طرح نگهداشت و افزایش تولید 27 مخزن

CALCULATION NOTE FOR CHEMICAL INJECTION AND STORAGE SHELTER

نگهداشت و افزایش تولید میدان نفتی بینک

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IDC: Inter-Discipline Check
IFC: Issued For Comment
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REVISION RECORD SHEET

PAGE	D00	D01	D02	D03	D04	PAGE	D00	D01	D02	D03	D04
1	X					66					
2	X					67					
3	X					68					
4	X					69					
5	X					70					
6	X					71					
7	X					72					
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53						118					
54						119					
55						120					
56						121					
57						122					
58						123					
59						124					
60						125					
61						126					
62						127					
63						128					
64						129					
65						130					

CONTENTS

1.0 INTRODUCTION

Binak oilfield in Bushehr province is a part of the southern oilfields of Iran, is located 20 km northwest of Genaveh city.

With the aim of increasing production of oil from Binak oilfield, an EPC/EPD Project has been defined by NIOC/NISOC and awarded to Petro Iran Development Company (PEDCO). Also PEDCO (as General Contractor) has assigned the EPC-packages of the Project to "Hirgan Energy - Design and Inspection" JV.

2.0 SCOPE

This report covers the structure & foundation calculation report of the “Chemical injection and Storage Shelter ”. The structure modelled by “SAP” software & the foundation modelled by “SAFE” software.

3.0 NORMATIVE REFERENCE

3.1 Local Codes and Standards

- INBC Part 6 “Iranian National Building Code
- INBC Part 7 “Iranian National Building Code
- INBC Part 9 “Iranian National Building Code
- INBC Part 10 “Iranian National Building Code
- Iranian Seismic Design Code for Petroleum Facilities(3rd edition)

3.2 International Codes and Standards

- ASCE 7-10 “Minimum Design Loads and Associated Criteria for Buildings and Other Structures-American Society of Civil Engineers”.
- ACI 318. “Building Code Requirements for Reinforced Concrete”, American Concrete Institute.
- AISC 358 “Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications.” American Institute of Steel Construction, Inc.
- AISC 360 - “Specification for Structural Steel Buildings”. American Institute of Steel Construction, Inc.

3.3 The Project Documents

- BK-GNRAL-PEDCO-000-ST-SP-0001 SPECIFICATION FOR CONCRETE WORK
- BK-GCS-PEDCO-120-ST-DW-0058 STRUCTURAL DRAWING FOR CHEMICAL INJECTION & STORAGE SHELTER

4.0 MATERIAL PROPERTIES

Material properties are delivered in the following table.

TABLE 1 -MATERIAL PROPERTIES

Foundation Concrete	F'c = 25 Mpa(28- day cylindrical sample)
Long. reinforcement bar	Fy = 400 Mpa(AIII)
Trans. reinforcement bar	Fy = 400 Mpa(AIII)
Bolt Type	HV 8.8
Electrode Type	E 70

5.0 STRUCTURE 'S SYSTEMS

The Structure's System is OMF in X direction and OCBF system in Y direction.Seismic Parameters according to Iranian seismic design code listed at below table.

TABLE 2 -MATERIAL PROPERTIES

	R	OMEGA	CD
X DIR	3.5	3	3
Y DIR	3.25	2	3.25

6.0 DESIGN LOAD

6.1 Dead load

Dead loads include the self-weight of the structure and all the permanent equipment which are supported by the structures

$\text{Corogated sheet} \quad : 8\ \text{kg/m}^2$

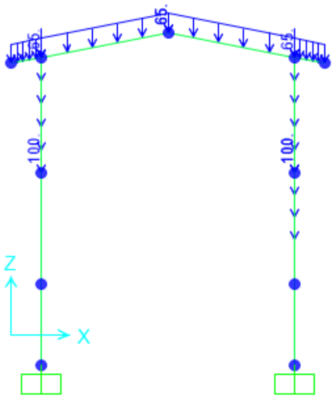
$\text{Z Purlin} \ : \ \ 8\ \text{kg/m}^2$

$\text{Insulation} \ \ : 10\ \text{kg/m}^2$

$\sum \text{sum} = \ \ 26\ \text{kg/m}^2$

Roof weight is assigned in software 26 kg/m2.

- At ended frame : $26 \times 2.5 = 65\ kg/m$
- At middle frame : $26 \times 5 = 130\ \ kg/m$



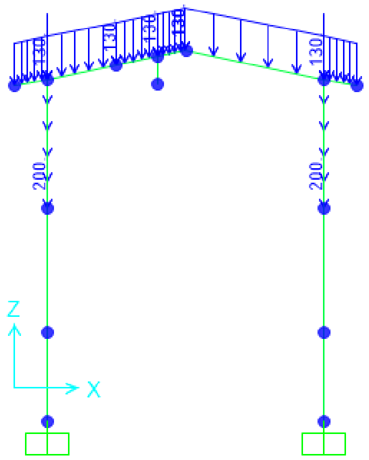


Figure 1-applied Dead load on ended axe(1&3) (kg/m) **Figure 2**-applied Dead load on middle axe 2(kg/m)

6.2 Live Loads

The design live load on an area shall be defined as the weight of all movable loads, including personnel, tools, and parts of dismantled equipment, cranes, hoist, and temporarily stored materials.

According to Iranian National Building code No.6 Live load in light slop roof is 50kg/m2 and assumed 1.3KN concentrated load has been applied at critical frame.

- At ended frame : $50 \times 2.5 = 125\ kg/m$
- At middle frame : $50 \times 5 = 250\ \ kg/m$

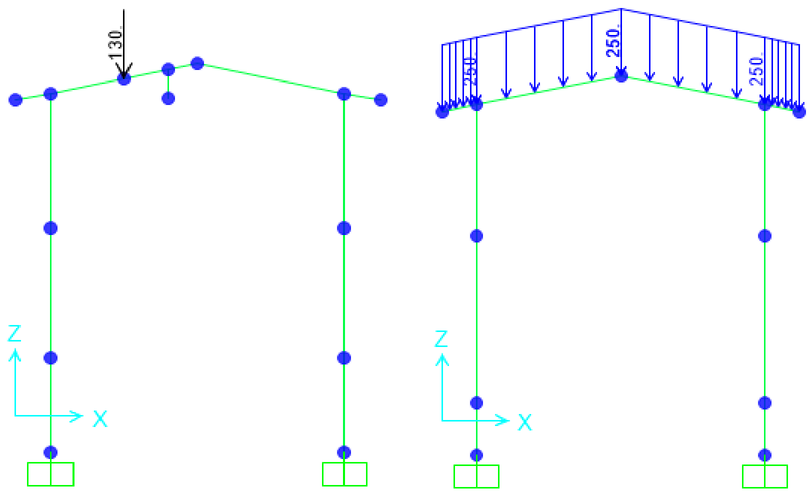


Figure 3-Applied live Load on frame 2 (kg/m)

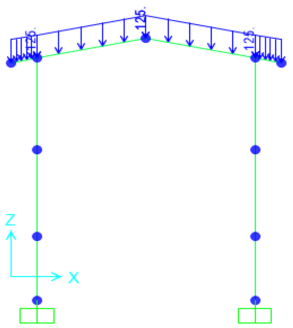


Figure 4-Applied live Load on frame 1&3 (kg/m)

6.3 SNOW LOADS

Snow load of this structure is calculated in accordance with Iranian National Building Code No.6 Latest edition.. Parameters which are used in calculation of snow force is presented in below:

$$P_r = P_s C_n C_h I_S C_s$$

Ps=25 kg/m2 , Is=1

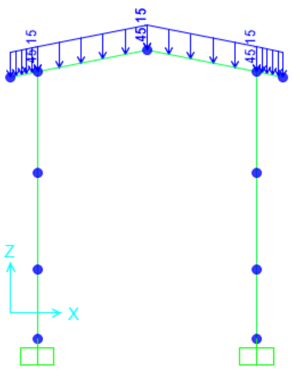
Cs= 0.91 (slope 11.31°)=1-$\frac{\alpha-\alpha_0}{70-\alpha_0} = 1 - \frac{11-5}{70-5} = 0.902$

Ch=1

Cn=0.8

$$P_r = P_s C_n C_h I_S C_s = 18..06\frac{kg}{m2}$$

- At ended frame : $18.06 \times 2.5 = 45.15\ kg/m$
- At middle frame : $18.06 \times 5 = 90.3\ \ kg/m$



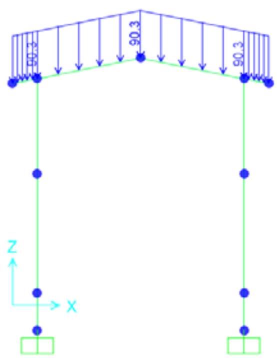
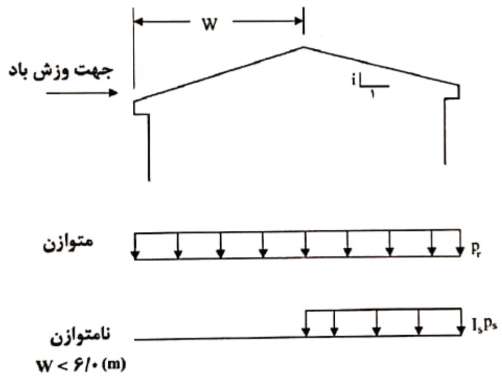


Figure 5-applied Snow load on ended axe(1&3) (kg/m) **Figure 6-**applied Snow load on middle axe 2(kg/m)

