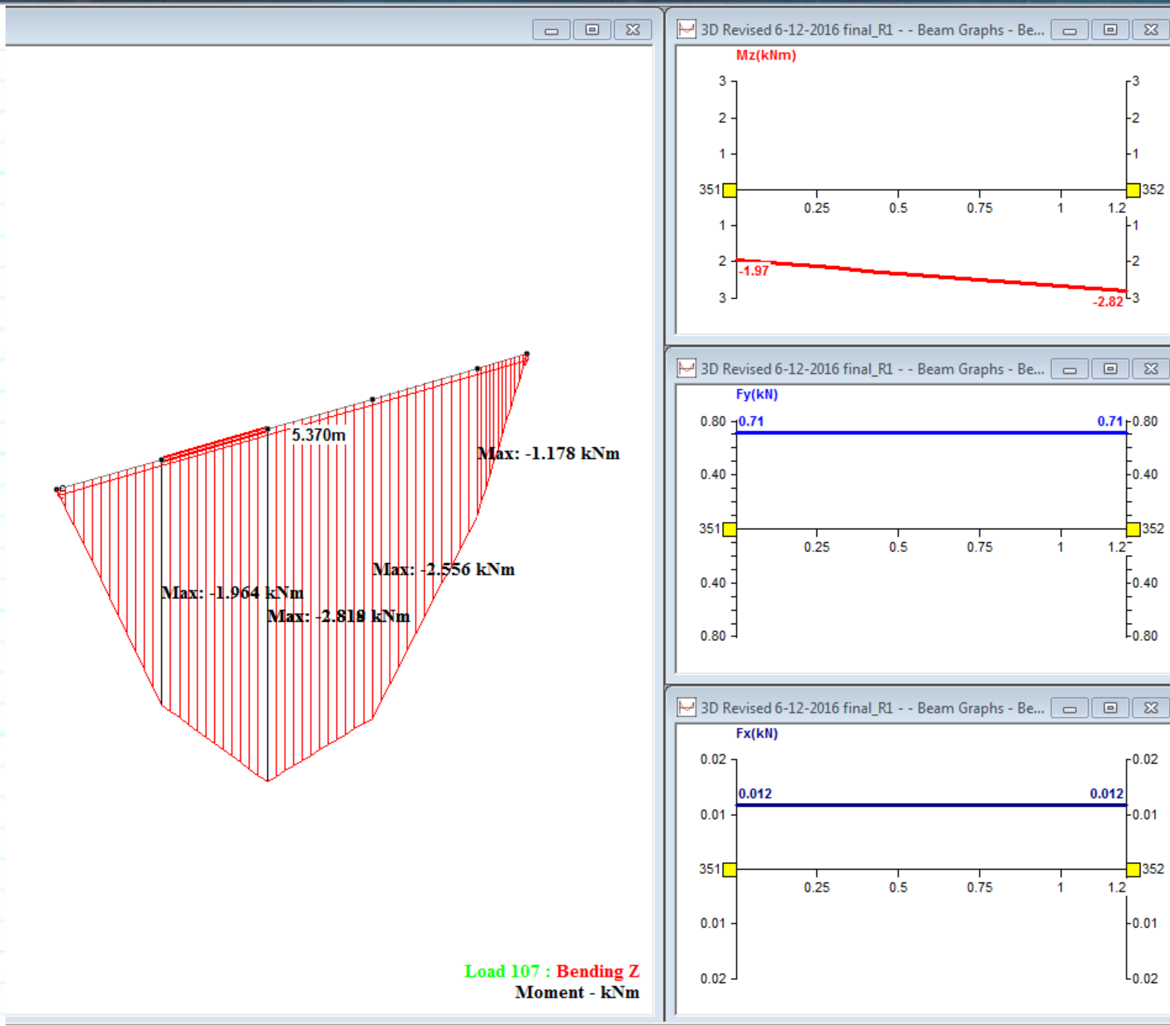
**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.47 ---- OK ----

Canopy Secondary Front Beam

B.M. , S.F. & Axial Force diagram for load Case 107 (1.2 DL + 1.0 WL + X (CPE - CPI)).

ΩMEGA ENGINEERING CONSULTANTS LIMITED	Project Name: 1333 Cameron Road Tauranga	Project No:
	For: Design Philosophy Report	

By:	Date: 16/1/2016	Checked By:	Checked Date:	Page No:	Rev:



MemDes Calculations @ 16:28:54 24-12-2016

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Project : 1333 Cameron Road Tauranga

Description : ***canopy secondary beam (front)***

Section : ***150PFC Grade 300+***

d = 150 mm b = 75 mm tf = 9.5 mm tw = 6.0 mm

Area = 2250 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa

Ixx = 8.34 E6mm4 Sx = 129.00 E3mm3 Zx = 111.00 E3mm3 rx = 60.80 mm

Iyy = 1.29 E6mm4 Sy = 46.00 E3mm3 Zy = 25.70 E3mm3 ry = 23.90 mm

Iw = 4.59 E9mm6 J = 54.90 E3mm4 kf = 1.000 Manufact. Type = HR

Zex = 129.00 E3mm3 Zey = 38.50 E3mm3

Check of Section Category Requirements

Specified Minimum Section Category = 3

Maximum Yield Stress Requirement Satisfied (Tbl 12.4)

NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***

**** MAJOR axis bending ****

Flange : Lambdae : Actual = 8.22, Allowable = 10.00 [OK]

[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.069 / 0.010 * 1.131 = 8.217]

Web : Lambdae : Actual = 24.70, Allowable = 101.00 [OK]

[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.131 / 0.006 * 1.131 = 24.702]

Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs

Torsional End Restraint Conds (TERC) are given as LL

=> kt = 1.00

User provided value of kl = 1.00

User provided value of kr = 1.00

Eff. Len Le = L * kt * kl * kr = 5.37 * 1.00 * 1.00 * 1.00 = 5.37m

Major Axis Bending

Design Action M*x = 2.8 kNm

Section Bending Capacity Msx = fyf Zex = 41.28 kNm

User provided value for Alpha-m = 1.00

Reference Buckling Moment Calculation : Mo

Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 5.3702 = 70.2 E06

Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5

Mo = 20.7 kNm (Eqtn 5.6.1.1(4))

Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.39 [(Ms/Mo) = 41.3 / 20.7 = 1.998]

Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained

Mbx = 1.00 * 0.39 * 41.3 = 16.0

Major axis capacity Ratio = M*x / Phi Mbx

= 0.19, ---- OK ----

Shear Calculations (Unstiffened Web)

Design Action V*x = 0.7 kN

Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.150 * 0.006

Nominal Shear Yield capacity Vw = 172.8 kN

Alphav = 9.58 >= 1.0 => full web shear capacity

Vu = Vw = 172.8 kN

Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A

Shear capacity ratio = V*x / Phi Vu

= 0.00, ---- OK ----

======================== SUMMARY =====================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.19 ---- OK ----

<u>Canopy Channel supporting purlin at tilt concrete wall</u>

Total Load on roof = 77.16 kN

assume that half tributerry area load will be transfer to channel

Total Load on roof transfer to channel = 77.16 / 2 kN = 38.58 kN

Length of building = 25.8 m

load on chanel per meter = 1.5 kN/m

moment on channel considering length of chanel from c/c span of column

= 1.5 x 5.37^2/ 8 = 5.40 kN.m

Shear on channel

= 1.5 x 5.37 /2 = 4.02 kN

MemDes Calculations @ 12:04:58 26-12-2016

==

Project : 1333 Cameron Road Tauranga

Description : Channel design supporting purlin

Section : **125PFC Grade 300+**

d = 125 mm b = 65 mm tf = 7.5 mm tw = 4.7 mm

Area = 1520 mm2 fyf = 320 MPa fyw = 320 MPa fu = 440 MPa

Ixx = 3.97 E6mm4 Sx = 73.00 E3mm3 Zx = 63.50 E3mm3 rx = 51.10 mm

Iyy = 0.66 E6mm4 Sy = 27.20 E3mm3 Zy = 15.20 E3mm3 ry = 20.80 mm

Iw = 1.64 E9mm6 J = 23.10 E3mm4 kf = 1.000 Manufact. Type = HR

Zex = 72.80 E3mm3 Zey = 22.80 E3mm3

Check of Section Category Requirements

Specified Minimum Section Category = 3

Maximum Yield Stress Requirement Satisfied (Tbl 12.4)

NOTE : Designer to ensure that Clause 12.8.3.3 is complied with ***

**** MAJOR axis bending ****

Flange : Lambdae : Actual = 9.10, Allowable = 10.00 [OK]

[Lambdae = FlgOutStand / FlgThick * (fyf/250)1/2 = 0.060 / 0.008 * 1.131 = 9.096]

Web : Lambdae : Actual = 26.48, Allowable = 101.00 [OK]

[Lambdae = WebDepth / WebThick * (fyw/250)1/2 = 0.110 / 0.005 * 1.131 = 26.479]

Section Geometry Check [Major] : Passed ---- OK ----

Member Effective Length Calcs

Torsional End Restraint Conds (TERC) are given as FU

=> kt = 1.00

User provided value of kl = 1.00

User provided value of kr = 1.00

Eff. Len Le = L * kt * kl * kr = 5.37 * 1.00 * 1.00 * 1.00 = 5.37m

Major Axis Bending

Design Action M*x = 5.4 kNm

Section Bending Capacity Msx = fyf Zex = 23.30 kNm

User provided value for Alpha-m = 1.00

Reference Buckling Moment Calculation : Mo

Temp. variable Pi2_E_Le2 = PI2 x E / Le2 = 3.1422 x 205 E6 / 5.3702 = 70.2 E06

Mo = [(Pi2_E_Le2 * Iy) * (G J +(Pi2_E_Le2 * Iw))]0.5

Mo = 9.5 kNm (Eqtn 5.6.1.1(4))

Alpha-s = 0.6*[((Ms/Mo)2+3)0.5 - (Ms/Mo)] = 0.33 [(Ms/Mo) = 23.3 / 9.5 = 2.447]

Alpha-m Alpha-s < 1.0, => Segment NOT Fully Restrained

Mbx = 1.00 * 0.33 * 23.3 = 7.7

Major axis capacity Ratio = M*x / Phi Mbx

= 0.78, ---- OK ----

Shear Calculations (Unstiffened Web)

Design Action V*x = 4.0 kN

Vw = 0.6 * Fyw * d * tw = 0.6 * 320000 * 0.125 * 0.005

Nominal Shear Yield capacity Vw = 112.8 kN

Alphav = 8.40 >= 1.0 => full web shear capacity

Vu = Vw = 112.8 kN

Mom-Shear Interaction Check : Moment ratio <= 0.75 => Clause 5.12.2 N/A

Shear capacity ratio = V*x / Phi Vu

= 0.04, ---- OK ----

Seismic Member Checks

Cl. 12.6.2.2 check : Alpha-m x Alpha-s Limit

Alpha-m x Alpha-s = 0.33, Tbl 12.6.2 Limit of 1.00 is NOT met, => NOK

Designer to check whether segment has yielding regions or not

[NOTE : If Member Category of 1, check if a value other than 1.35 may be used]

Cl 12.6.2.5 check, Full Lateral Restraint is not required, OK

Tbl 12.8.3.1(b) Check : Not req'd for a non-compression or non-yielding seismic member

======================= SUMMARY ====================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.78 ---- OK ----

<u>Design of RB 16 Roof Bracing</u>

Axial tension force in memberN* = 3.47 kN

for design of tension member $N^* \leq Ø\, Nt$

Design Tensile capacity = ØNt = 0.9 x 201 x 300

= 54.27 > 3.47 kN.Hence O.K.

