NISOC	نگهداشت و افزایش تولید میدان نفتی بینک سطح الارض احداث ردیف تراکم گاز در ایستگاه جمع آوری بینک	HIRGAN ENERGY

شماره پیمان:	CALCULATION REPORT FOR GAS COMPRESSORS SHELTER								شماره صفحه : 1 از 36
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	BK	GCS	PEDCO	120	ST	CN	0003	D00	

طرح نگهداشت و افزایش تولید 27 مخزن

CALCULATION REPORT FOR GAS COMPRESSORS SHELTER

نگهداشت و افزایش تولید میدان نفتی بینک

D00	JUL. 2022	IFC	R.Berlouie	M.Fakharian	M.Mehrshad	
Rev.	**Date**	**Purpose of Issue/Status**	**Prepared by:**	**Checked by:**	**Approved by:**	**CLIENT Approval**

Class:2 | **CLIENT Doc. Number:F0Z-709119**

Status:

- **IDC: Inter-Discipline Check**
- **IFC: Issued For Comment**
- **IFA: Issued For Approval**
- **AFD: Approved For Design**
- **AFC: Approved For Construction**
- **AFP: Approved For Purchase**
- **AFQ:** Approved For Quotation
- **IFI: Issued For Information**
- **AB-R: As-Built for CLIENT Review**
- **AB-A: As-Built –Approved**

REVISION RECORD SHEET

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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.

GENERAL DEFINITION

The following terms shall be used in this document.

CLIENT:	National Iranian South Oilfields Company (NISOC)
PROJECT:	Binak Oilfield Development – Surface Fcilities; New Gas Compressor Station
EPD/EPC CONTRACTOR (GC):	Petro Iran Development Company (PEDCO)
EPC CONTRACTOR:	Joint Venture of : Hirgan Energy – Design & Inspection (D&I) Companies
VENDOR:	The firm or person who will fabricate the equipment or material.
EXECUTOR:	Executor is the party which carries out all or part of construction and/or commissioning for the project.
THIRD PARTY INSPECTOR (TPI):	The firm appointed by EPD/EPC CONTRACTOR (GC) and approved by CLIENT (in writing) for the inspection of goods.
SHALL:	Is used where a provision is mandatory.
SHOULD:	Is used where a provision is advisory only.
WILL:	Is normally used in connection with the action by CLIENT rather than by an EPC/EPD CONTRACTOR, supplier or VENDOR.
MAY:	Is used where a provision is completely discretionary.

2.0 SCOPE

This report covers the structure calculation report of the “GAS COMPRESSORS SHELTER ”. The structure modelled by “SAP” software & the foundation modelled by “SAFE” software.

3.0 NORMATIVE REFERENCES

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-GNRAL-PEDCO-000-ST-SP-0003 SPECIFICATION FOR FABRICATION OF STEEL STRUCTURES
- BK-GNRAL-PEDCO-000-ST-SP-0004 SPECIFICATION FOR GROUTING
- BK-GNRAL-PEDCO-000-ST-SP-0005 SPECIFICATION FOR ERECTION OF STEEL STRUCTURES
- BK-GNRAL-PEDCO-000-ST-DW-0002 STANDARD DRAWING FOR ANCHOR BOLTS
- BK-GNRAL-PEDCO-000-ST-DW-0011 GENERAL NOTES-STEEL STRUCTURES
- BK-GNRAL-PEDCO-000-ST-DW-0014 STANDARD DRAWING FOR STRUCTURAL BUILT-UP SECTIONS

3.4 ENVIRONMENTAL DATA

Refer to "Process Basis of Design; Doc. No. ----------------------".

3.5 ORDER OF PRECEDENCE

In case of any conflict between the contents of this document or any discrepancy between this document and other project documents or reference standards, this issue must be reported to the CLIENT. The final decision in this situation will be made by CLIENT.

4.0 MATERIAL PROPERTIES

Material properties are delivered in the following table.

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.