6.4 -UNBALANCED SNOW LOADS

According to Iranian National Building Code No.6 (latest edition)) Unbalanced snow load have been considered for roof slope between 4%~60%..in this structure Calculation of this load represents as below:



$$for\ lu < 6\ m \qquad P_r = I_s P_g = 25 \frac{kg}{m2}$$

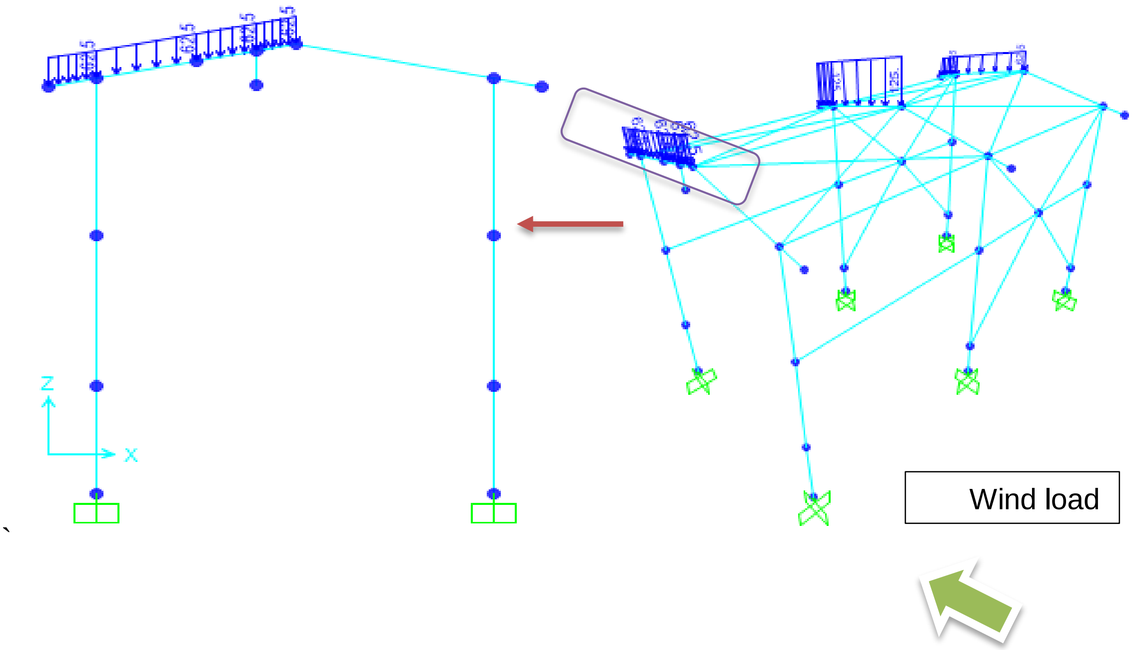


Figure 7-applied unbalancexd Snow Load(SN)

- At ended frame : $25 \times 2.5 = 62.5\ \ kg/m$

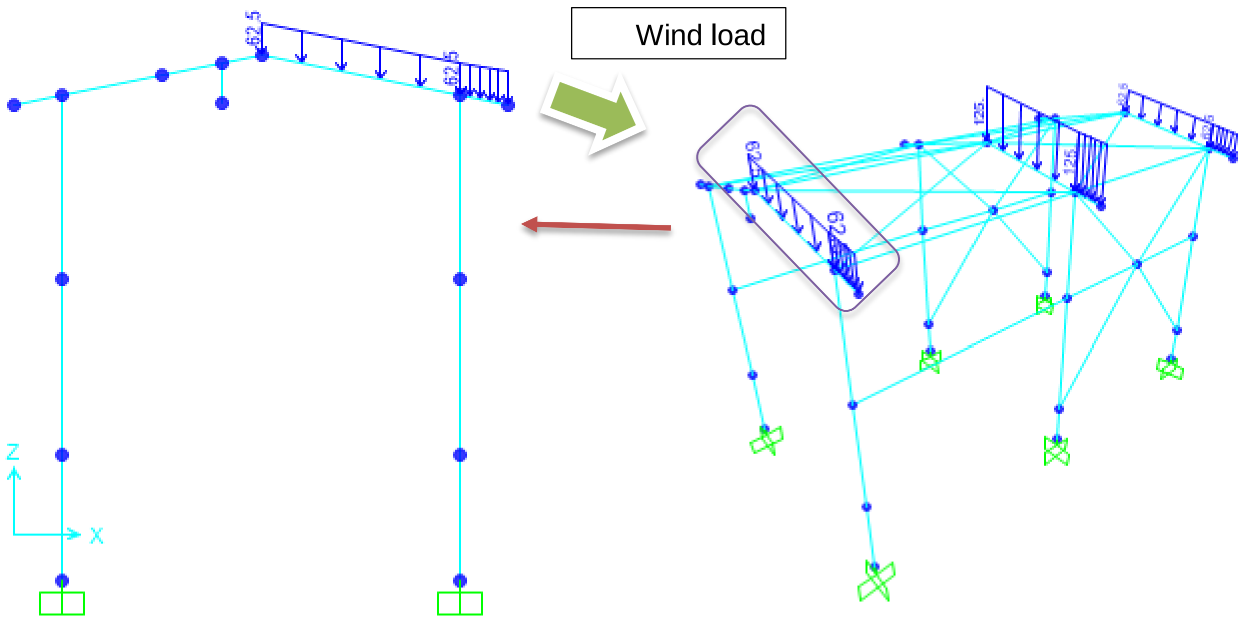


Figure 8-applied unbalancexd Snow Load(SP)

6.5 SEISMIC LOADS

All structures are in area with high risk zone of seismic and until finalizing of "Geotechnical Final Report" soil type consider is type III. Equivalent static method is used for calculation of seismic loads. Parameters which are used in calculation of earthquake force and seismic coefficient is presented in below.

Seismic loads are calculated according to Iranian Seismic Design code for petroleum facilities (3rd Edition)

For OMF system (X direction)

Rux=3.5
Omega=3
Cd=3

For OCBF system (Y direction)

Ruy=3.25
Omega=2
Cd=3.25
Soil Type : Type III

According to Iranian Seismic Design code for petroleum facilities (3rd Edition)

- **<u>For X direction</u>**

input						
A	Ru	I	Cd	B(0.2 SEC)	B(1 SEC)	T(etabs or sap)
0.3	3.5	1	3	2.75	1.9	0.3
output						
SDs	SD1	Sa	T0	Ts	Cu	
0.825	0.57	0.825	0.138181818	0.690909091	0.235714286	

A=0.3

$Ru = 3.5$(According to table 4-4 code 038 3r Edition)

$Cd = 3.0$(According to table 4-4 code 038 3r Edition)

$I = 1$

B(0.2s)=2.75(According to Soil type)

B(1s)=1.9(According to Soil type)

$T = 0.3\ according\ to\ sap\ software$

T0<T<Ts $Sa = A.B(0.2s) = 0.825$

$C_u x = \frac{S_a.I}{R_u}$=0.235

- **<u>For Y direction:</u>**

A	Ru	I	Cd	B(0.2 SEC)	B(1 SEC)	T(etabs or sap)
0.3	3.25	1	3.25	2.75	1.9	0.3
SDs	SD1	Sa	T0	Ts	Cu	
0.825	0.57	0.825	0.138181818	0.690909091	0.254	

$A = 0.3$ (In this area)

$Ru = 3.25$(According to table 4-4 code 038 3r Edition)

$Cd = 3.25$(According to table 4-4 code 038 3r Edition)

$I = 1$

B(0.2s)=2.75(According to Soil type)

B(1s)=1.9(According to Soil type)

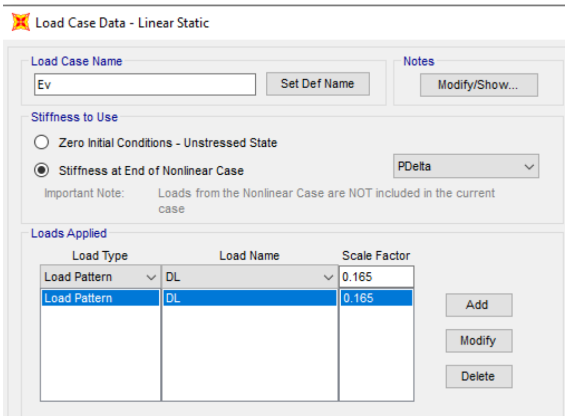
$T = 0.3\ according\ to\ sap\ model$

T0<T<Ts $Sa = A.B(0.2s) = 0.825$

$C_u y = \frac{S_a.I}{R_u}$=0.254

-Ev : Vertical seismic load applied at model according to section 2-2-3-2 (code 038)

$$E_v = \alpha S_{DS} = 0.2 * 0.825 = 0.165$$



Ev applied at model as a portion of dead load as above.

6.6 CRANE LOAD

Distribution of crane load is as below :

Capacity : 2000 kg

Monorail weight +Trolley weight : 2000 kg

Cis: crane vertical impact load

Cvs+Cis=KvCvs

Kvs =1.25 (according to INBC no.6)

Cvs=1.25x(2000+2000)= 5000 kg = 5 ton

Cls=0.1x5000 = 500 kg (in Y direction)

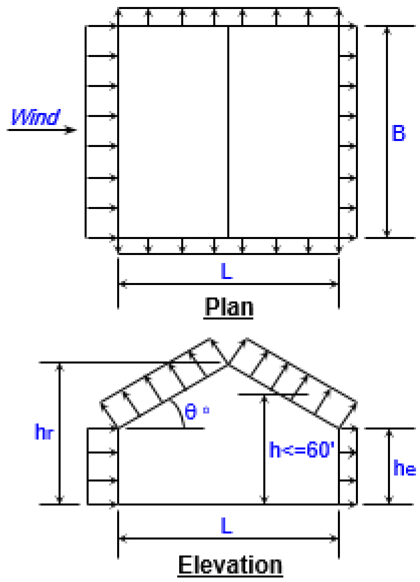
6.7 WIND LOADS

Wind loads are calculated according to ASCE07-10 and applied at model as below:

V=120 km/h(According to Iranian National Building Code No.6 last edition)

$$V_{ASCE} = 1.53 \times \sqrt{1.6Iw} \times V_{INBC} \qquad V\left(\frac{m}{s}\right)$$

$$V_{ASCE} = 1.53 \times \sqrt{1.6Iw} \times V_{INBC} = 1.53 \times \sqrt{1.6} \times 120 \times \frac{10}{36} = 64.5\ mph$$



Building classification =I building and other structures that represent a low risk to human life in the event of failure (Risk Category)

Exposure Category=C(open terrain with scattered obstructions having heights generally<30ft . this category includes flat open country and grass lands.

Ridge height =5.7 m

Eave height =5.2 m

Building width=5 m

Building length=10m

Roof type =Gable

Topo factor Kzt=1

Direct factor kd=1

Enclose =yes

Hurricane rfegion =no

Roof angle =11.31

Mean roof height =5.45 m

$q_{h=}0..00256 * Kz * Kzt * Kd * v^2 = 7.97$ psf

GC_{pi} $Coefficient\ (Positive\ Internal\ pressure) = 0.18$

GC_{pi} $Coefficient\ (Negative\ Internal\ pressure) = -0.18$

External Pressure Coeffients, GCpf (Fig. 28.4-1):	
For Load Case A:	
Wall Zone 1 =	0.45
Roof Zone 2 =	-0.69
Roof Zone 3 =	-0.42
Wall Zone 4 =	-0.35
Wall Zone 5 =	---
Wall Zone 6 =	---
Wall Zone 1E =	0.69
Roof Zone 2E =	-1.07
Roof Zone 3E =	-0.60
Wall Zone 4E =	-0.52

External Pressure Coeffients, GCpf (Fig. 28.4-1):	
For Load Case B:	
Wall Zone 1 =	0.40
Roof Zone 2 =	-0.69
Roof Zone 3 =	-0.37
Wall Zone 4 =	-0.29
Wall Zone 5 =	-0.45
Wall Zone 6 =	-0.45
Wall Zone 1E =	0.61
Roof Zone 2E =	-1.07
Roof Zone 3E =	-0.53
Wall Zone 4E =	-0.43
Roof Zone 5E =	0.61
Wall Zone 6E =	-0.43

Zone 1 is windward wall for interior zone.
Zone 2 is windward roof for interior zone.
Zone 3 is leeward roof for interior zone.
Zone 4 is leeward wall for interior zone.
Zones 5 and 6 are sidewalls.
Zone 1T is windward wall for torsional case
Zone 3T is leeward roof for torsional case

Zone 1E is windward wall for end zone.
Zone 2E is windward roof for end zone.
Zone 3E is leeward roof for end zone.
Zone 4E is leeward wall for end zone.
Zone 5E & 6E is sidewalls for end zone.
Zone 2T is windward roof for torsional case.
Zone 4T is leeward wall for torsional case.