DESIGN OFCONCRETE TILT PANEL

Calculation of center of Rigidity

W1 = weight of wall at grid A

W2 = weight of wall at grid B

Thickness of concrete tilt panel = 150 mm

COR = w1 x 0.075 + w2 x (10.235 - 0.075) / (w1 + w2)

= 227.47 x 0.075 + 51.21 x 10.16 / 278.68

= 1.92

Calculation of center of Mass

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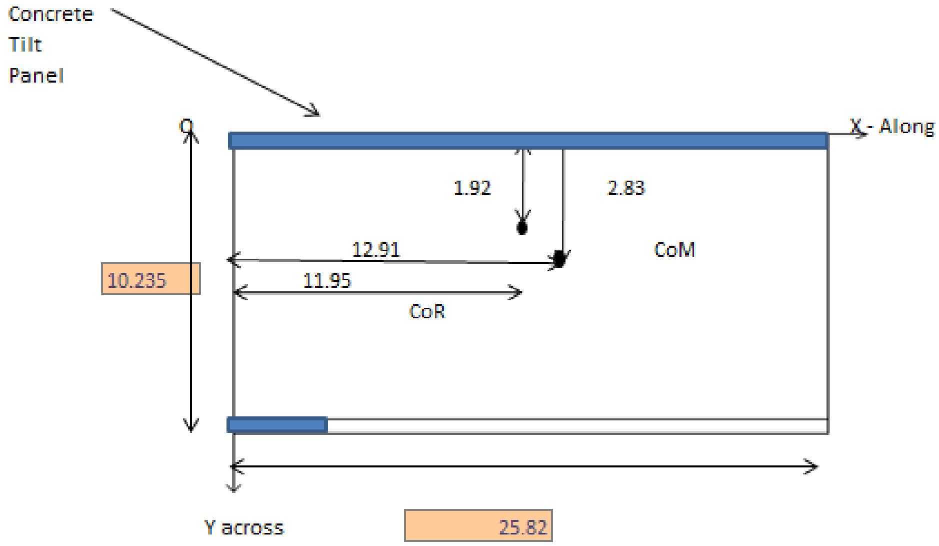
ALONG DIRECTION

	weight	C.G	Moment
wall GL - A	227.47	12.91	2936.64
wall GL - B	51.21	3.03	155.166
wall GL - 1	10.18	0.28	2.8504
wall GL - 6	20.5575	25.82	530.795
Roof	77.165	12.91	996.2
	386.583		4621.65
	x =	11.9551	

ACROSS DIRECTION

	weight	C.G	Moment
wall GL - A	227.47	0.075	17.0603
wall GL - B	51.21	10.235	524.134
wall GL - 1	10.18	5.11	52.0198
wall GL - 6	20.5575	5.11	105.049
Roof	77.165	5.11	394.313
	386.583		1092.58
	y =	2.82624	



Accidental Eccentricity :

X	=	0.1 * X
	=	0.1 x 25.82
	=	2.582 m
Y	=	0.1 * Y
	=	0.1 x 10.235
	=	1.03 m

Eccentricity :

X	=	12.91-11.95
	=	0.96 m
Y	=	2.83-1.92
	=	0.91

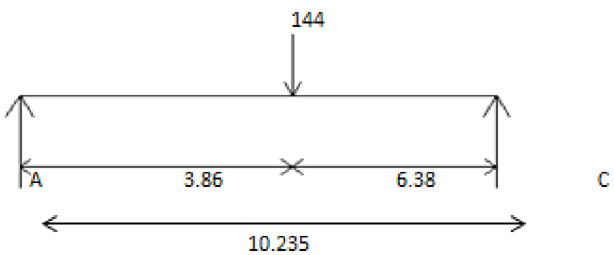
Total Eccentricity :

X	=	2.582+0.960000000000001
	=	3.542 m
X	=	2.582-0.960000000000001
	=	1.622 m
Y	=	0.91+1.03
	=	1.94 m
Y	=	0.91-1.03
	=	-0.12 m

Mass	=	386.583 kN
Seismic Weight	=	0.372 X 386.583
	=	144 kN

Along Direction :

Two Concrete tilt panel walls :



= (3.86 x 144)/(10.235)

= 54.4 kN

Design Walls on grid C 54.4 kN in plane

Walls = 2 Nos

Length = 2.569 m

2 walls per side @ 2.569 m in length each

Force per panel = 54.4/2

= 27.2 kN

Reaction on grid A wall

144-54.4= 89.6

Walls = 10 Nos

Length = 2.569 m

10 walls per side @ 2.569 m in length each

Force per panel = 89.6/10

= 8.96 kN

Across Direction :

Wall on Grid A and Grid C are in same dirction and the provide resistance to momnets in same along directio. So forces need to be add with across dirction

M = 3.542 x 144

= 510.048 kNm

L = 10.235 m

Force per wall = 510.048/10.235

= 49.9 kN

2 panels = 49.9/2

= 24.95 kN per panel

On Grid C total Force = 27.2+24.95

52.15 per panel

Now total load of concrete wall as from pg no. 4 is 3.6kN/m^2.

Seismic mass coefficient = 0.372

total weight of wall = 0.372 x 3.6 = 1.34kN/m^2.

= load on one panel = 1.34 x 2.569 = 3.45 kN/m

moment due to seismic load = 3.45 x 5.4^2/ 8 = 12.57 kN.m

Now check for fire load.
width of one panel = 2.569 m

= load on one panel = 0.5 x 2.569 = 1.28 kN/m

moment = 1.28 x 5.4^2/ 2 = 18.72 kN.m

Fire load is more critical than seismic for out of plane.

lets Try HD12 @ 125 c/c As = 904 mm^2/m

a = As x fy / 0.85 fc b

a = 904 x 500 / 0.85 x 30 x 1000 = 17.8 mm

ØM = 0.85 fy As (d - a/2)

= 0.85 x 500 x 904 (75 - 17.8 / 2)

= 25.39 kN.m> 18.72 kN.m

Check for Shear

Shear on one Panel = 1.28 x 5.4 = 7

Vc = 0.2 x fc ^0.5 x 1000 x 75 = 82.5 kN

Vs = V/0.75 - 82.5 < 0

As = 0.7bws2 / fy

consider 450 mm spacing

= 0.7 x 150 x 450 / 500

= 94.5 mm^2

take HD12 @ 450 c/c .

Now, for In plane the force apply on one panel is 52.15 kN

Max height of roof is 5.4 m.

Mmax = 52.15 x 5.4 = 281.61 kN.m

For determining the in plane capacity, we will use outer 4 bars of the wall

4HD12 = 452 mm^2 d = 2569 - 50 - (125 x 2) = 2269

a = As x fy / 0.85 fc b

a = 452 x 500 / 0.85 x 30 x 125 = 71 mm

ØM = 0.85 fy As (d - a/2)

= 0.85 x 500 x 452 (2269 - 71 / 2)

= 429.05 kN.m> 281.61kN.m

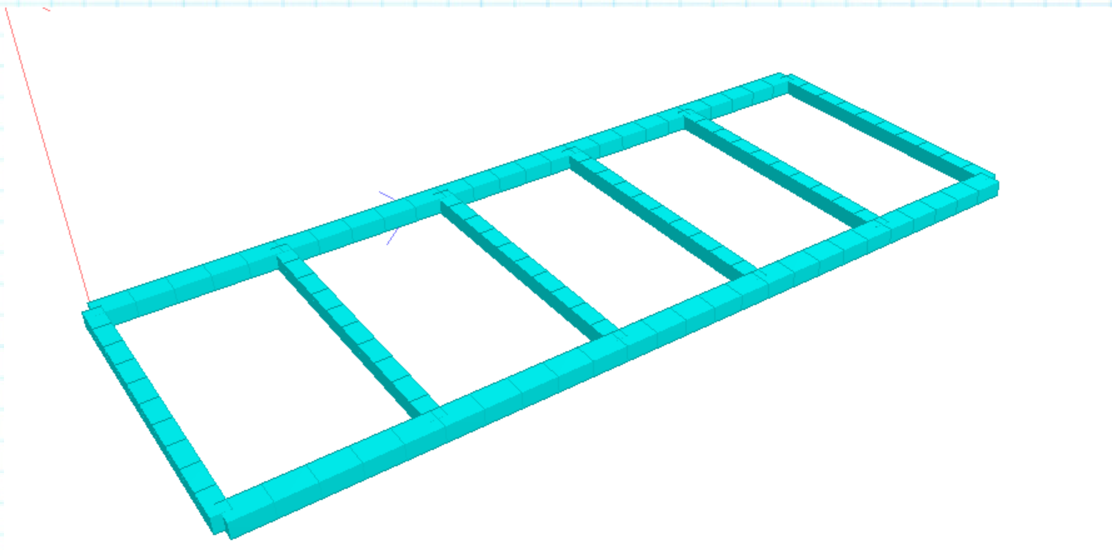
Hence O.K.

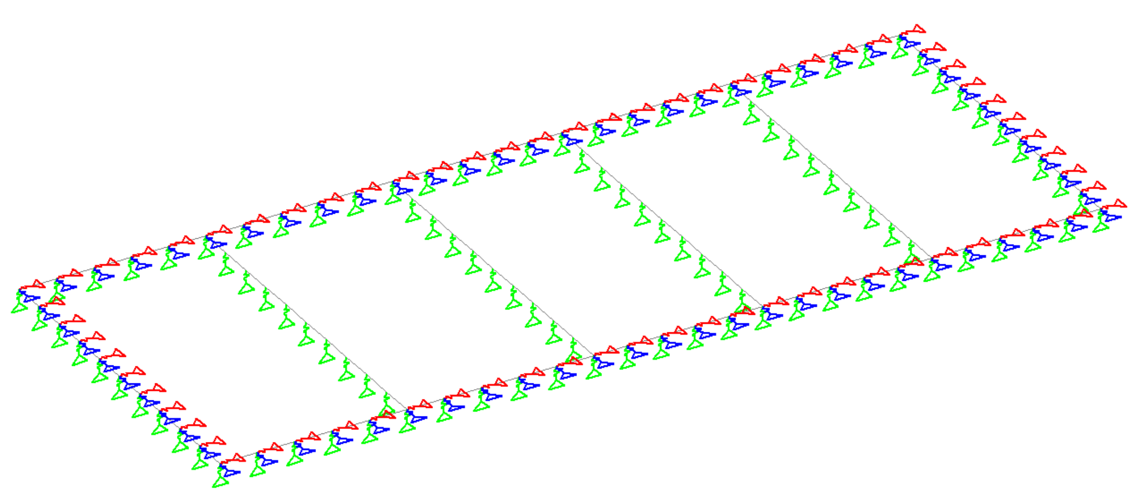
Calculation of Ground Beam

Here for design of ground beam, we generate a staad model of ground beam. the reaction obtain from main geometry column apply on the same point in ground beam staad model as joint load. For applying joint load, we have generated all the load cases which was consider in main model. And same load combination is also consider.