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

6.0 INPUT DATA

6.1 DESIGN LOAD

6.1.1. DEAD LOAD

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

Corogated sheet : $8\ \mathrm{kg/m^2}$ Z Purlin : $8\ \mathrm{kg/m^2}$ Insulation : $10\ \mathrm{kg/m^2}$

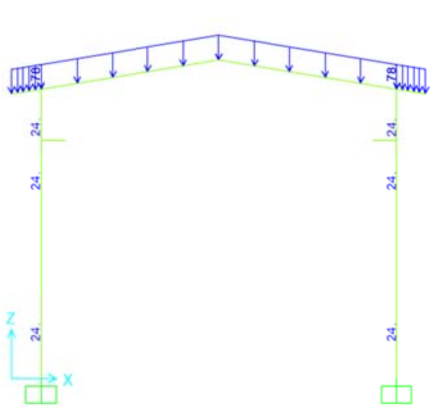
$$\sum \mathrm{sum} = 26\ \mathrm{kg/m^2}$$

Roof weight is assigned in software 26 kg/m2.

- At ended frame : $26 \times 3 = 78\ kg/m$ At middle frame : $26 \times 6 = 156\ kg/m$

For wall: Consider galvanize sheet with weight 8kg/m2 (8 kg/m2 assign to the model)

- At ended frame : $8 \times 3 = 24\ kg/m$ At middle frame : $6 \times 8 = 48\ kg/m$



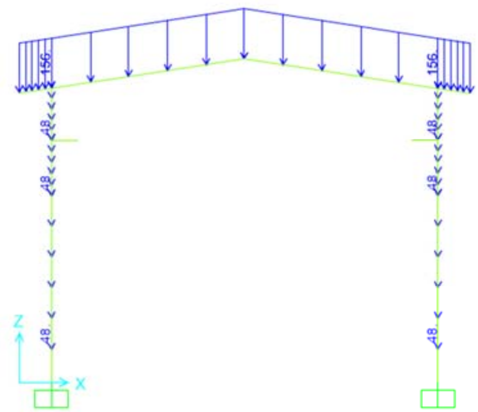


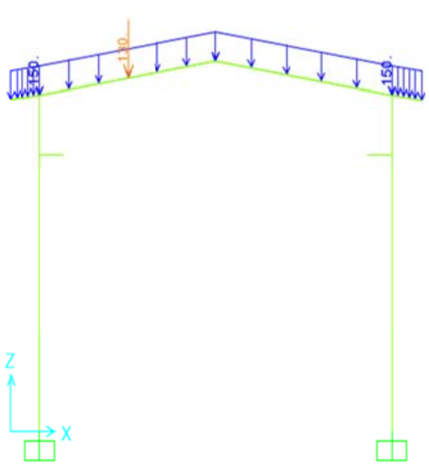
Figure 1-applied Dead load on ended axe(1&3&6&8&11&13) (kg/m) **Figure 2-**applied Dead load on middle axe (2&7&12) (kg/m)

6.1.2. 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 : $3 \times 50 = 150\ kg/m$ At middle frame : $6 \times 50 = 300\ kg/m$



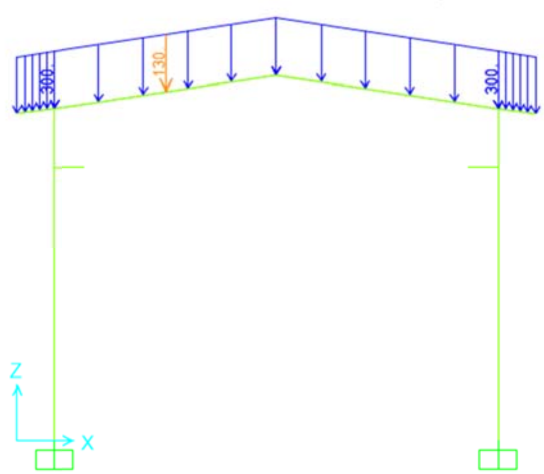


Figure 3-Applied live Load on frame (1&3&6&8&11&13)(kg/m) **Figure 4-**Applied live Load on frame (2&7&10)(kg/m)

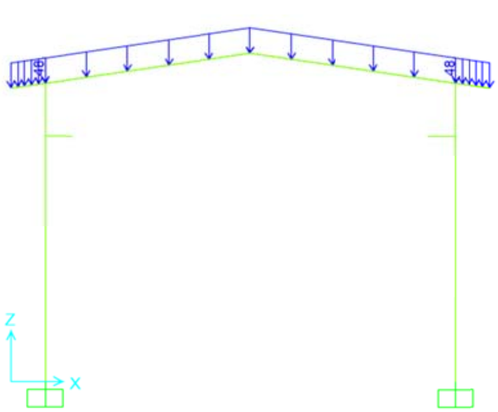
6.1.3. SNOW LOADS

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:

Ps=25 kg/m2 Is=1 Cs= 0.91 (slope 15)=1-$\frac{\alpha-\alpha_0}{70-\alpha_0} = 1 - \frac{15-10}{70-10} = 0.91$

Ch=1 Cn=0.8 $P_r = P_s C_n C_h I_S C_s = 16.00 \frac{kg}{m2}$

- At ended frame : $3 \times 16 = 48\ kg/m$ At middle frame : $6 \times 48 = 96\ kg/m$



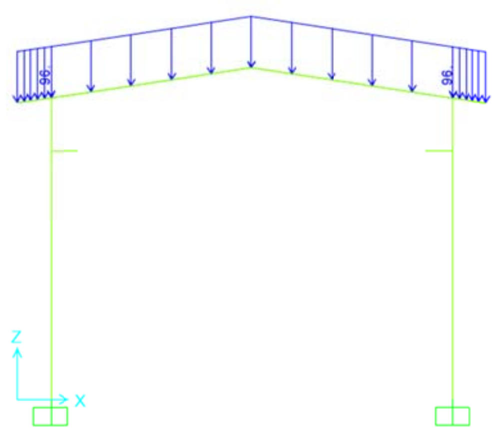
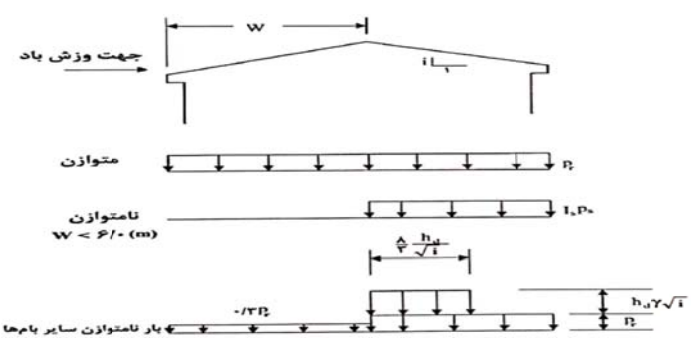


Figure 5-Applied Snow Load on frame (1&3&6&8&11&13)(kg/m) **Figure 6**-Applied Snow Load on frame (2&7&10)(kg/m)

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:



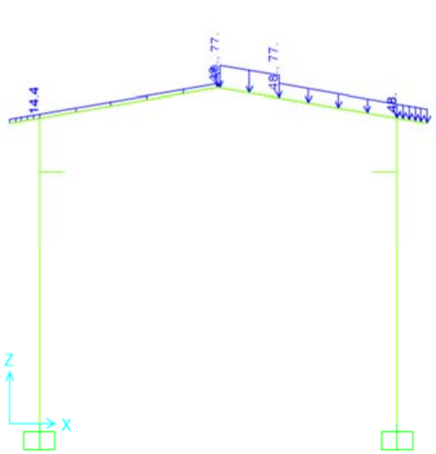
$$h_d = ۰٫۱۲\sqrt[۳]{l_u}\sqrt[۴]{۱۰۰P_s + ۵۰} - ۰٫۵$$

$$h_d = 0.12\sqrt[3]{l_u}\sqrt[4]{100P_s + 50} - 0.5 = 0.12\sqrt[3]{6}\sqrt[4]{100 * 0.25 + 50} - 0.5 = 0.142\ m$$