According to ASCE 07-10 Design wind pressure for the building of all height shall be determined by the following equation : $P = qGCp - qi(GCpi)$

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.45	2.19	5.06	Zone 1	0.40	1.75	4.62
Zone 2	-0.69	-6.94	-4.07	Zone 2	-0.69	-6.94	-4.07
Zone 3	-0.42	-4.75	-1.88	Zone 3	-0.37	-4.39	-1.52
Zone 4	-0.35	-4.22	-1.35	Zone 4	-0.29	-3.75	-0.88
Zone 5	---	---	---	Zone 5	-0.45	-5.02	-2.15
Zone 6	---	---	---	Zone 6	-0.45	-5.02	-2.15
Zone 1E	0.69	4.07	6.94	Zone 1E	0.61	3.43	6.30
Zone 2E	-1.07	-9.97	-7.10	Zone 2E	-1.07	-9.97	-7.10
Zone 3E	-0.60	-6.20	-3.33	Zone 3E	-0.53	-5.66	-2.79
Zone 4E	-0.52	-5.57	-2.70	Zone 4E	-0.43	-4.86	-1.99
Zone 5E	---	---	---	Zone 5E	0.61	3.43	6.30
Zone 6E	---	---	---	Zone 6E	-0.43	-4.86	-1.99

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (kg/m2)		Surface	*GCpf	p = Net Pressures (kg/m2)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.45	10.69	24.70	Zone 1	0.40	8.56	22.57
Zone 2	-0.69	-33.85	-19.85	Zone 2	-0.69	-33.85	-19.85
Zone 3	-0.42	-23.20	-9.19	Zone 3	-0.37	-21.40	-7.39
Zone 4	-0.35	-20.58	-6.57	Zone 4	-0.29	-18.29	-4.28
Zone 5	---	---	---	Zone 5	-0.45	-24.51	-10.51
Zone 6	---	---	---	Zone 6	-0.45	-24.51	-10.51
Zone 1E	0.69	19.84	33.85	Zone 1E	0.61	16.73	30.74
Zone 2E	-1.07	-48.64	-34.63	Zone 2E	-1.07	-48.64	-34.63
Zone 3E	-0.60	-30.25	-16.24	Zone 3E	-0.53	-27.63	-13.62
Zone 4E	-0.52	-27.17	-13.17	Zone 4E	-0.43	-23.74	-9.73
Zone 5E	---	---	---	Zone 5E	0.61	16.73	30.74
Zone 6E	---	---	---	Zone 6E	-0.43	-23.74	-9.73

Figure 5-Wind Load calculation((kg/m2))

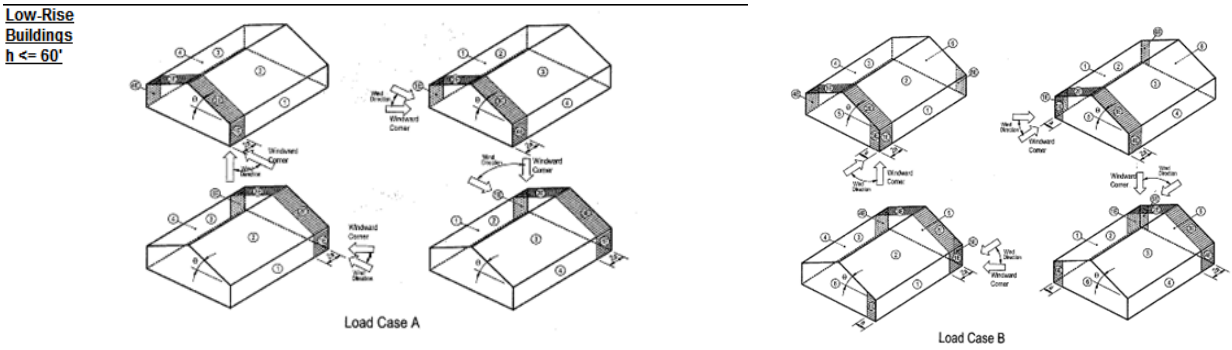


Figure 9-Wind Load Direction

-Wind Load Apply at Frame 2

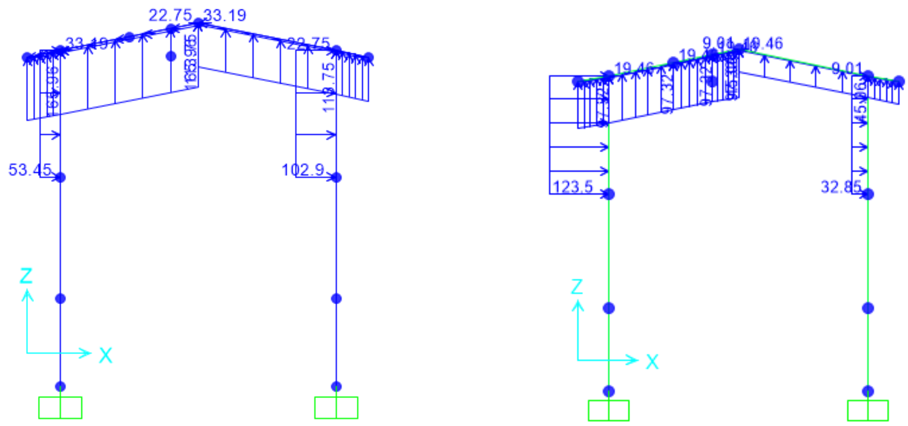


Figure 10-Wind Load apply(wx &wx1)

Load Case A

(Wx) Due to G+Cpi :

1: $Wx = 10.69 \times 5 = 53.45\ \frac{kg}{m}$

2: $Wx = -33.85 \times 5 = -169.25\ \frac{kg}{m}$

3: $Wx = -23.20 \times 5 = -116\ \frac{kg}{m}$

4: $Wx = -20.58 \times 5 = -102.9\ \frac{kg}{m}$

Wx1: Due to G-Cpi

1: $Wx = 24.70 \times 5 = 123.50\ \frac{kg}{m}$

2: $Wx = -19.85 \times 5 = -99.25\ \frac{kg}{m}$

3: $Wx = -9.19 \times 5 = -45.95\ \frac{kg}{m}$

4: $Wx = -6.57 \times 5 = -32.85\ \frac{kg}{m}$

Wind Load Apply at Frame 2 :

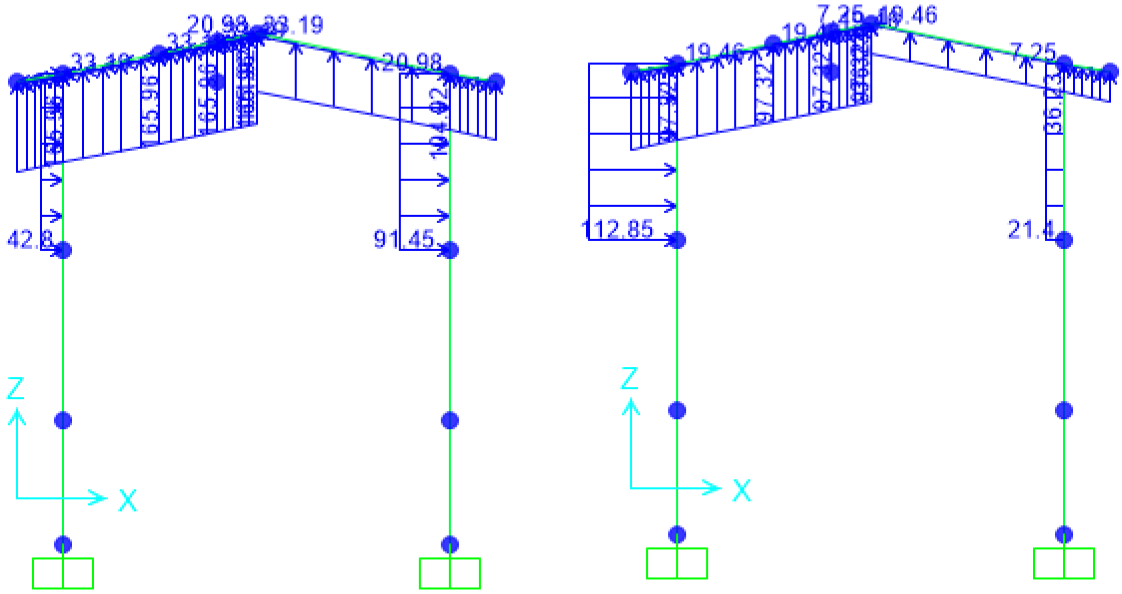


Figure 11-Wind Load apply(wy &wy1)

Load Case A

(WY) Due to G+Cpi :