For generation staad model, we have generate spring support at 1 m interval. for generation of spring support the value of sub grade reaction taken is 260/0.01 x 0.750 = 19500 kN/m^3

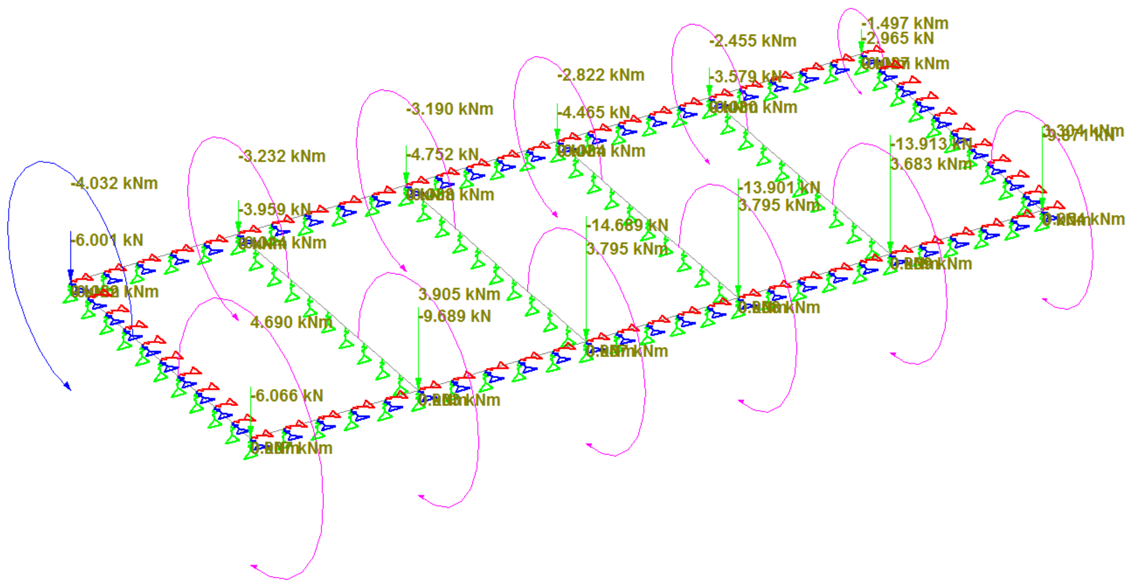
Geometry Model (3D) :



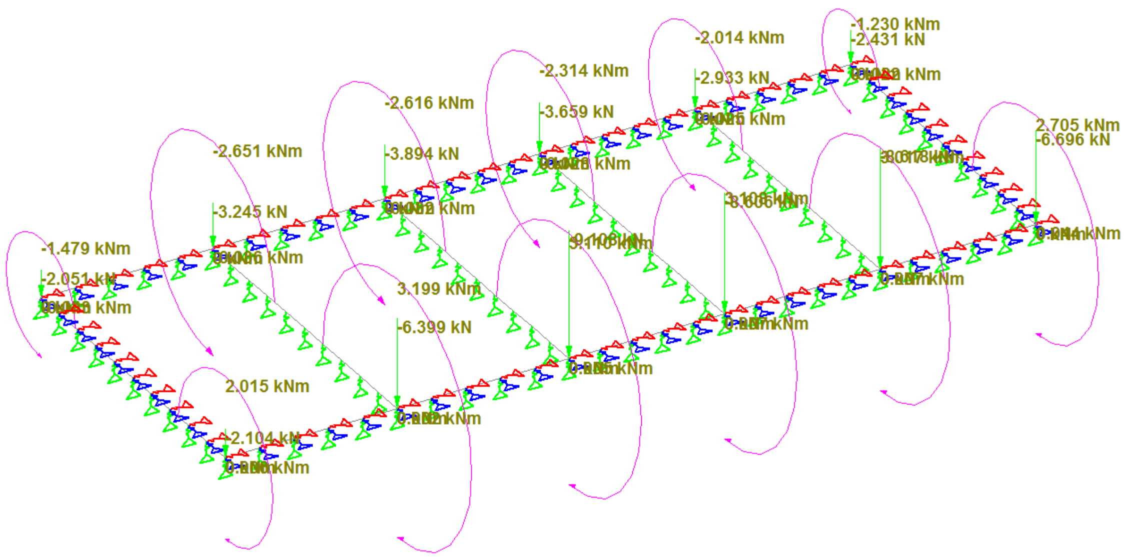


Load Data :

Dead Load :



Live load :



Wind +X (CPE + CPI):

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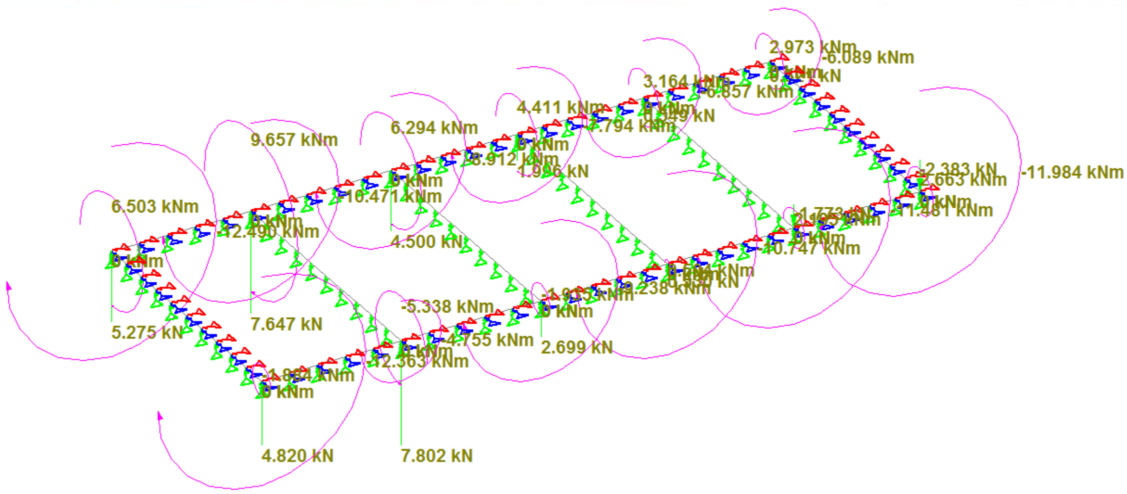

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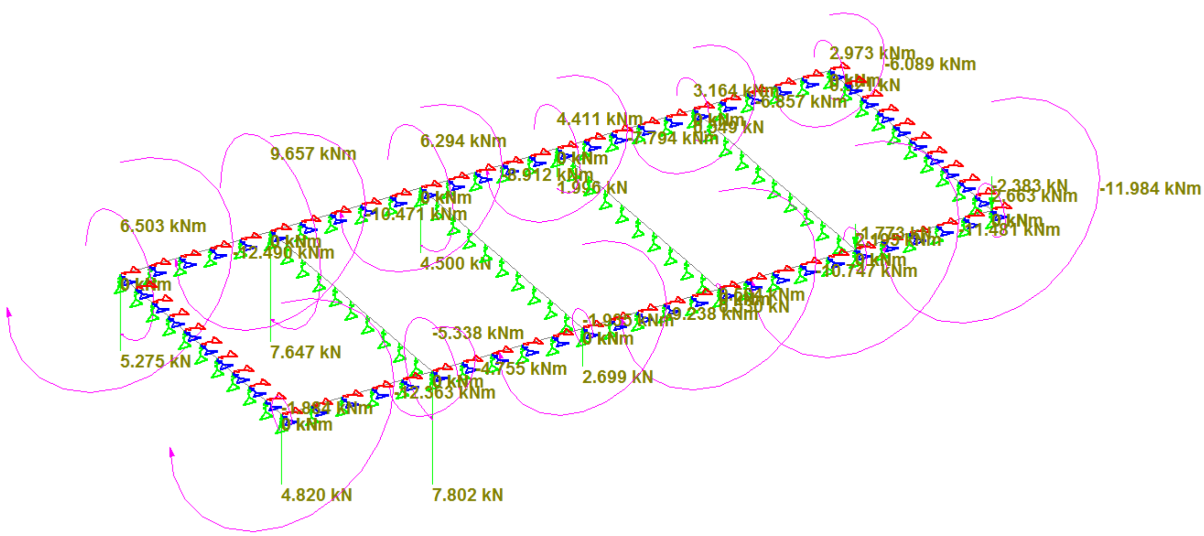

Wind - X (CPE + CPI):

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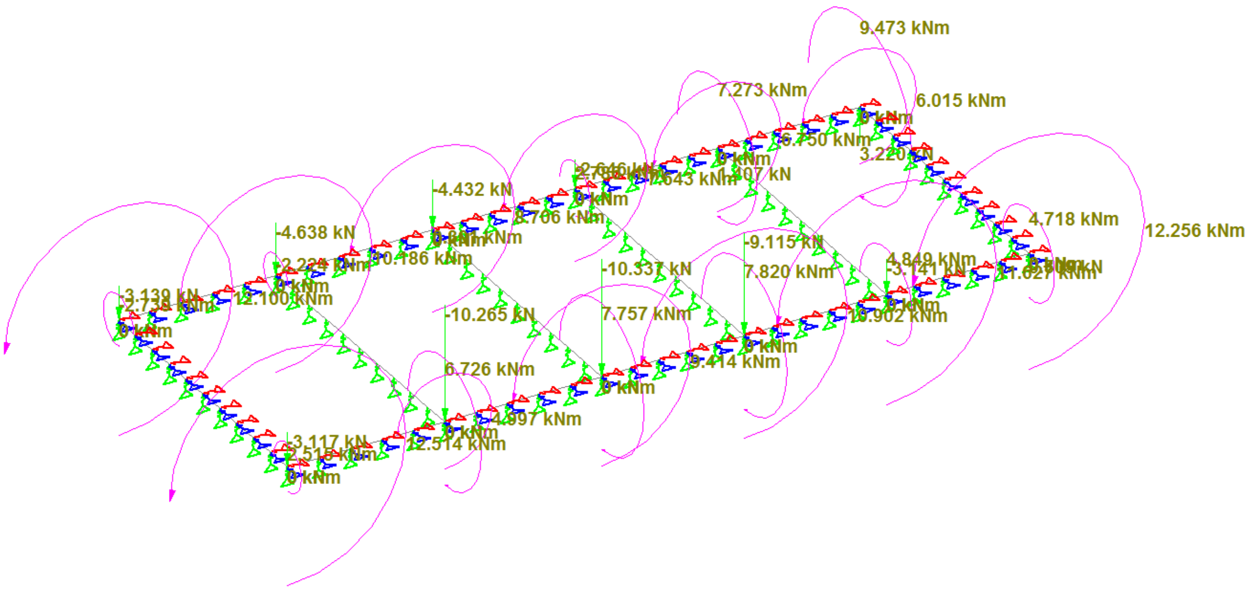