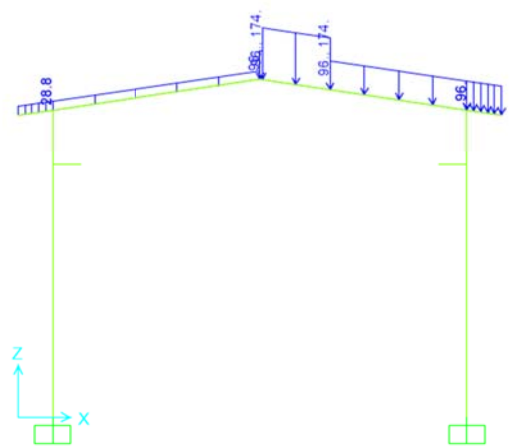
$$x = \frac{8}{3} * \frac{hd}{\sqrt{i}} \qquad x = \frac{8}{3} * \frac{0.142}{\sqrt{0.15}} = 0.98$$

$$\gamma = 0.43P_g + 2.2 = 0.43 * 0.25 + 2.2 = 2.31KN/m3$$

$$h_d\gamma\sqrt{i} = 0.142 * 2.31 * \sqrt{0.15} = \frac{0.13KN}{m2} = 13kg/m2$$



Load on frame (1&3&6&8&11&13) (kg/m)



Load on frame (2&7&12) (kg/m)

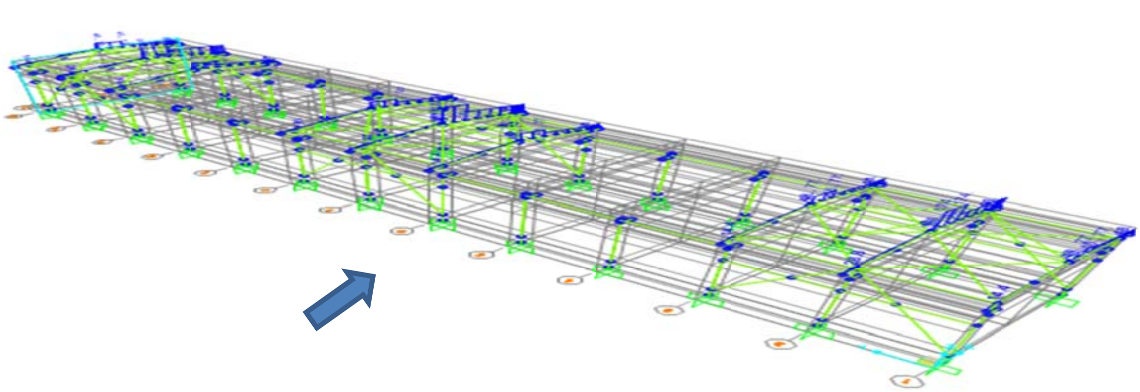
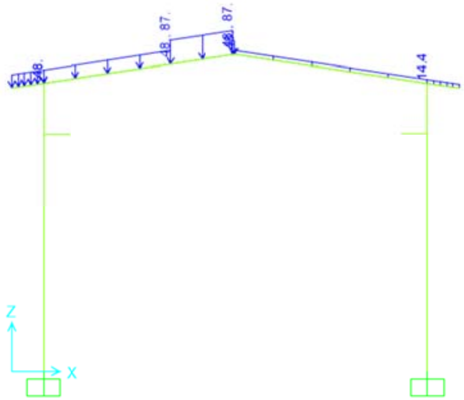
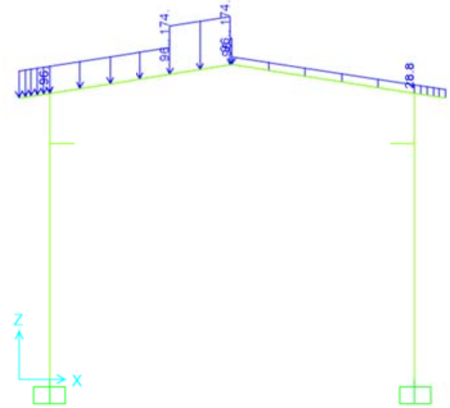


Figure 7-Applied Unbalanced SNOW Load on frame-SP-



Load on frame (1&3&6&8&11&13) (kg/m)



Load on frame (2&7&12) (kg/m)