1: $Wy = 8.56 \times 5 = 42.80\ \frac{kg}{m}$

2: $Wy = -33.85 \times 5 = -169.25\ \frac{kg}{m}$

3: $Wy = -21.40 \times 5 = -107\ \frac{kg}{m}$

4: $Wy = -18.29 \times 5 = -91.45\ \frac{kg}{m}$

5: $Wy = -24.51 \times 0.25 = -6\frac{kg}{m}$

6: $Wy = -24.51 \times 0.25 = -6\ \frac{kg}{m}$

WY1: Due to G-Cpi

1: $Wy1 = 22.75\ \times 5 = 112.85\ \frac{kg}{m}$

2: $Wy1 = -19.85 \times 5 = -99.25\ \frac{kg}{m}$

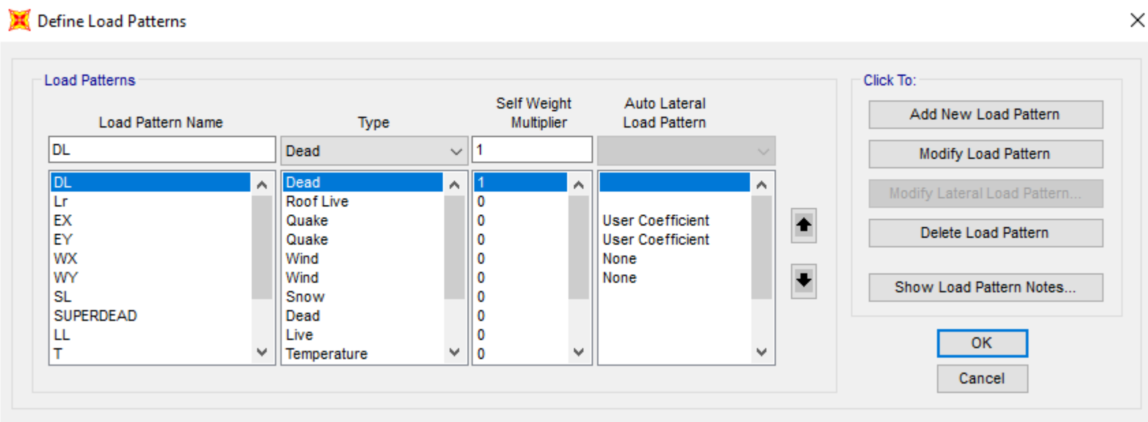
3: $Wy1 = -7.39 \times 5 = -36.95\ \frac{kg}{m}$

4: $Wy1 = -4.28 \times 5 = -21.40\ \frac{kg}{m}$

5: $Wy1 = -10.51 \times 0.25 = -2.62\frac{kg}{m}$

6: $Wy1 = -10.51 \times 0.25 = 2.62\ \frac{kg}{m}$

7.0 SAP loading table



8.0 Load combinations

According to code ASCE7 structures, components, and foundations shall be designed, so that their design strength equals or exceeds that effect of factored loads in the following combination:

- *1.4(D)*
- *(1.2D) +1.6(L)+0.5(Lr/S/R)*
- *1.2D+1.6(Lr/S/R) + (L/0.5W)*
- *1.2D+1.0(W) + L+.5(Lr/S)*
- *1.2D+1.0E+L+0.2S*
- *0.9D+1.0W*
- *0.9D+1.0E*

Load listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect considering soil reactions.

- *D*
- *D+L*
- *D+(Lr/S/R)*
- *D+0.75(L)+0.75(Lr/R/S)*
- *D+(0.6W or 0.7E)*
- *D+0.75L+0.75(0.6W)+0.75(Lr/S/R)*
- *D+0.75L+0.75(0.7E)+0.75S*
- *0.6D+0.6W*
- *0.6D+0.7E*

9.0 STRUCTURE ANALYSIS AND DESIGN

9.1 ANALYSIS

Structural analysis is done by SAP2000 software. In model loads are applied, some graphical outputs from model are shown as follows.

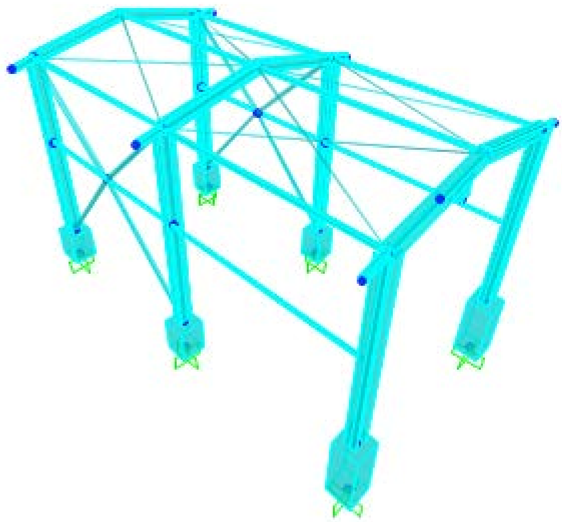


Figure 12-3D VIEW OF SAP MODEL

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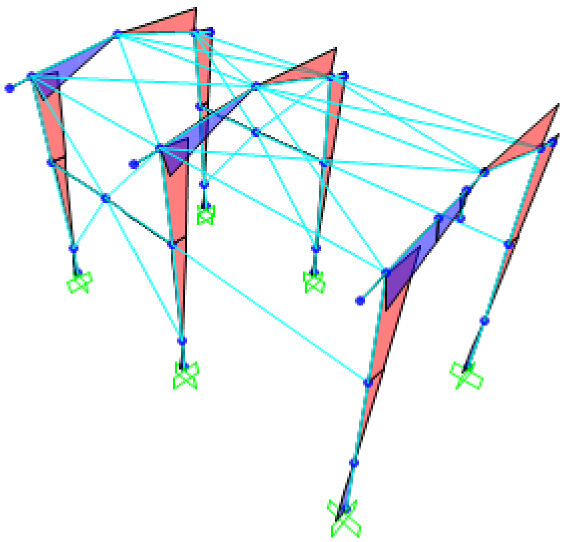


Figure 13: MOMENT 3-3 UNDER EX LOAD

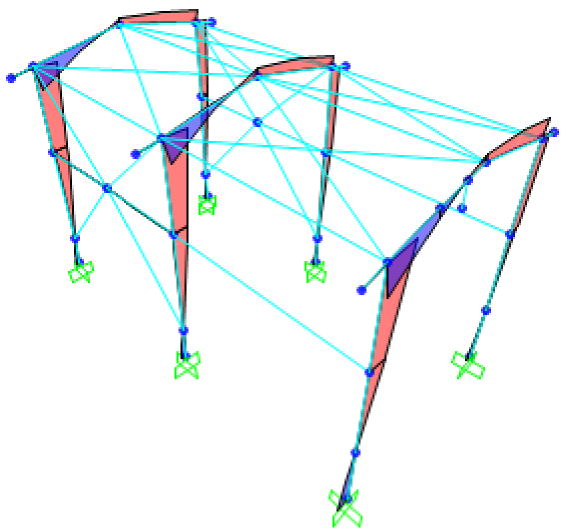


Figure 14: MOMENT 3-3 WX LOAD

9.2 FLEXTURAL DESIGN OF CRANE BEAM

According to below output from sap software maximum crane beam moment under critical load combination is 605863 kg.cm :

$$\emptyset Mn = 0.9xZxFy = 0.9x2400x561 = 1211760 \ kg.cm$$

$$M_u = 605863 \ kg.cm$$

$$s = \frac{6.058}{12.11} = 0.5 \ \ ok$$

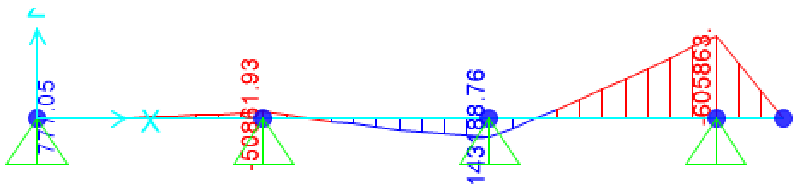


Figure 15: MOMENT 3-3 UNDER SERVICE LOAD COMBINATION ON CRANE BEAM

9.3 DEFLECTION CONTROL :

Maximum beam deflection under crane live load is :

$$\delta = \frac{PL^3}{48EI} = \frac{2000*500\text{^}3}{48*2.1*10^6*9723} = 0.25 \ \ \text{cm}< 0.625 \text{ ok}$$

$$\frac{L}{800} = \frac{500}{800} = 0.625\ cm$$

9.4 DRIFT CONTROL :

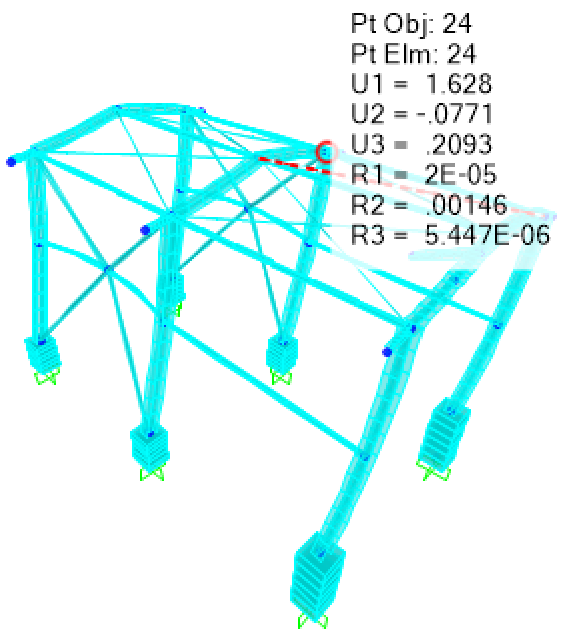


FIGURE 16: Moment 3-3 under service load combination on crane beam

Maximum displacement according to above output from sap model under critical service load combination is about 1.62 cm which is less than allowable drift.