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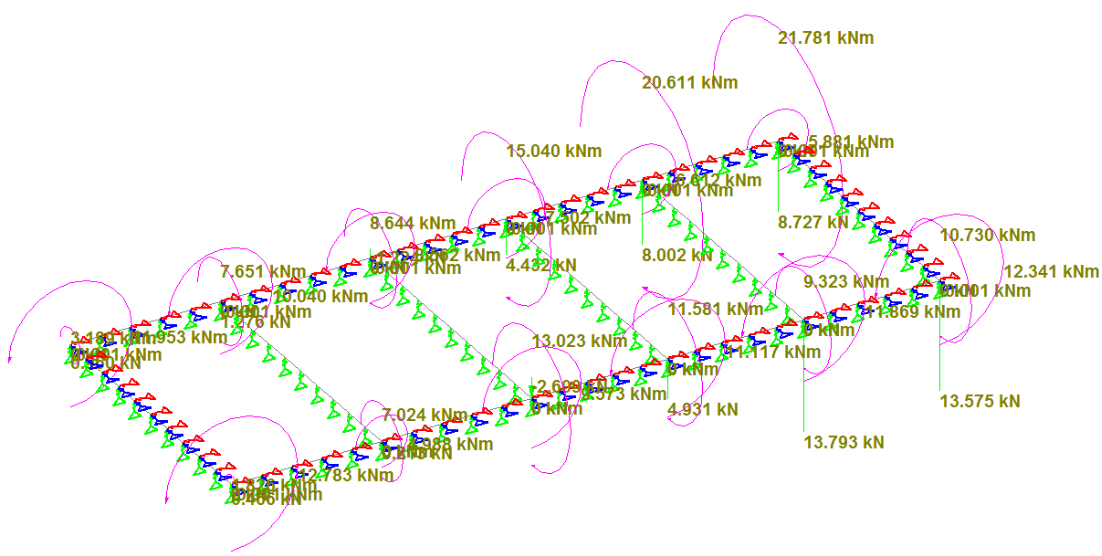

Wind + Z (CPE + CPI):

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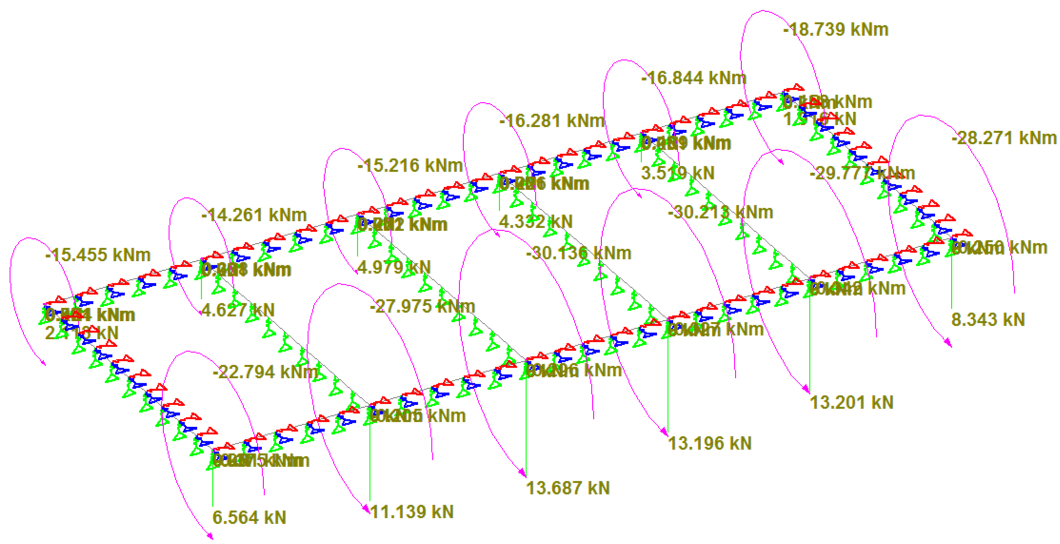
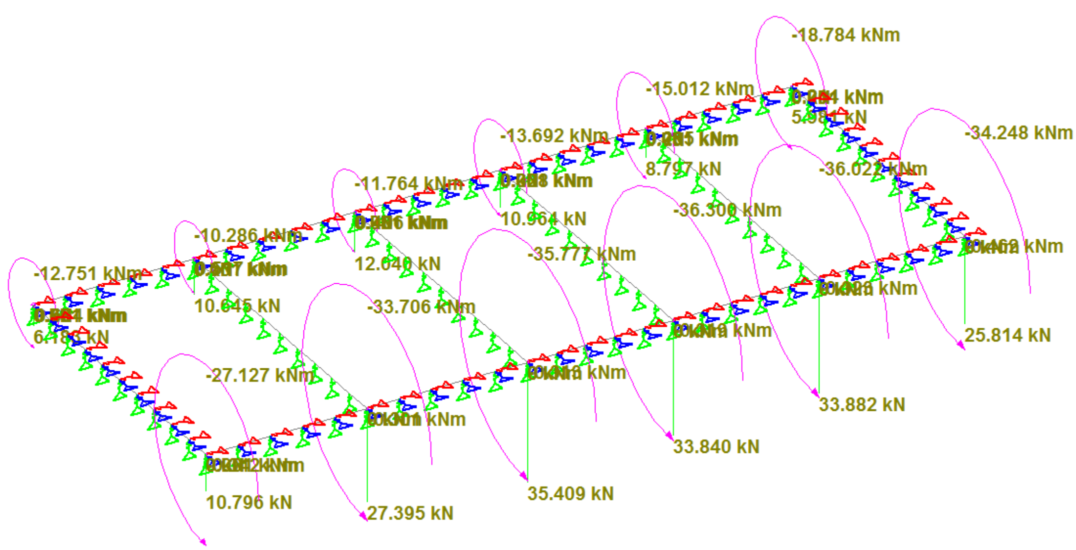
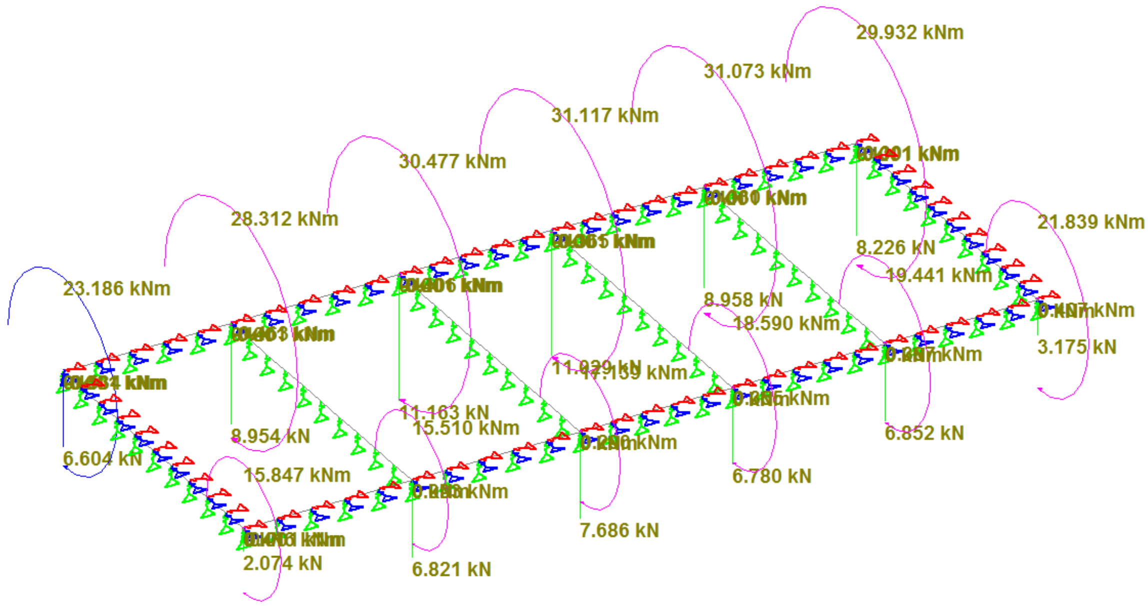


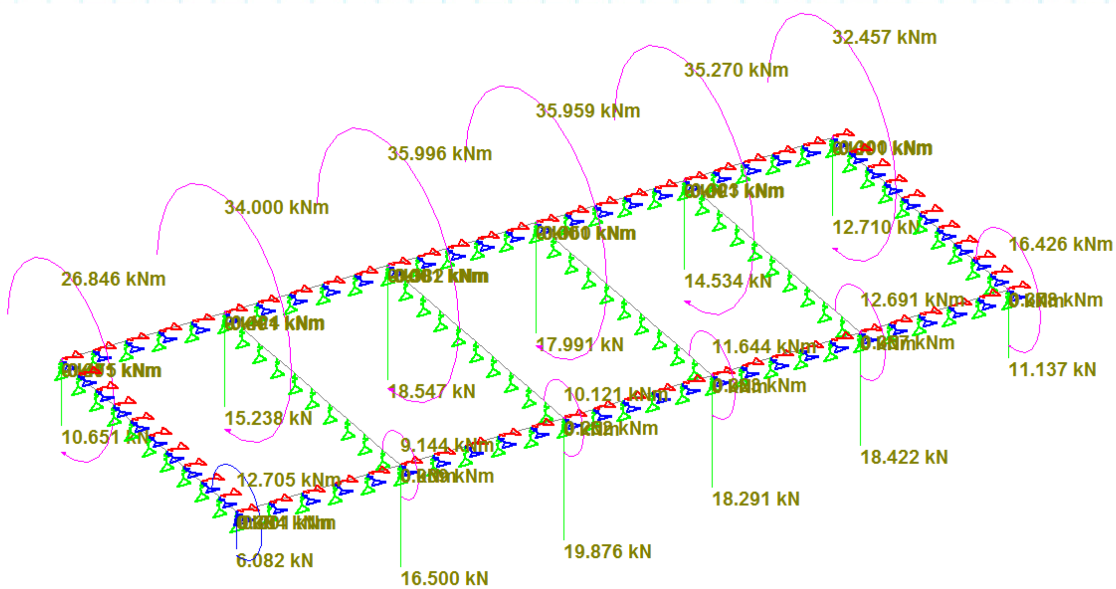
Wind + Z (CPE - CPI):