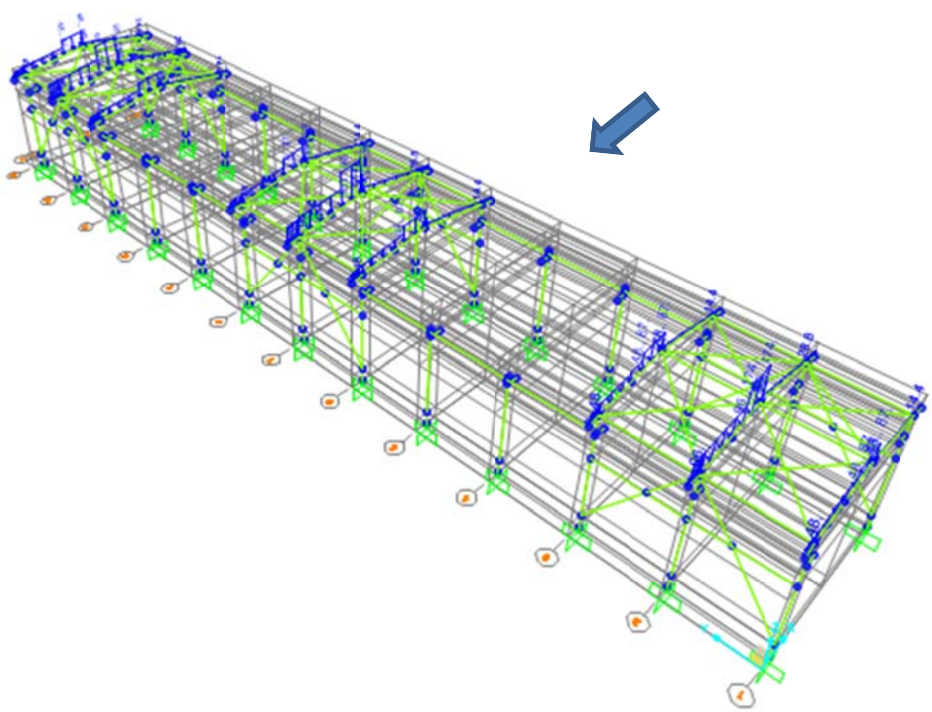


Figure 8-Applied Unbalanced SNOW Load on frame-SN-

6.1.4. 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)

- **For X direction**

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

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

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

$\mathrm{I} = 1$

B(0.2s)=2.75(According to Soil type) B(1s)=1.9(According to Soil type)

$\mathrm{T} = 0.3$ according to sap software

T0<T<Ts $\mathrm{Sa} = \mathrm{A}.\mathrm{B}(0.2\mathrm{s}) = 0.825$ $\mathrm{C_u x} = \frac{\mathrm{S_a.I}}{\mathrm{R_u}}$=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)

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

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

$\mathrm{I} = 1$

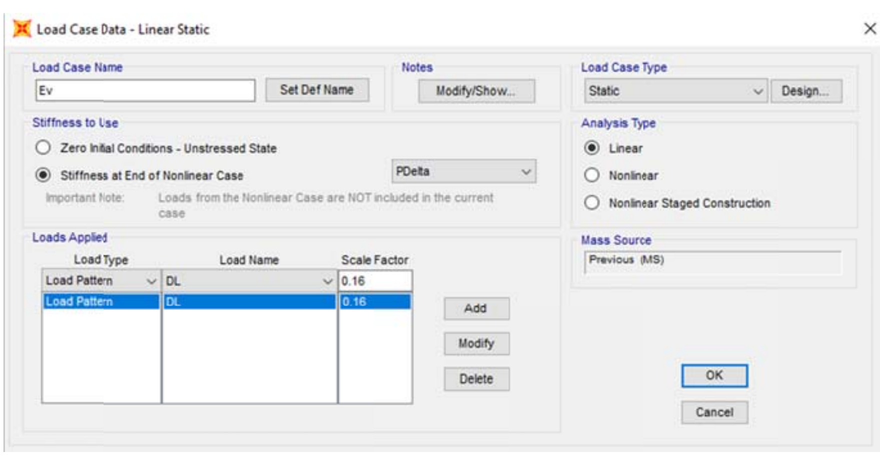
B(0.2s)=2.75(According to Soil type) B(1s)=1.9(According to Soil type)

$\mathrm{T} = 0.3$ according to sap model

T0<T<Ts $\mathrm{Sa} = \mathrm{A.B}(0.2s) = 0.825$ $\mathrm{C_u y} = \frac{\mathrm{S_a.I}}{\mathrm{R_u}}$=0.254