$$allowable\ drift\ is\ \frac{h}{180} = 2.5cm$$

10.0 STRUCTURAL DESIGN RESULTS

10.1 Graphical output

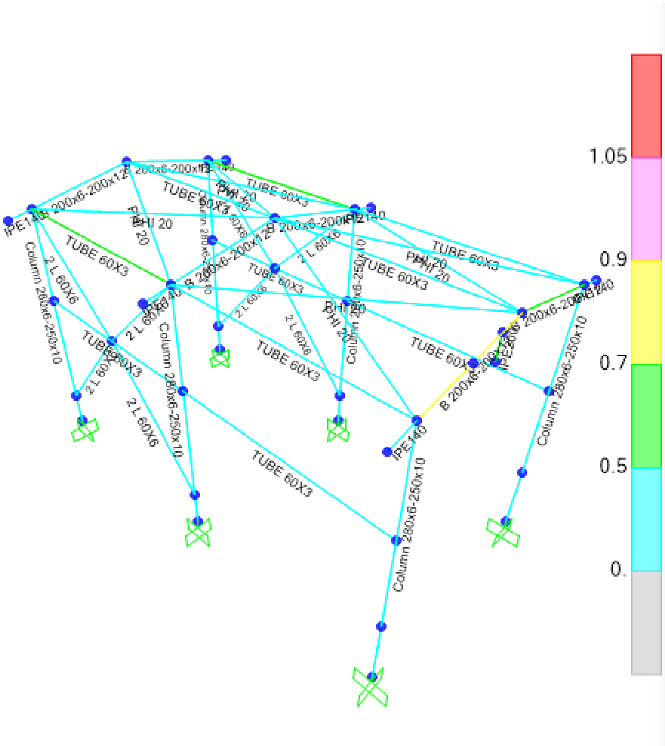
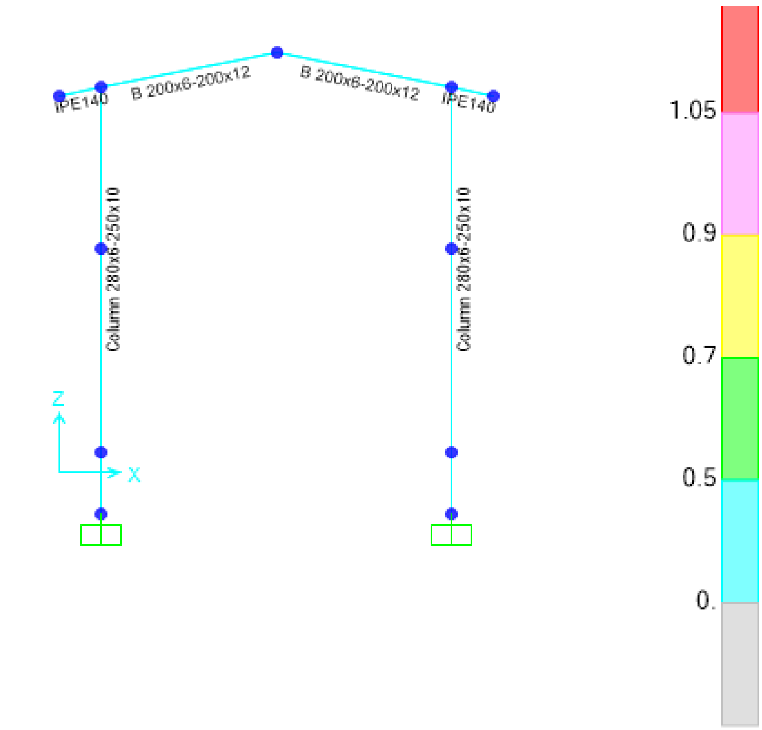
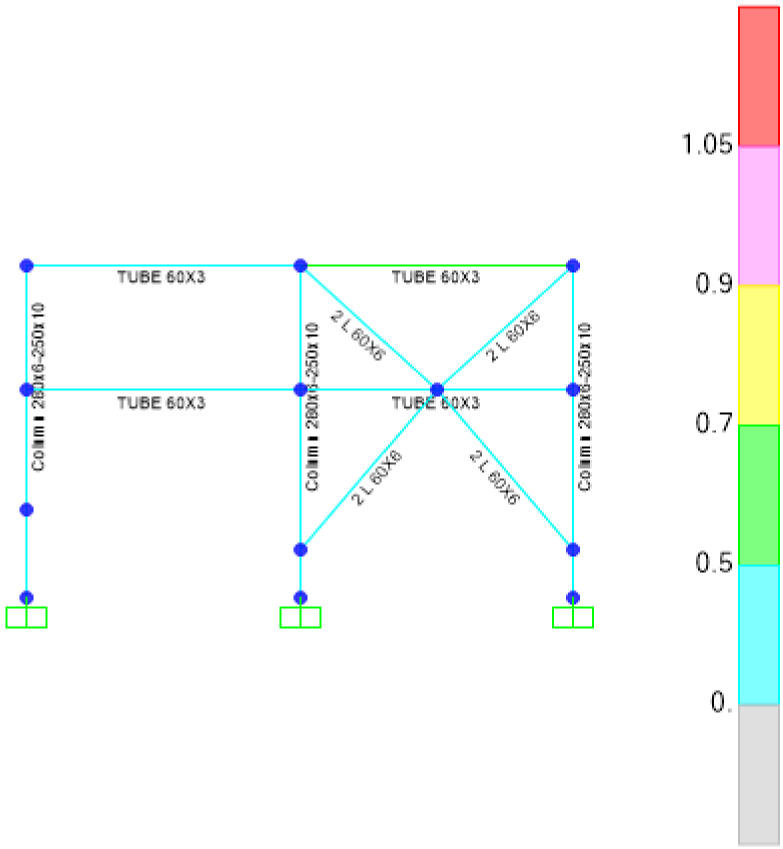


Figure 17: Steel Design Output



IPE140
B 200x6-200x12
B 200x6-200x12
IPE140
Column 280x6-250x10
Column 280x6-250x10
Z
X
1.05
0.9
0.7
0.5
0.



TUBE 60X3
TUBE 60X3
TUBE 60X3
TUBE 60X3
2 L 60X6
2 L 60X6
2 L 60X6
2 L 60X6
Column 280x6-250x10
Column 280x6-250x10
Column 280x6-250x10
1.05
0.9
0.7
0.5
0.

FIGURE 18: **Demand Capacity Ratio of Elements**

11.0 STRUCTURE CONNECTIONS

11.1 Special moment frame

-BEAM TO COLUMN

According to AISC 358-10 chapter 6, connection types 4ES and 8ES shall be used. Prequalified Special moment frame connections have been determined according to AISC 341- 10, Part 1, Section 10; AISC 358-10, Chapter 6 & AISC Design Guides 4, 13. Introductory figures & tables are attached. (Design procedure is mainly based on AISC 358 – 10 section 6.10)

High pretension bolts, used are equivalent to HV8.8 with ultimate strength of Fu=8,000 kg/cm2. The units used in the above mentioned codes & based calculation are kg-cm.

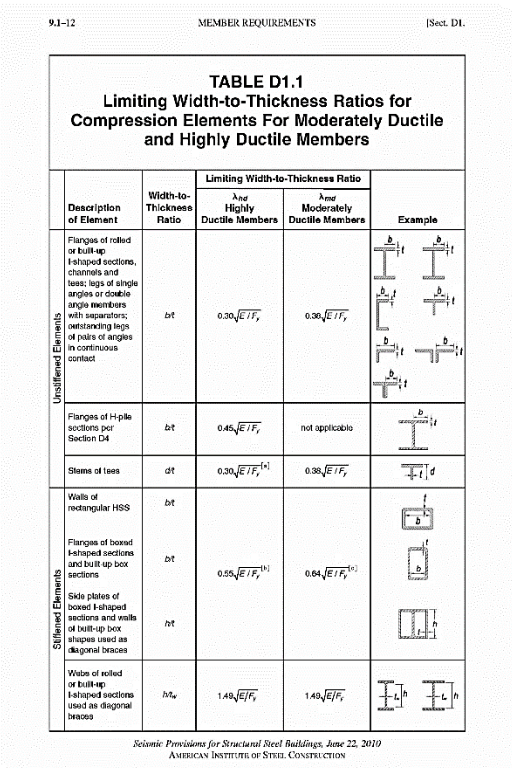

9.1–12 MEMBER REQUIREMENTS [Sect. D1.

TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements For Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$	
Unstiffened Elements	Flanges of H-pile sections per Section D4	b/t	$0.45\sqrt{E/F_y}$	not applicable	
Unstiffened Elements	Stems of tees	d/t	$0.30\sqrt{E/F_y}$ [a]	$0.38\sqrt{E/F_y}$	
Stiffened Elements	Walls of rectangular HSS	b/t	$0.55\sqrt{E/F_y}$ [b]	$0.64\sqrt{E/F_y}$ [c]	
Stiffened Elements	Flanges of boxed I-shaped sections and built up box sections	b/t			
Stiffened Elements	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t			
Stiffened Elements	Webs of rolled or built-up I-shaped sections used as diagonal braces	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$	

Seismic Provisions for Structural Steel Buildings, June 22, 2010
AMERICAN INSTITUTE OF STEEL CONSTRUCTION

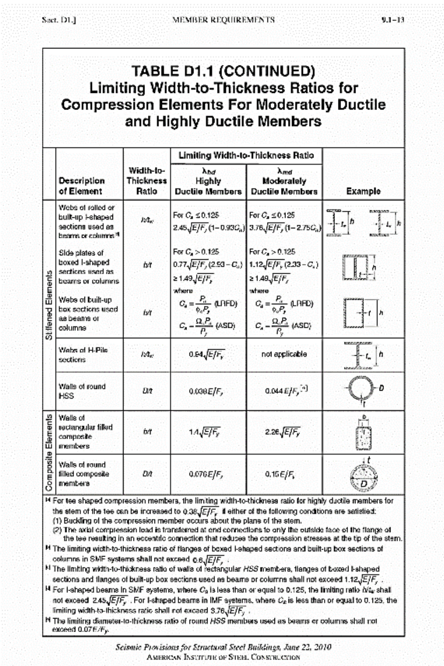

Sect. D1.] MEMBER REQUIREMENTS 9.1–13

TABLE D1.1 (CONTINUED)
Limiting Width-to-Thickness Ratios for Compression Elements For Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Stiffened Elements	Webs of rolled or built-up I-shaped sections used as beams or columns [d]	h/t_w	For $C_a \le 0.125$ $2.45\sqrt{E/F_y}(1-0.93C_a)$	For $C_a \le 0.125$ $3.76\sqrt{E/F_y}(1-2.75C_a)$	
Stiffened Elements	Side plates of boxed I-shaped sections used as beams or columns	h/t	For $C_a > 0.125$ $0.77\sqrt{E/F_y}(2.93-C_a) \ge 1.49\sqrt{E/F_y}$	For $C_a > 0.125$ $1.12\sqrt{E/F_y}(2.33-C_a) \ge 1.49\sqrt{E/F_y}$	
Stiffened Elements	Webs of built-up box sections used as beams or columns	h/t	where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	
Stiffened Elements	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$	not applicable	
Stiffened Elements	Walls of round HSS	D/t	$0.038E/F_y$	$0.044E/F_y$ [e]	
Composite Elements	Walls of rectangular filled composite members	b/t	$1.4\sqrt{E/F_y}$	$2.26\sqrt{E/F_y}$	
Composite Elements	Walls of round filled composite members	D/t	$0.076E/F_y$	$0.15E/F_y$	

[a] For tee shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee can be increased to $0.38\sqrt{E/F_y}$ if either of the following conditions are satisfied:
(1) Buckling of the compression member occurs about the plane of the stem.
(2) The axial compression load is transferred at end connections to only the outside face of the flange of the tee resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.
[b] The limiting width-to-thickness ratio of flanges of boxed I-shaped sections and built-up box sections of columns in SMF systems shall not exceed $0.6\sqrt{E/F_y}$.
[c] The limiting width-to-thickness ratio of walls of rectangular HSS members, flanges of boxed I-shaped sections and flanges of built-up box sections used as beams or columns shall not exceed $1.12\sqrt{E/F_y}$.
[d] For I-shaped beams in SMF systems, where C_a is less than or equal to 0.125, the limiting ratio h/t_w shall not exceed $2.45\sqrt{E/F_y}$. For I-shaped beams in IMF systems, where C_a is less than or equal to 0.125, the limiting width-to-thickness ratio shall not exceed $3.76\sqrt{E/F_y}$.
[e] The limiting diameter-to-thickness ratio of round HSS members used as beams or columns shall not exceed $0.07E/F_y$.