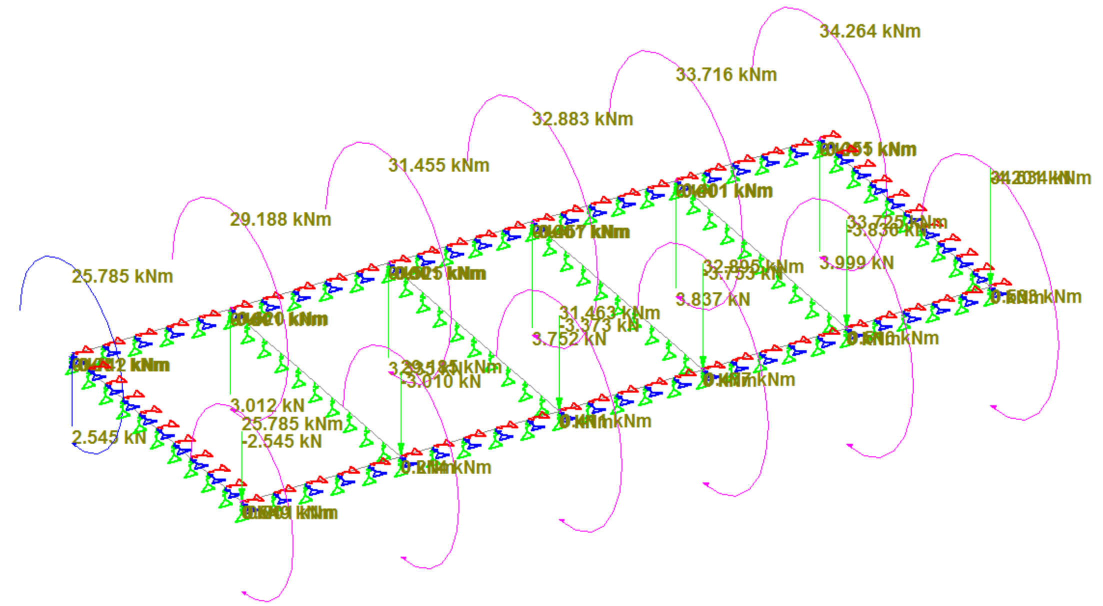
Wind - Z (CPE + CPI):



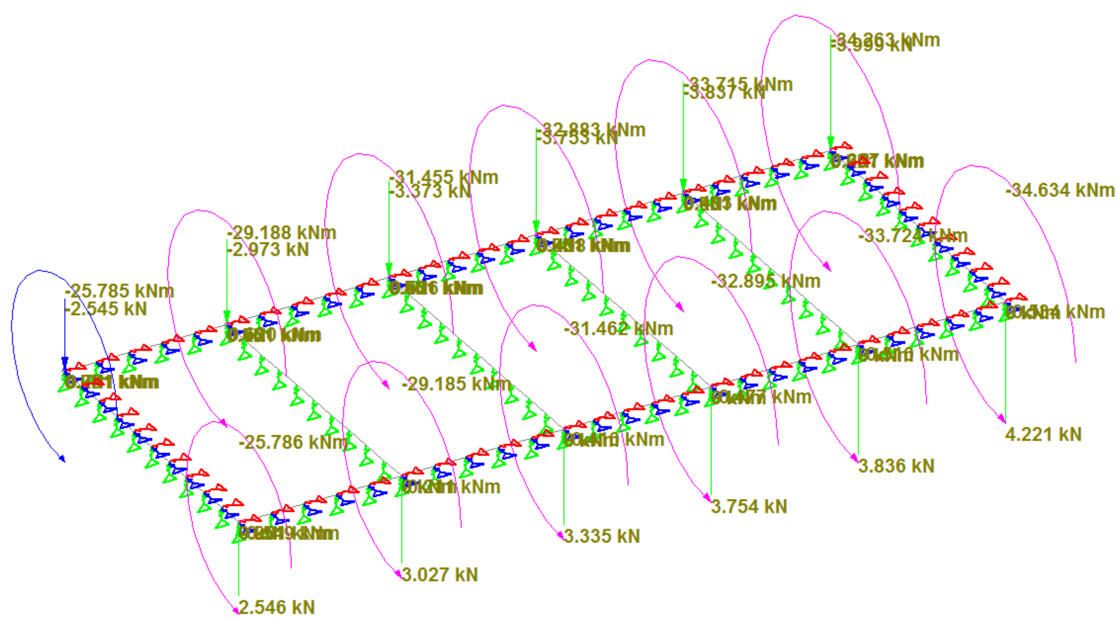
Wind - Z (CPE - CPI):