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

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



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

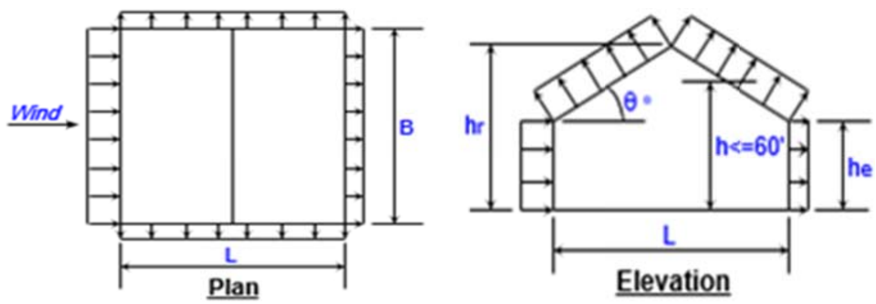
6.1.5. WIND LOADS

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

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

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

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



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 =9.9 m   Eave height =9 m   Building width=12 m   Building length=72m

Roof type =Gable

Topo factor Kzt=1

Direct factor kd=1

Enclose =yes

Hurricane rfegion =no

Roof angle =8.50

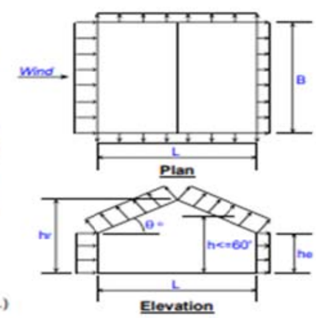
Mean roof height =9.45 m

WIND LOADING ANALYSIS - Main Wind-Force Resisting System
Per ASCE 7-10 Code for Enclosed or Partially Enclosed Buildings
Using Method 2: Analytical Procedure (Section 27 & 28) for Low-Rise Buildings

VINBC(km/h) 120

Input Data:

wind Speed,V=	64.51046	
Bldg. Classification =	I	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	30.16	ft. (hr >= he)
Eave Height, he =	27.21	ft. (he <= hr)
Building Width =	39.34	ft. (Normal to Building Ridge)
Building Length =	236.07	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6)
Enclosed? (Y/N)	N	(Sect. 26.2 & Table 26.11-1)
Hurricane Region?	N	



Resulting Parameters and Coefficients:

Roof Angle, θ =	8.50	deg.
Mean Roof Ht., h =	28.69	ft. (h = he, for angle <=10 deg.)

Check Criteria for a Low-Rise Building:
1. Is h <= 60' ? Yes, O.K. 2. Is h <= Lesser of L or B? Yes, O.K.

External Pressure Coeff's., GCpf (Fig. 28.4-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.11-1):

+GCpi Coef. =	0.55	(positive internal pressure)
-GCpi Coef. =	-0.55	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 28.3-1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 28.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.96	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Kzt*Kd*V^2 (Sect. 28.3.2, Eq. 28.3-1)
qh = 8.71 psf qh = 0.00256*Kh*Kzt*Kd*V^2 (qz evaluated at z = h)
Design Net External Wind Pressures (Sect. 28.4.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.4-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.4-1):

a =	3.93	ft.
2*a =	7.87	ft.

3.6432

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	0.43	-5.09	41.69	Zone 1	0.40	-6.38	40.40
Zone 2	-0.69	-52.73	-5.95	Zone 2	-0.69	-52.73	-5.95
Zone 3	-0.40	-40.21	6.56	Zone 3	-0.37	-39.12	7.65
Zone 4	-0.32	-37.11	9.67	Zone 4	-0.29	-35.72	11.06
Zone 5	---	---	---	Zone 5	-0.45	-42.53	4.25
Zone 6	---	---	---	Zone 6	-0.45	-42.53	4.25
Zone 1E	0.65	4.44	51.21	Zone 1E	0.61	2.55	49.33
Zone 2E	-1.07	-68.89	-22.11	Zone 2E	-1.07	-68.89	-22.11
Zone 3E	-0.57	-47.51	-0.74	Zone 3E	-0.53	-45.93	0.85
Zone 4E	-0.48	-43.76	3.02	Zone 4E	-0.43	-41.67	5.10
Zone 5E	---	---	---	Zone 5E	0.61	2.55	49.33
Zone 6E	---	---	---	Zone 6E	-0.43	-41.67	5.10

Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:

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 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.