Seismic Provisions for Structural Steel Buildings, June 22, 2010
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$$M_f = M_{pr} + V_u S_h \qquad (6.10\text{-}1)$$

where

M_{pr} = probable maximum moment at plastic hinge, kip-in. (N-mm), given by Equation 2.4.3-1

S_h = distance from face of column to plastic hinge, in. (mm)

= the lesser of $d/2$ or $3b_{bf}$ for an unstiffened connection (4E)

= $L_{st} + t_p$ for a stiffened connection (4ES, 8ES)

V_u = shear force at end of beam, kips (N)

$$= \frac{2M_{pr}}{L_h} + V_{gravity} \qquad (6.10\text{-}2)$$

b_{bf} = width of beam flange, in. (mm)

d = depth of connecting beam, in. (mm)

L_h = distance between *plastic hinge locations*, in. (mm)

L_{st} = length of end-plate the stiffener, as shown in Figure 6.5, in. (mm)

t_p = thickness of end-plate, in. (mm)

$V_{gravity}$ = beam shear force resulting from $1.2D + f_1L + 0.2S$ (where f_1 is a load factor determined by the applicable building code for live loads, but not less than 0.5), kips (N)

$$d_{b\ \text{req'd}} = \sqrt{\frac{2\,M_f}{\pi\phi_n F_{nt}\left(h_1 + h_2 + h_3 + h_4\right)}} \qquad (6.10\text{-}4)$$

where

F_{nt} = nominal tensile strength of bolt from the AISC *Specification*, ksi (MPa)

h_i = distance from the centerline of the beam compression flange to the centerline of the ith tension bolt row.

h_o = distance from centerline of compression flange to the tension-side outer bolt row, in. (mm)

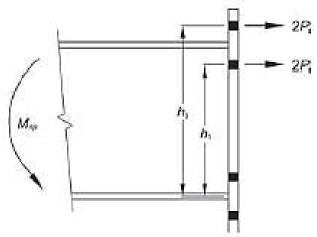


Figure 19: 4ES (FOUR EXTENDED STIFFENED)

11.2 DESCRIPTION OF DESIGN PROCEDURE & PARAMETERS

Compact sections shall be selected according to AISC 360-10.

bf, tf, hw and tw are sizes of girder.

Ry is the ratio of expected to specified structural steel yield strength.

$M_{pr} = C_{pr}.R_y.F_y.Z$ probable maximum moment at plastic hinge (according to AISC358- 10).

$S_h =$ distance from the face of column to plastic hinge $= L_{st} + t_p$ (according to AISC358- 10).

$L_{st} =$ Length of the end-plate stiffener.

$t_p =$ end-plate thickness.

$L_n =$ distance between plastic hinges.

$M_f = M_{pr} + V_p.L_n$ moment at the face of te column (according to AISC358- 10).

$V_p = \frac{2M_{pr}}{L_n} + \frac{wL_n}{2}$ Shear force at the end of the beam (according to AISC358- 10).

$M_{ff} =$ since amplified moment is allowed to be used for Special frame connections. It is extracted from the load combination using ΩM_E $(\Omega x = 3 , \Omega y = 2)$.

$F_u =$ bolt ultimate strength (HV8.8).

$F_{nt} =$ nominal tensile strength of bolt.

$\phi_n =$ non-ductile resistance factor (= 0.9 according to AISC358-10).

$h_i =$ distance from the centerline of the beam compression flange to the centerline of ith tension bolt row.

$(d_b) =$ required bolt dimension (according to AISC358-10)

$d_b =$ selected bolt diameter

phi Rn 4S = bolt shear rupture strength of the connection (according to AISC358-10)

F_b $4S$ bolt stress according to flexural < phi Fnt.

phi Rn yield = local column web yielding strength at beam flanges (according to AISC358-10).

phi Rn buck. = unstiffened column web buckling strength at beam compression flange (according to AISC358-10).

phi Rn cripp. = unstiffened column web crippling strength at beam compression flange (according to AISC358-10).

d_c = overall depth of column.

t_{wc} = column web thickness including doubler plates with plug weld.

Astiff = calculated required continuity plates area.

t_p = thickness of end-plate calculated according to plate theories formulas regarding plate boundary conditions. (it submits exactly the same amount using AISC 358 formulas).

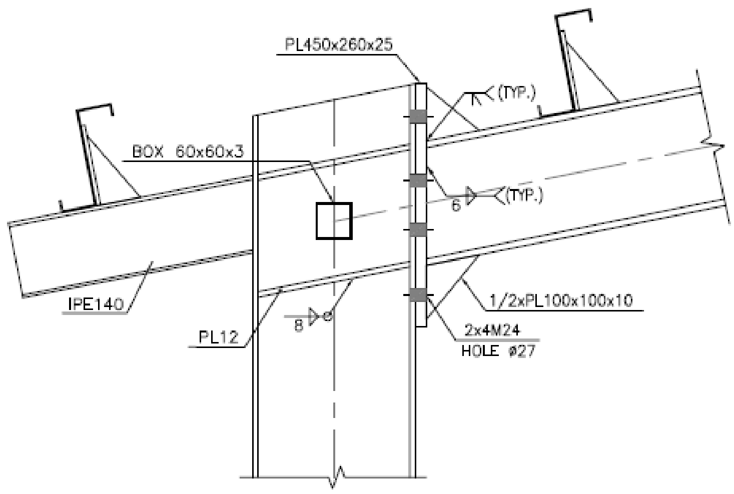


FIGURE 20: BEAM TO COLUMN CONNECTION

11.3 Connection Force Calculation :

$$S_h = \text{MIN}\left(\frac{d}{2}, 3b_{bf}\right) = 11.2\ cm$$

$$L_h = L - 2Sh = 4.776m$$

$$Wu = 1.2D + L + 0.2s = 0.424\ t/m$$

11.4 Beam Specification:

$$b_f(cm) = 20\ \ , t_f = 1.2cm\ \ , h_w(cm) = 20\ , t_w(cm) = 0.6$$

$$Z = 579.33cm3$$

$$M_p = Z * Fy = 1918734.33kg.cm$$

$$1.1 \le C_{pr} = \frac{F_y + F_u}{2Fy} = 1.27 \le 1.2 \quad C_{pr} = 1.2$$

$$M_{pr} = C_{pr}R_yM_p = 1.2 \times 1.2 \times 1918734.33 = 2020065.361\ kg.cm = 20.20\ t.m$$

$$R_y = 1.2$$

$$V^H{}_{gravity} = \frac{W_u L_h}{2} = 9.09\ ton$$
$$V_{pr} = \frac{2M_{pr}}{L_h} = 9.04\ ton$$
$$M_u = M_{pr} + \frac{W_u S h^2}{2} = 2.02 \times 10^6\ kg.cm = 20.2\ ton.m$$

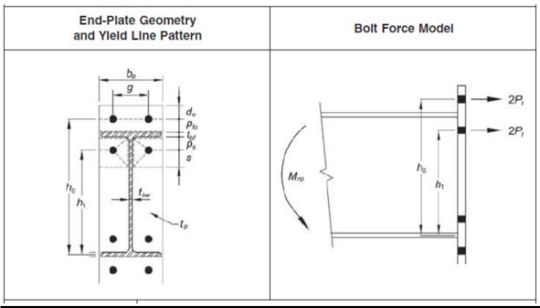
11.5 End plate thickness calculation :

Mu=20.2 ton.m
$\emptyset_d = 1.0$
$F_{yp} = 2400\ kg/cm2$
$s = \frac{1}{2}\sqrt{b_p g}$ =14 cm
$Y_p = \frac{b_p}{2}[h_1\left(\frac{1}{P_{fi}} + \frac{1}{S}\right) + h0\left(\frac{1}{P_{fo}}\right) - \frac{1}{2}] + \frac{2}{g}[h1(P_{fi} + S)] = 141.71$

$t_{p\ req} = \sqrt{\frac{1.11 M_u}{\varphi_d F_{yp} Y_p}} = 2.5\ cm$ We Used Plate 450 x 270 x 25

11.6 End Plate Specification :

bp(cm)	27
tp(cm)	2.5
g(cm)	14
s	9.72
pfi	6
pfo	6



11.7 Determine Multiplier force of the flange :

$$F_{fu} = \frac{M_u}{d_b - t_{bf}} = 95298.65\ kg$$

11.8 Shear yield control of the end plate :

$$R_n = 0.6 F_{yp} b_p t_p = 97200\ kg = 97.20\ ton$$

$$\frac{F_{fu}}{2} \leq \emptyset_d R_n \qquad \frac{95298.65}{2} = 47649.32 \leq 1 \times 97200 \quad ok$$

11.9 Shear rupture control of end plate

$$An = tp(bp - 2dcal) = 43.72cm2$$
$$dcal = 2.6cm$$
$$R_n = 0.6F_{up}A_n = 97051.9\ kg = 97.051ton$$
$$\frac{F_{fu}}{2} \leq \emptyset_n R_n \quad 47649 < 0.9 \times 97.05 = 87.345\ \ ok$$

11.10 Control of shear rupture resistance of screws:

$$V_u = 9.094\ ton$$
$$\emptyset_n = 0.9$$
$$n_b = 4$$
$$F_u = 8000\ kg/cm2$$
$$F_{nv} = 0.55Fu = 4400\frac{kg}{cm2}$$
$$d_b = 2.4cm$$
$$A_b = \frac{(\pi d_b)^{\wedge}2}{4} = 4.52\ cm2$$
$$R_n(kg) = n_b F_{nv} A_b = 79580.16$$
$$V_u = 9.094 \leq \emptyset_n R_n = 0.9 * 79.58 = 71.622\ ton \quad ok$$

11.11 Continuity plate requirement :

<u>Column flange design force:</u>

$$\emptyset d = 1$$
$$Yc = 166.52\ cm$$
$$t_{cf} = 1$$
$$F_{yc} = 2400kg/cm2$$
$$M_{cf} = F_{yc}Y_c t_{cf}^2 = 57.6\ ton.m$$
$$\emptyset_d R_n = \frac{\emptyset_d M_{cf}}{(d - t_{bf})} = 271.6\ ton$$

11.12 local column web yielding control :

$$\emptyset d = 1$$
$$Ct = 1$$
$$K_c = 1$$

$F_{yc} = 2400\ kg/cm2$
$t_{cw} = 0.6\ cm$
$t_{bf} = 1.2\ cm$
$t_p = 2.5cm$
$F_{fu} = 95.298\ \ ton$
$R_n = C_t(6Kc + tbf + 2tp) = Fyc\ tcw = 48.528\ ton$
$F_{fu} < \emptyset_d R_n$
According to above calculation Ffu>$\emptyset_d R_n$ *so* Continuty plate is needed.