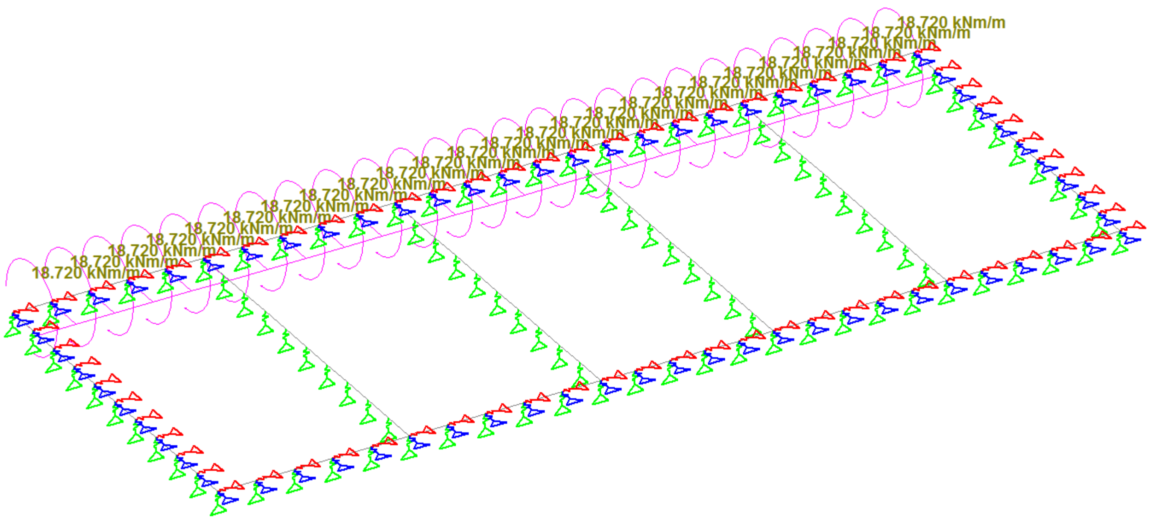
Earthquake +Z



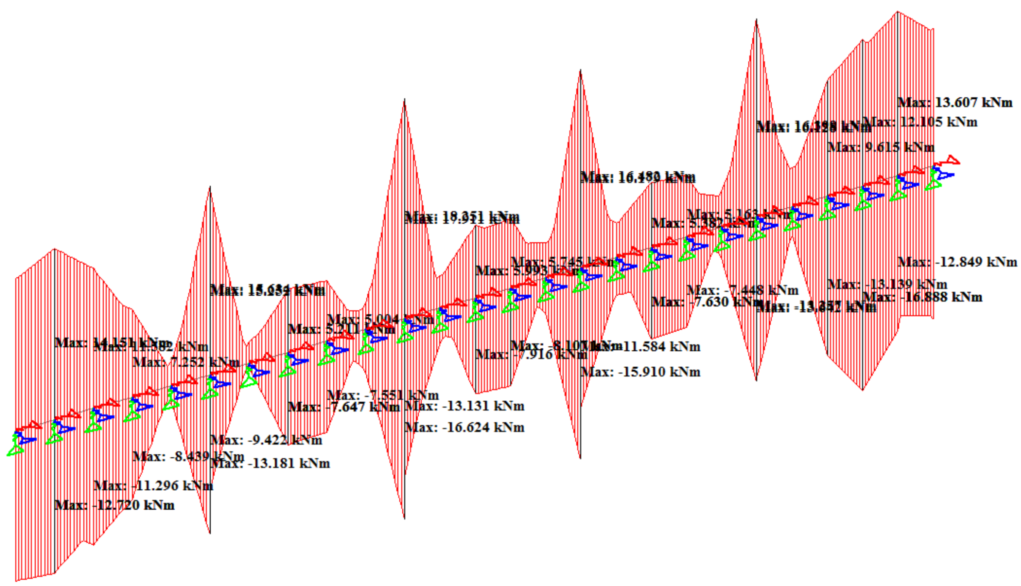
Earthquake - Z



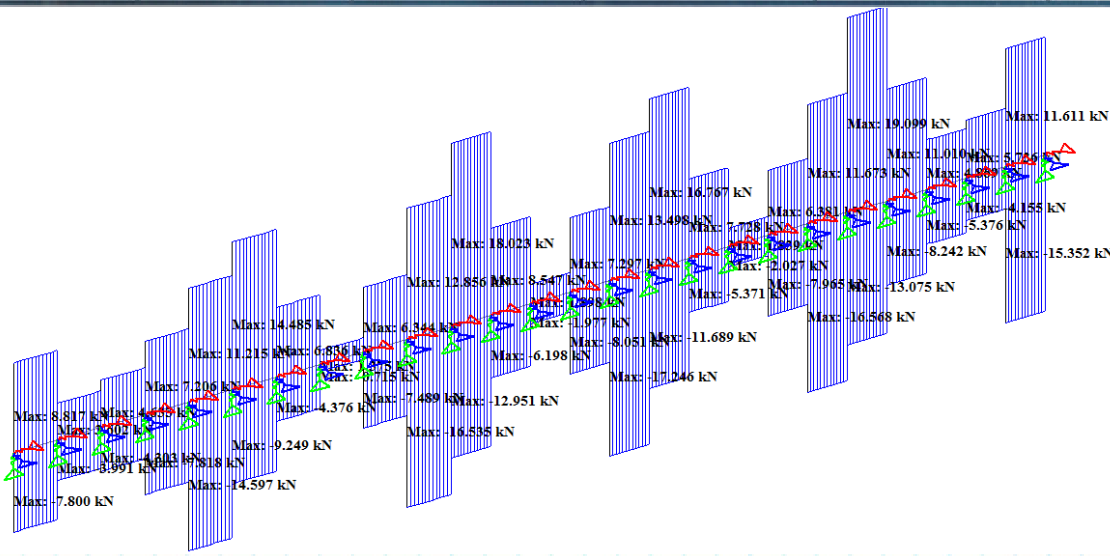
Fire case



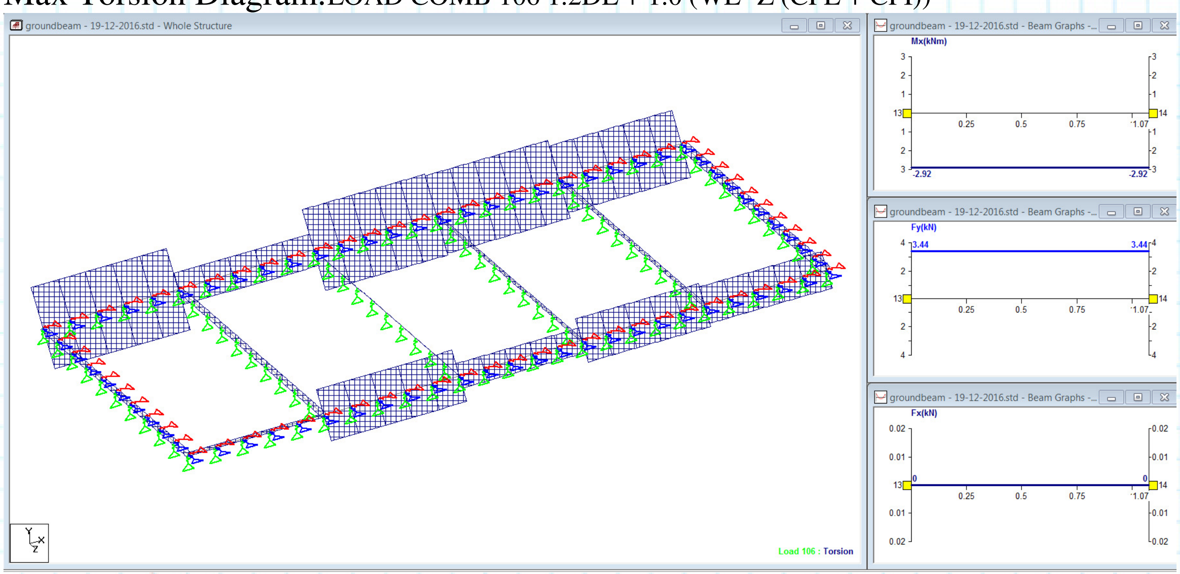
Max moment for envelope



Max Shear for envelope



Max Torsion Diagram:LOAD COMB 106 1.2DL + 1.0 (WL- Z (CPE + CPI))



Maximum Settlement:

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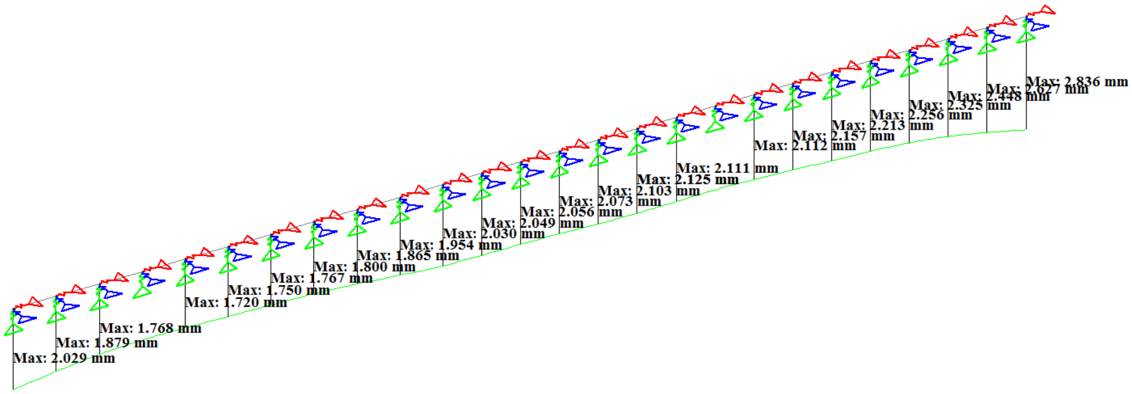
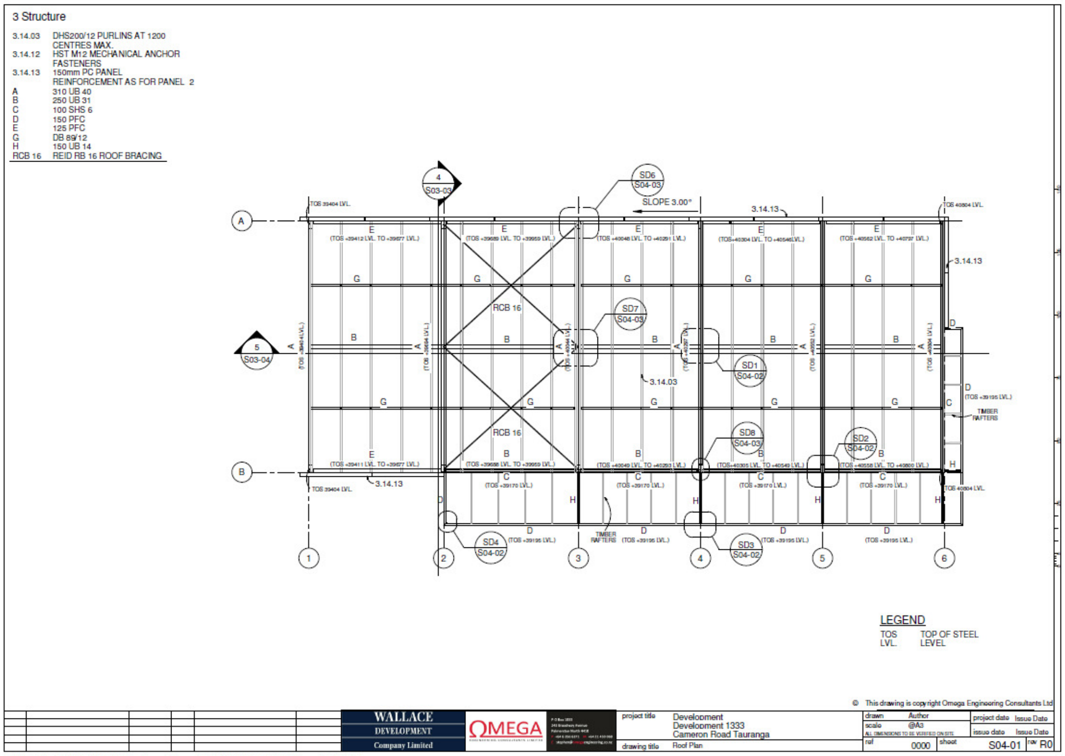
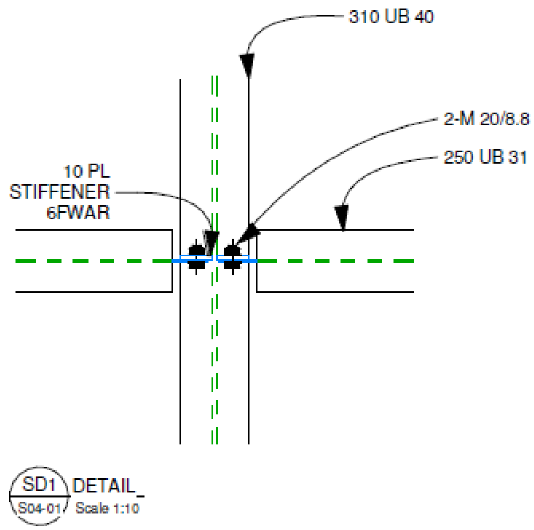


- **Connection design**

1) Typical Detail SD1/SD2: Single plate bolted-welded connection between web of UB 310X40.4 & web of UB 250X31.4



Location of Connection



3D Revised 6-12-2016 final_R1 -.std - Beam End Force

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
131	116	37	18.161	-6.271	1.211	0.000	0.000	0.000
131	108	37	17.877	-5.446	1.211	0.000	0.000	0.000
58	105	37	15.635	5.138	-0.999	0.000	0.000	0.000
58	113	37	15.591	4.286	-0.999	0.000	0.000	0.000
58	116	296	15.355	6.482	-1.226	0.000	-1.439	-7.613
58	108	296	15.311	5.629	-1.226	0.000	-1.439	-6.611
131	103	294	14.912	-2.198	0.382	0.000	-0.469	-2.694
131	111	294	14.628	-1.373	0.382	0.000	-0.469	-1.683
131	118	37	13.620	-6.271	-2.634	-0.000	0.000	0.000
131	110	37	13.336	-5.446	-2.634	-0.000	0.000	0.000
58	118	296	11.054	6.482	2.664	-0.000	3.128	-7.613
58	110	296	11.010	5.629	2.664	-0.000	3.128	-6.611
58	112	296	9.979	2.507	1.889	0.000	2.219	-2.945
58	104	296	9.936	1.655	1.889	0.000	2.219	-1.943
131	114	37	8.208	2.082	-2.149	0.000	0.000	0.000
131	106	37	7.924	2.907	-2.149	0.000	0.000	0.000
131	115	37	7.624	-2.033	2.963	0.000	0.000	0.000
131	117	37	7.554	-2.033	-0.987	-0.000	0.000	0.000
131	107	37	7.340	-1.208	2.963	0.000	0.000	0.000
131	109	37	7.270	-1.208	-0.987	-0.000	0.000	0.000
58	106	37	6.792	0.758	-2.663	0.000	0.000	0.000
58	114	37	6.748	-0.095	-2.663	0.000	0.000	0.000
58	117	296	6.544	2.102	0.999	-0.000	1.173	-2.468
58	115	296	6.540	2.102	-2.998	0.000	-3.521	-2.468
58	109	296	6.500	1.249	0.999	-0.000	1.173	-1.467
58	107	296	6.497	1.249	-2.998	0.000	-3.521	-1.467
131	104	294	4.342	2.040	2.029	0.000	-2.487	2.500
131	112	294	4.058	2.865	2.029	0.000	-2.487	3.511
131	102	294	2.330	-6.681	-0.000	-0.000	0.000	-8.188
131	105	294	1.819	-5.535	0.503	-0.000	-0.617	-6.783

Design Forces:

Governing L/C :- 0.9DL + 1.0 (WL- X (CPE - CPI))

Axial force= 18.16 kN

Shear force = 6.27 kN

<u>Bolted connection between 10 mm thick single plate & web of UB250X31.4:</u>

No. of bolts = n = 2

Bolt diameter = M20/8.8 Grade = 20mm

Capacity of single bolt in Shear = 129 kN (Thread excluded from shear plane)

Capacity of two bolts in Shear = 2 X 129 = 258 kN

Maximum shear in bolt= 18.16 /2= 9.07 kN (Hence, OK)

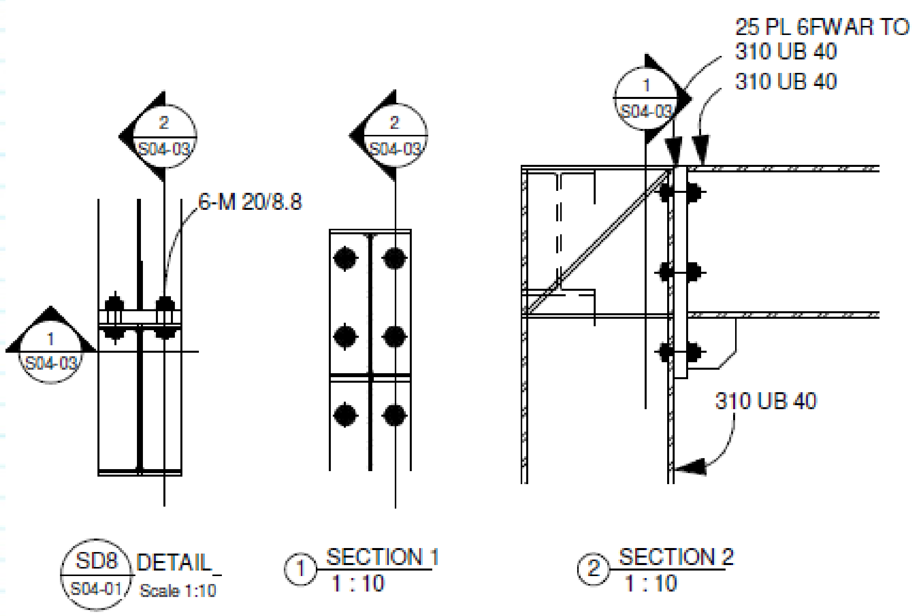
<u>Welded connection between 10mm thick stiffener plate & web of UB 310X40.4</u>

Web thickness of UB 310X40.4 = 6 mm

Weld size = 6mm (6FWAR)

2) **Typical Detail SD8/SD2: Moment end plate connection between beam UB 310X40.4 & flange of column UB310X40.4**

Design Sketch



3D Revised 6-12-2016 final_R1 -.std - Beam End Force

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
45	119	37	0.000	-6.657	-0.000	0.001	0.000	10.706
45	114	30	0.000	3.147	0.000	-0.020	-0.000	10.704
45	120	37	0.000	0.852	0.000	-0.001	0.000	9.737
45	117	30	0.000	0.237	0.000	-0.001	0.000	8.781
45	103	30	0.000	3.990	-0.000	0.019	0.000	8.748
45	106	37	0.000	-4.051	0.000	0.020	-0.000	8.180
45	103	37	0.000	-2.334	0.000	-0.019	-0.000	7.061
45	111	30	0.000	2.570	-0.000	0.019	0.000	6.088
45	114	37	0.000	-3.180	0.000	0.020	-0.000	5.113
45	111	37	0.000	-1.463	0.000	-0.019	-0.000	3.994
45	104	30	0.000	-2.252	0.000	0.019	-0.000	0.126
45	110	30	0.000	-5.537	0.000	-0.001	0.000	-1.688
45	112	30	0.000	-3.672	0.000	0.019	-0.000	-2.534
45	109	37	0.000	-1.261	-0.000	0.001	0.000	-4.148
45	118	30	0.000	-6.957	0.000	-0.001	0.000	-4.348
45	107	37	0.000	3.036	0.000	-0.000	-0.000	-4.848
45	117	37	0.000	-0.390	-0.000	0.001	0.000	-7.215
45	115	37	0.000	3.907	0.000	-0.000	-0.000	-7.914
45	104	37	0.000	1.088	-0.000	-0.019	0.000	-8.476
45	120	30	0.000	0.978	-0.000	0.001	0.000	-9.420
45	112	37	0.000	1.959	-0.000	-0.019	0.000	-11.543
45	107	30	0.000	-5.040	0.000	0.000	-0.000	-15.342
45	115	30	0.000	-6.460	0.000	0.000	-0.000	-18.002
45	110	37	0.000	3.113	0.000	0.001	0.000	-19.936
45	108	37	0.000	7.730	0.000	-0.000	-0.000	-20.530
45	118	37	0.000	3.984	0.000	0.001	0.000	-23.003
45	116	37	0.000	8.601	0.000	-0.000	-0.000	-23.597
45	108	30	0.000	-12.554	-0.000	0.000	-0.000	-30.181
45	116	30	0.000	-13.974	-0.000	0.000	-0.000	-32.840

Design Forces:

Governing L/C :- 0.9DL + 1.0 (WL- X (CPE - CPI))

Moment = 32.84 kNm

Shear force = 13.97kN

No. of bolts = n = 6

Bolt diameter = M20/8.8 Grade = 20mm

Tension in Bolts= M/D = 32.84 / 0.3 = 109.46kN

Capacity of two 20 mm bolt in tension = 163X 2 = 326 > 109.46 kN (Hence OK)

Shear on bolt = 13.97/6 = 2.32 kN

Shear capacity of 20mm bolt = 129 kN > 2.32kN (Hence OK)

<u>Plate thickness check:-</u>

Bending in plate = 109.46 X 0.05= 5.47 kNm

$S = 5.47 \times 10^6/250 = 21880\ mm^3$

Now, $S = bd^2/4$

So , d = SQRT (4 X S /b)

d = 23 mm

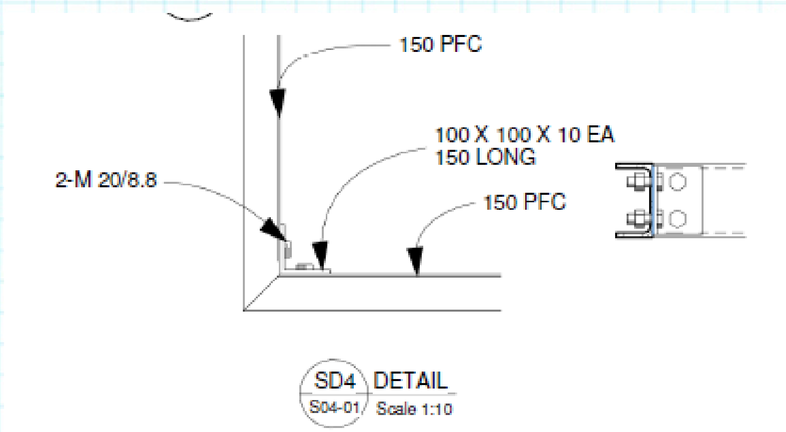
So provide plate thickness of 25mm

<u>Welded connection between UB 310X40.4 and 400X165X10 mm plate</u>

Web thickness of UB 310X40.4 = 6 mm

Weld size = 6 mm (6FWAR)

3) Typical Detail SD4: Connection between two PFC 150 with Angle 100X100X10



3D Revised 6-12-2016 final_R1 -.std - Beam End Force

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
203	107	119	0.012	1.637	0.009	-0.003	-0.011	0.000
203	102	119	-0.007	1.570	-0.005	-0.003	0.006	0.000
203	103	119	0.052	1.569	0.034	-0.002	-0.048	0.000
203	206	119	0.013	1.508	0.009	-0.002	-0.012	0.000
203	115	119	0.013	1.443	0.009	-0.002	-0.012	0.000
203	202	119	0.053	1.440	0.034	-0.002	-0.048	0.000
203	111	119	0.053	1.375	0.034	-0.002	-0.048	0.000
203	201	119	-0.005	1.176	-0.003	-0.002	0.004	0.000
203	105	119	-0.070	1.176	-0.046	-0.002	0.064	0.000
203	204	119	-0.070	1.046	-0.046	-0.002	0.063	0.000
203	113	119	-0.069	0.982	-0.046	-0.002	0.063	0.000
203	101	119	-0.004	0.873	-0.003	-0.002	0.003	0.000
203	109	119	-0.008	0.776	-0.006	-0.002	0.007	0.000
203	110	119	-0.001	0.776	-0.001	-0.001	0.001	0.000
203	119	119	-0.015	0.646	-0.011	-0.002	0.014	0.000
203	208	119	-0.007	0.646	-0.005	-0.001	0.006	0.000
203	209	119	-0.001	0.646	-0.001	-0.001	0.001	0.000
203	120	119	0.010	0.646	0.007	-0.001	-0.009	0.000
203	117	119	-0.007	0.582	-0.005	-0.001	0.006	0.000
203	118	119	-0.001	0.582	-0.001	-0.001	0.000	0.000
203	116	351	-0.016	0.478	-0.012	-0.001	0.001	-0.573
203	207	351	-0.016	0.413	-0.012	-0.001	0.001	-0.496
203	106	119	-0.086	0.380	-0.058	-0.001	0.079	0.000
203	108	351	-0.016	0.284	-0.011	-0.001	0.001	-0.340
203	205	119	-0.086	0.251	-0.058	-0.001	0.078	0.000
203	114	119	-0.085	0.186	-0.057	-0.001	0.078	0.000
203	104	119	0.050	0.177	0.032	-0.000	-0.046	0.000
203	203	119	0.051	0.047	0.033	-0.000	-0.046	0.000
203	112	351	-0.051	0.017	-0.033	0.000	0.007	-0.021
203	112	119	0.051	-0.017	0.033	-0.000	-0.047	0.000

Design force :-

Governing L/C :- 1.2DL + 1.0 (WL+ X (CPE - CPI))

Shear force = 1.63 kN

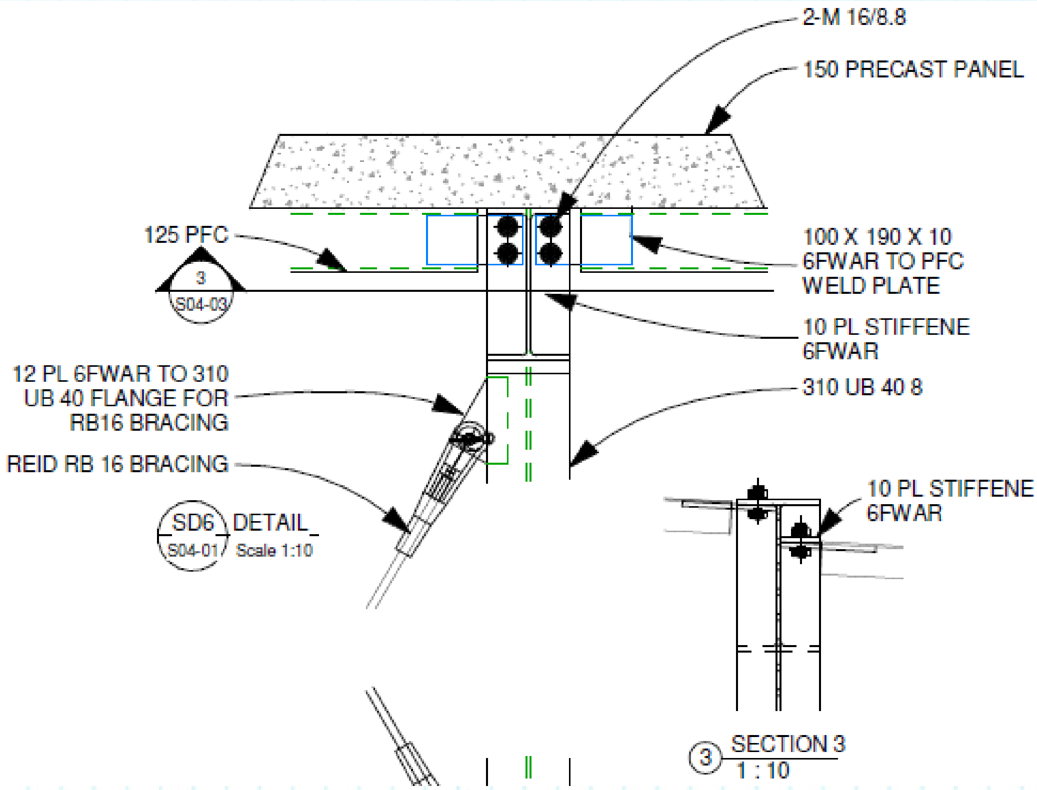
Axial force = 0.012 kN

No of Bolt = 2

Bolt diameter = M20/8.8 grade = 20mm

Shear force in bolt = 1.63/2 = 0.815kN
Shear capacity of bolt = 129> 0.815 kN (Hence OK)
Use connection angle as 100X100x10, 150 long with PFC 150 by 2 M20/8.8 grade bolt.