11.13 PURLIN DESIGN

11.13.1. PROPERTY OF PURLIN(Z180X2.5)

Section Name: Z180*2.5

Properties

Cross-section (axial) area	7.8025	Section modulus about 3 axis	42.3052
Moment of Inertia about 3 axis	386.0346	Section modulus about 2 axis	7.024
Moment of Inertia about 2 axis	45.3049	Plastic modulus about 3 axis	29.4654
Product of Inertia about 2-3	94.7543	Plastic modulus about 2 axis	6.4081
Shear area in 2 direction	4.5172	Radius of Gyration about 3 axis	7.0339
Shear area in 3 direction	2.8393	Radius of Gyration about 2 axis	2.4097
Torsional constant	0.1249	Shear Center Eccentricity (x3)	0.

FIGURE 21-Section Property Of Purlin

According to above table :

$A = 7.80\ cm2$

$J = 0.12\ cm4$

$Ix = 386.06\ cm4$

$Iy = 45.304\ cm4$

$rx = 7.033\ cm$

$ry = 2.41\ cm$

$ho = 18\ cm$

$SY = 42.305\ cm3$

$SX = 7.024\ cm3$

FOR Z 180 :

$\mathrm{D + L = 26 + 50 = 76\ kg/m^2}$

$\mathrm{P_y = 76.\cos 11 = 74.6\ \ kg/m^2}$

$\mathrm{P_x = 76.\sin 11 = 14.5\ \ kg/m^2}$

$$\text{middle of span}: \ M_y = \frac{w.L^2}{8} = \frac{76 \times 1.0 \times 5^2}{8} = 237.5 \text{ kg.m}$$

$$\text{middle of span}: \ M_x = \frac{w.L^2}{360} = \frac{14.5 \times 1 \times 5^2}{360} = 1.00 \text{kg.m}$$

$$f_b = \frac{M_y}{s_y} + 2\frac{M_x}{s_x} = \frac{237.5 \times 100}{42.305} + 2\frac{1.00 \times 100}{7.024} = 561.39 + 28.47 = 589.86 < 1440 \text{ ok}$$

$$\text{moment on sagrod support}: \ M_y = \frac{w.L^2}{9} = \frac{76 \times 1.0 \times 5^2}{9} = 211.11 \text{ kg.m}$$

$$\text{moment on sagrod support}: \ M_x = \frac{w.L^2}{90} = \frac{14.5 \times 1.0 \times 5^2}{90} = 4.02 \text{ kg.m}$$

$$f_b = \frac{M_y}{s_y} + 2\frac{M_x}{s_x} = \frac{211.11 \times 100}{42.305} + 2\frac{4.02 \times 100}{7.024} = 498.99 + 114.46 = 613.45 < \ 1440 \quad \text{ok}$$

11.13.2.UN DEFORMED SHAPE CONTROL:

$$\text{dead + live loads}: \ \Delta = \frac{5 \times q \times L^4}{384 \times E \times I} = \frac{5 \times 0.76 \times 1.0 \times 500^4}{384 \times 2.04 \times 10^6 \times 1350} = 0.22\text{cm} \ < \ \frac{L}{240} = 2.0\text{cm}$$

$$\text{for live loads}: \ \Delta = \frac{5 \times q \times L^4}{384 \times E \times I} = \frac{5 \times 0.5 \times 1.0 \times 500^4}{384 \times 2.04 \times 10^6 \times 1350} = 0.14\text{cm} \ < \ \frac{L}{360} = 1.3 \text{ cm}$$

11.14 Base Plate

Design force :

$$V_{tr} = \frac{M_p}{H} = \frac{ZF_y}{H} = \frac{842.6 \times 2400 \times 10^{-5}}{4.5} = 4.49\ ton$$

$$V_{long} = R_y A_g F_y cos\theta = 1.25 \times 4000 \times 13.68 \times cos\ 50 = 43966\ kg = 43.966 ton$$

Shear check in transverse direction :

According to INBC No.10 section 10-2-6-2-1

$$\emptyset V_n = 0.6 F_y A_w C_v\ \emptyset_v$$

$$\frac{h}{t_w} = \frac{28}{0.6} = 46.67 < 2.24\sqrt{\frac{2.06\times10^6}{2400}} = 65 \quad \text{then}\ \ C_v = 1\ \ \emptyset_v = 1$$

$$\emptyset V_n = 0.6 \times 2400 \times (28 \times 0.6) \times 1 \times 1 = 24.192\ ton > 4.49\ ton$$

Shear check in longitudinal direction :

According to INBC No.10 section 10-2-6-7-2

$$K_v = 1.2$$

$$\frac{h}{t_w} = \frac{\frac{b}{2}}{t_f} = \frac{12.5}{1} = 12.5$$

$$A_w = 2 b_f t_f = 2 \times 25 \times 1 = 50\ cm2$$

$$\frac{h}{t_w} < 1.1\sqrt{\frac{K_v E}{F_y}} = 35.3 \qquad then\ C_v = 1\ \ \emptyset_v = 0.9$$

$$\emptyset V_n = 0.9 \times 0.6 \times 2400 \times 50 \times 1 = 64800\ kg = 64.8\ ton > 43.9\ ton$$

According to above acalculation The column section is ok for shear check .

Bolt control in shear

$$V_{max} = 43.96\ ton$$

$$F_{nv} = 0.55 F_u = 3300\ kg$$

$$\emptyset = 0.75$$

$$A_{nb}=\frac{V_{max}}{\emptyset F_{nv}}=\frac{43960}{0.75\times 3300}=17.76\ cm^2 \quad USE\ 4M27(Area=22.90\ cm2)$$

$$Mpc=F_y Z_{3-3}=2400\times 842.6\times 10^{-5}=20.22\ ton$$

$$\frac{\sum Mpc}{Hs}=\frac{20.22}{4.5}=4.49\ \ Vc2$$

$$Mp=1.1RyFyZc=1.1\times 1.2\times 2400\times 842.6\times 10^{-5}=26.69\ ton.m$$

$$M_{cx}=\min(256.69{,}5.52)=5.5\ ton.m$$

$$f_c=250\ kg/cm2$$

$$\frac{A2}{A1}=1\quad ,\emptyset_c=0.65 \qquad f_{pmax}=\emptyset_c\ 0.85\ f'c\sqrt{\frac{A2}{A1}}=132kg/cm2$$

$$q_{max}=f_{pmax}\times B=6630\ kg/cm2$$

<u>For ordinary & critical load combination</u>

$$M_u=3.33\ ton.m$$

$$P_u=6.87\ ton$$

$$Y_{min}=\frac{P_u}{q_{max}}=\frac{6870}{6630}=1.03cm$$

$$\varepsilon_{max}=\frac{N}{2}-\frac{Ymin}{2}=24.48\ cm$$

$$e=\frac{M_u}{P_u}=\frac{3.33}{6.87}=0.48m=48cm>\varepsilon_{max}\quad tension\ applied.$$

$$d=6cm$$

$$f=\frac{N}{2}-d=\frac{50}{2}-6=19cm$$

$$Y=\left(f+\frac{N}{2}\right)\pm\sqrt{\left(f+\frac{N}{2}\right)^2-\frac{2Mu}{qmax}}=(19+25)\pm\sqrt{(19+25)^2-\frac{2\times 7.5\times 10^5}{6630}}$$

$$Yall=2.66\ cm$$

$$\sum T=qmaxY-Pu=6630\times 2.66=17.635\ ton.m$$

Tension Strength control of anchor bolts :

$d_b = 24mm$

$A_{nb} = 4.52\ cm2$

$T_u = \frac{17.635}{2} = 8.8ton$

$f_{uT} = \frac{T_u}{A_{nb}} = \frac{8.8 \times 1000}{4.52} = 1946\ kg/cm2$

$F_{nT} = 0.75F_u = 4500kg/cm2$

$\emptyset = 0.75$

$R_{nt} = F_{nt}A_{nb} = 4500 \times 4.52 \times 10^{-3} = 20.34\ ton$

$\emptyset R_{nt} = 15.25\ ton\ \geq Tu = 8.8\ ton\ ok$

Shear control of Anchor Bolts:

$Vu = \frac{\sum V}{N} = \frac{4.49}{4} = 1.12\ ton$

$A_{nb} = 4.52\ cm2$

$\emptyset = 0.75$

$F_{nv} = 0.55Fu = 3300kg/cm2$

$R_{nv} = F_{nv}A_{nb} = 3300 \times 4.52 \times 10^{-3} = 18.87\ ton$

$\emptyset R_{nv} = 11.18 > 1.12\ \ ok$

11.15 General requirements of embedment in concrete:

According to ACI318 appendix D:

Concrete breakout strength of anchor in tension : the nominal concrete breakout strength Ncbg shall not exceed

$N_{cbg} = \frac{A_{NC}}{A_{NCO}} \omega_{ec}\omega_{ed,N}\omega_{c,N}\omega_{cp,N}N_b$

$A_{CNO} = 9hef^2$

$A_{NC} = (S1 + 3hef)(S2 + 3hef)$

$K_c = 10$

$N_b = 10\gamma_a\sqrt{f_c}h_{ef}^{1.5} = 10 \times 1 \times \sqrt{25} \times 126.6^{1.5} = 71.2\ KN$

$$\lambda_{ef} = \max\left(\frac{Ca,max}{1.5}, \frac{S1}{3}, \frac{S2}{3}\right) = \left(\frac{\frac{60}{1.5,380}}{3}\right) = 126.6mm$$

$C_{a,max} = \max(Ca1, Ca2, Ca3, Ca4) = 60\ mm$

$A_{NCO} = 9 \times 126.6^2 = 144248.04\ \ mm2$

$A_{NC} = (60 + 3 \times 126.6) \times (60 + 3 \times 126.6) = 193424.04mm2$

$\omega_{ec,N} = 1$

$C_{a,min} = 60\ mm$

$h_{ef} = 126.6\ mm$

$$\omega_{ed,N} = 0.7 + 0.3 \times \frac{60}{1.5 \times 126.6} = 0.79$$

$\omega_{c,N} = 1$

$\omega_{cp,N} = 1$

$$N_{cbg} = \frac{A_{NC}}{A_{NCO}}\omega_{ec}\omega_{ed,N}\omega_{c,N}\omega_{cp,N}N_b = \frac{193424.04}{144248.04} \times 1 \times 0.79 \times 1.25 \times 71.2 = 94.27\ KN$$

$\emptyset = 0.7$

$\emptyset N_{cbg} = 6.72ton > 1.75\ \ ok$

Concrete strength to withstand against tension in braced frame column under combination with Ω factor is acceptable.