4) Typical Detail SD6: Plate 100X190X10 connection between PFC 125 and UB 310X40.4



Design Forces:

Shear force = 4.02kN

<u>Bolted connection between plate 100X190X10 and column stiffener plate :</u>

No. of bolts = n = 2

Bolt diameter = M16/8.8 Grade = 16mm

Tension force in single bolt = 4.02/2 = 2.01kN

Capacity of single bolt in tension =104 kN > 2.01 kN

Welded connection between plate 100X190X10 and web of PFC 125

Web thickness of PFC 125 = 6 mm

Weld size = 6mm (6FWAR)

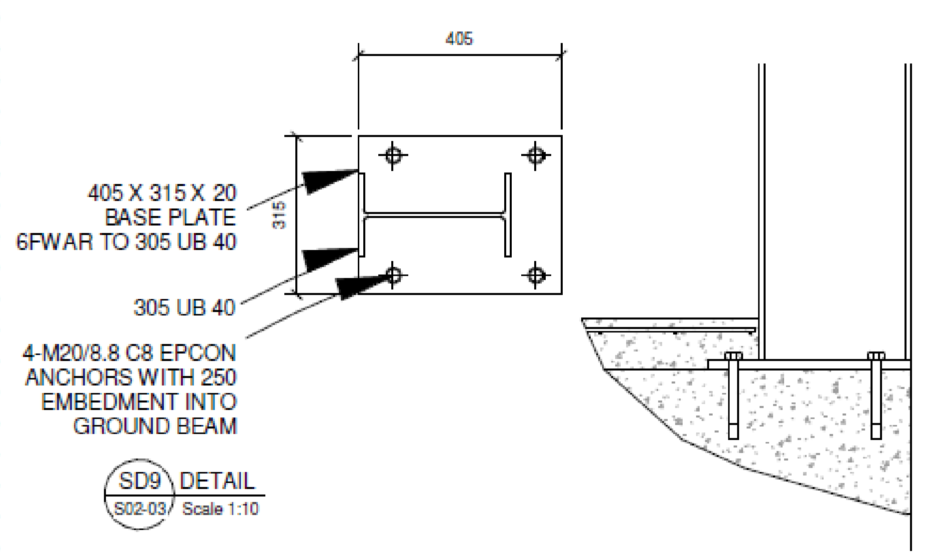
5) **Typical Detail SD9: Base Plate Design for Column UB 310X40.4**

3D Revised 6-12-2016 final_R1 -.std - Support Reactions:

All / Summary / Envelope

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	11	105 1.2DL + 1.0 (WL+ Z (CPE + CPI))	**5.598**	11.536	-2.508	-8.684	0.000	-12.320
Min Fx	11	111 0.9DL + 1.0 (WL+ X (CPE + CPI))	**-5.362**	11.267	-2.159	-5.635	-0.000	11.936
Max Fy	5	102 1.2DL + 1.5LL	0.008	**31.288**	-6.314	-9.219	0.000	-0.016
Min Fy	5	116 0.9DL + 1.0 (WL- X (CPE - CPI))	-0.079	**-22.189**	16.192	32.361	-0.000	0.307
Max Fz	3	116 0.9DL + 1.0 (WL- X (CPE - CPI))	-0.070	-18.674	**16.414**	30.193	-0.000	0.298
Min Fz	4	118 0.9DL + 1.0 (WL- Z (CPE - CPI))	-0.129	-11.675	**-17.179**	-31.091	0.001	0.464
Max Mx	10	120 1.0DL + 1.0 EARTHQUAKE - Z	0.084	7.416	13.916	**36.170**	-0.001	-0.372
Min Mx	11	119 1.0DL + 1.0 EARTHQUAKE + Z	0.289	14.072	-13.401	**-37.939**	0.000	-0.587
Max My	1	119 1.0DL + 1.0 EARTHQUAKE + Z	-0.241	3.456	-7.571	-21.754	**0.001**	0.794
Min My	1	120 1.0DL + 1.0 EARTHQUAKE - Z	0.209	8.546	15.769	29.817	**-0.001**	-0.689
Max Mz	1	104 1.2DL + 1.0 (WL- X (CPE + CPI))	-3.876	-2.414	-4.511	-7.111	0.001	**12.789**
Min Mz	2	106 1.2DL + 1.0 (WL- Z (CPE + CPI))	3.876	6.813	-5.747	-7.454	0.001	**-12.791**

Design sketch



Design Force:-

Axial force = 22.18 kN (uplift in column)

Shear (Fz) = 16.19 kN

Moment (Mx)= 32.36 kNm

No of Anchor bolt = 4

Anchor bolt diameter= 20 mm (M20 /8.8 Epicon anchor)

Tension on anchor bolt due to moment = M/D = 32.36/0.28= 115.57 kN

Total tension on one anchor bolt = (22.18/4) + (115.57/2) = 63.33 kN

Design tensile resistance of M20/8.8 EPICON C8 anchor with 250mm embedment = 76.2 kN > 63.33 kN (Hence OK)

Shear on one anchor bolt = 16.19 /4 = 4.04 kN

Shear capacity of one anchor bolt = 78.4 kN > 4.04 kN (Hence OK)

Base plate thickness check:-

Bending in plate = 115.57 X 0.05= 5.77 kNm

S= 5.77 X10^6/250 = 23080 mm^3

Now, S = bd^2 /4

So , d = SQRT (4 X S /b)

d = 17.54 mm

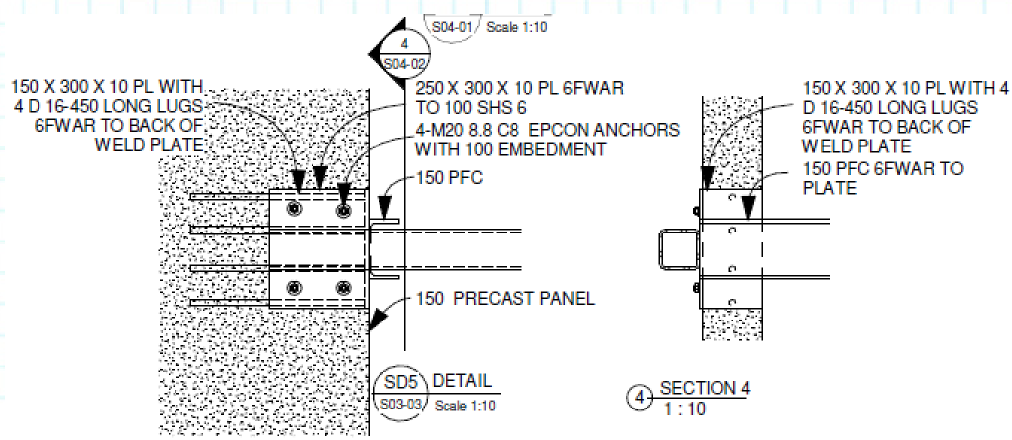
Provide thickness of base plate = 20mm

6) Typical Detail SD5: plate connection of SHS 100X100x6 with precast panel

3D Revised 6-12-2016 final_R1 -.std - Beam End Force

Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
287	107	127	0.000	0.670	-0.000	-0.034	0.000	-0.007
287	115	127	0.000	0.610	-0.000	-0.033	0.000	-0.007
287	102	127	0.000	0.482	0.000	-0.008	-0.000	-0.000
287	105	127	-0.000	0.442	0.000	0.581	-0.000	-0.005
287	120	127	-0.000	0.437	-0.000	-0.028	-0.000	-0.010
287	113	127	-0.000	0.383	0.000	0.582	-0.000	-0.005
287	101	127	0.000	0.268	0.000	-0.005	-0.000	-0.000
287	103	127	-0.000	0.211	0.000	-0.579	-0.000	0.007
287	104	127	-0.000	0.152	0.000	-0.571	-0.000	0.010
287	111	127	-0.000	0.152	0.000	-0.578	-0.000	0.007
287	114	172	0.000	0.120	0.000	-0.600	0.000	0.025
287	112	127	-0.000	0.093	0.000	-0.570	-0.000	0.010
287	108	127	0.000	0.086	-0.000	-0.021	0.000	-0.007
287	110	127	0.000	0.067	0.000	0.005	-0.000	0.007
287	106	172	0.000	0.060	0.000	-0.599	0.000	0.014
287	109	127	0.000	0.058	0.000	0.009	-0.000	0.007
287	119	172	0.000	0.041	-0.000	-0.021	0.000	-0.002
287	116	127	0.000	0.026	-0.000	-0.020	0.000	-0.007
287	118	127	0.000	0.008	0.000	0.006	-0.000	0.007
287	117	172	0.000	0.001	-0.000	-0.010	0.000	-0.007
287	117	127	0.000	-0.001	0.000	0.010	-0.000	0.007
287	118	172	0.000	-0.008	-0.000	-0.006	0.000	-0.009
287	116	172	-0.000	-0.026	0.000	0.020	-0.000	0.002
287	119	127	0.000	-0.041	0.000	0.021	-0.000	0.010
287	109	172	0.000	-0.058	-0.000	-0.009	0.000	-0.019
287	106	127	-0.000	-0.060	-0.000	0.599	-0.000	-0.002
287	110	172	0.000	-0.067	-0.000	-0.005	0.000	-0.020
287	108	172	-0.000	-0.086	0.000	0.021	-0.000	-0.010
287	112	172	0.000	-0.093	-0.000	0.570	0.000	-0.028
287	114	127	-0.000	-0.120	-0.000	0.600	-0.000	-0.002

Design Sketch



Design force = 0.67kN

No of Anchor bolt = 4

Anchor bolt diameter= 20 mm (M20 /8.8 Epcon anchor with 100mm embedment)

Shear on one anchor bolt = 0.67 /4 = 0.17 kN

Shear capacity of one anchor bolt = 78.4 kN > 0.17 kN (Hence OK)