Concrete breakout of anchor in shear :

The nominal concrete breakout strength Vcbg in shear shall not exceed :

$$V_{cbg} = \frac{A_{VC}}{A_{VCO}}\omega_{ec,V}\omega_{ed,V}\omega_{c,V}\omega_{h,V}V_b$$

$A_{Vco} = 4.5Ca1^2$

$$\frac{A_V}{A_{Vco}} = 0.66 < n = 2$$

$$h_a = \min(t_f, 1.5Ca_{1,2}) = (1600, 1.5 \times 60) = 90mm$$

$$A_{vc} = 120 \times 90 = 10800\ mm2$$

$$A_{vco} = 4.5Ca1^2 = 16200mm2$$

$$\omega_{ed,V} = 1$$

$$\omega_{e,v} = 1.4$$

$$\omega_{h,v} = \sqrt{\frac{1.5Ca_1}{ha}} = 1$$

$$V_b = \min\left(0.6\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\,\lambda_a\sqrt{fc}\,Ca1^{1.5}\ , 3.7\lambda_a\sqrt{fc}\,Ca1^{1.5}\right) = \min(\ 10.353\ KN, 8.5KN) = 8.5\ KN$$

$$\lambda_a = 1$$

$$V_{cbg} = \frac{A_{VC}}{A_{VCO}}\omega_{ec,V}\omega_{ed,V}\omega_{c,V}\omega_{h,V}V_b = \frac{10800}{16200} \times 1 \times 1.4 \times 1 \times 8.5 = 7.93\ ton > 2\ ton\ \ ok$$

-REQUIRED THICKNESS

Maximum Axial Load according to SAP2000 model is about 4.3 ton Under critical load combination:

$t = l\sqrt{\frac{2P_U}{0.9F_yBN}}$ =11.7$\sqrt{\frac{2x4300}{0.9x2400x50x50}}$ = 0.46cm   used th=2 cm

$m = \frac{N-0.95d}{2} = \frac{500-0.95*280}{2}$=117

$n = \frac{B-0.8bf}{2} = \frac{500-0.8*250}{2}$=150

$$\lambda n' = \lambda\frac{\sqrt{dbf}}{4} = 1 * \frac{\sqrt{240 * 250}}{4} = 61$$

L=max(m,n,λn)=117

12.0 FOUNDATION DESIGN

12.1 Soil pressure and settlement

Until finalize of geotechnical report for this area we consider => qa= 2kg/cm2

Based on Bowels experimental formula for subgrade modulus => Ks = 1.345qall

Loading used for foundation design, have been received from SAP analysis.

12.2 DESIGN

Concrete Foundation are designed according to ACI 318-14. Required loads are derived from SAP data, and design process will be done according to ACI code based on ultimate strength procedure.

$f'_c = 25\text{Mpa} \qquad f_y = 400\text{Ma}$

12.3 FOUNDATION DESIGN CONTROL

12.3.1CHECK OF STRESS FOR FOUNDATION

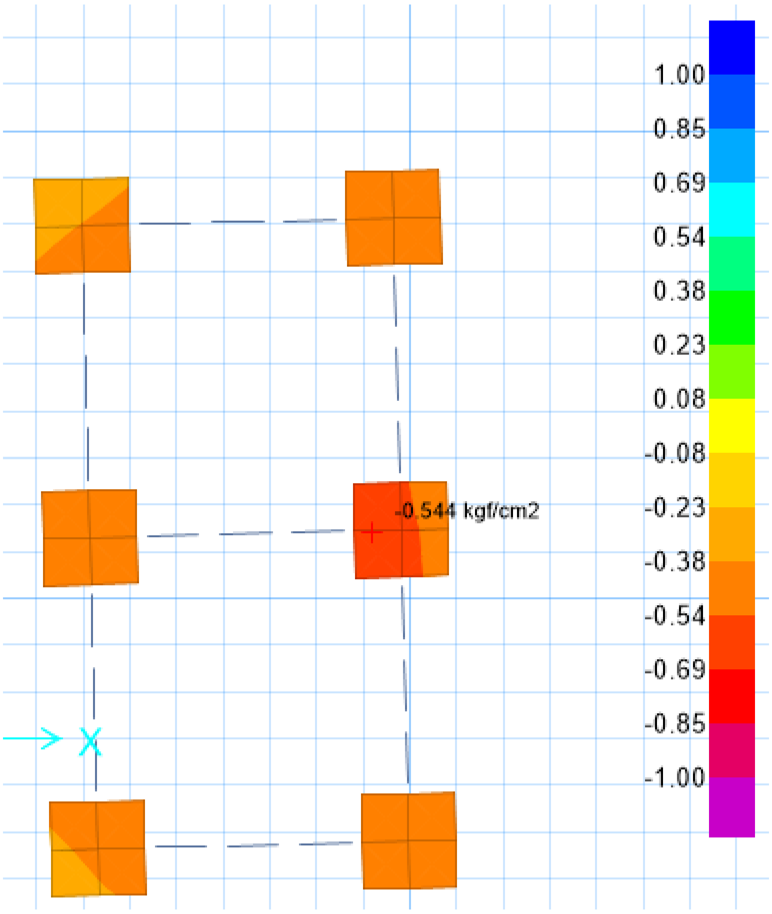


FIGURE 22 - Check of Stress for Foundation (kg/cm2)

According to SAFE report, Max soil pressure under the foundation is:

$q_n = 0.544\text{kg/cm}^2 \;<\; 2\;\; \text{kg/cm}^2\;\; \text{ok}$

12.3.2. CHECK OF DISPLACEMENT FOR FOUNDATION

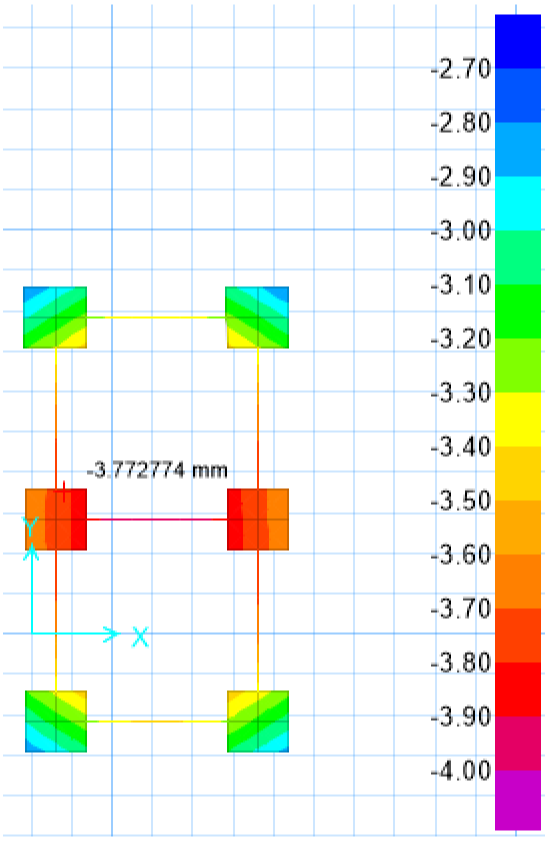


FIGURE23- Check of Displacement for Foundation(mm)

According to SAFE report, Max soil displacement under the foundation is:

$$d_n = 3.75 \text{ mm} \ < \ 25 \ \text{ cm} \quad \text{ok}$$

12.3.3 REINFORCING CONTROL

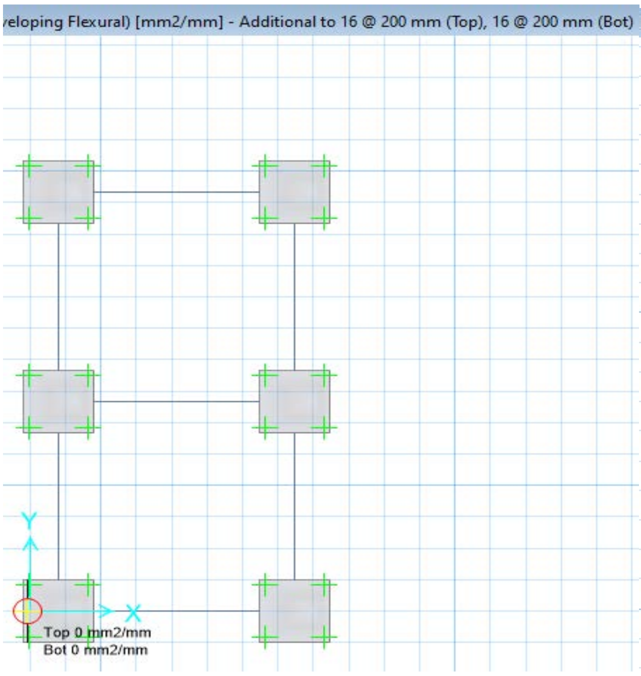
Minimum rebar for foundation:

$$A_{s\,min} = 0.0018bh$$

$$A_{smin} = \frac{1}{2} 0.0018 \text{ bh} = \frac{1}{2} 0.0018 \times 100 \times 50 = 4.5 \quad \text{cm}^2/\text{m}$$

$$A_{s\,used} = \emptyset 16@200 = 10.05\text{cm}^2$$

12.3.4PUNCHING SHEAR CONTROL



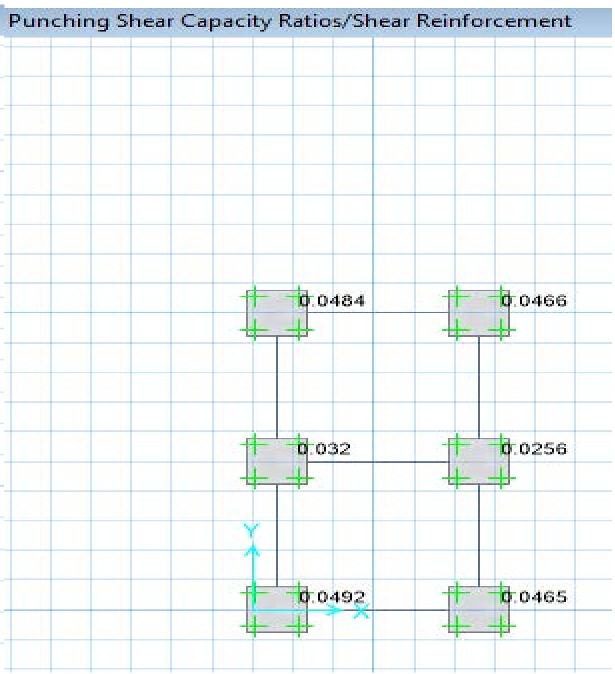


FIGURE 24: ADDITIONAL REINFOREMENT **FIGURE 25: PUNCH SHEAR CONTROL**