Figure 9-Wind Load calculation((kg/m2))

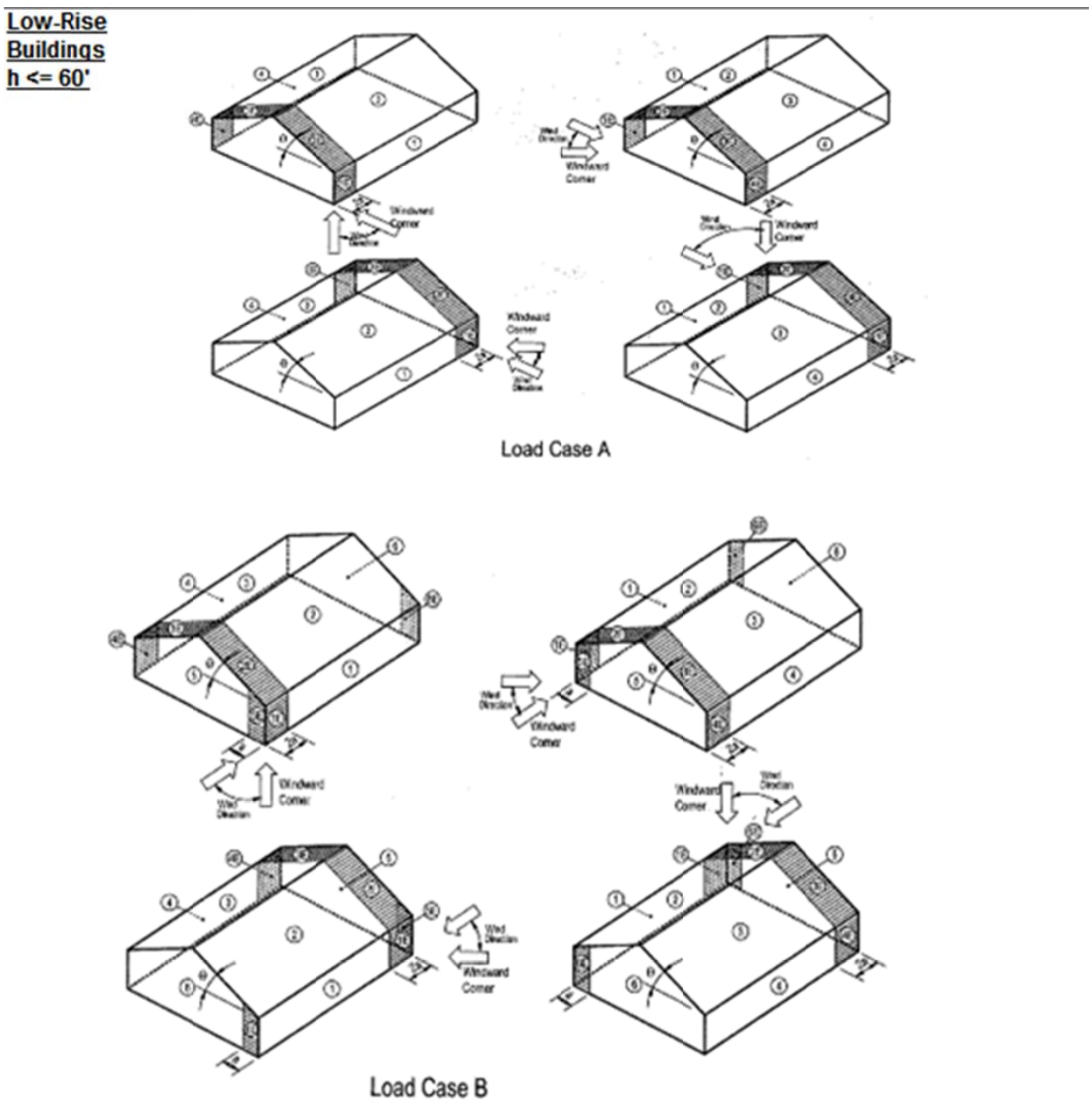


Figure 10-Wind Load Direction

Load Case A

(Wx) Due to G+Cpi :

1: $Wx = -5.09 \times 6 = -30.54\ \frac{kg}{m}$

2: $Wx = -52.73 \times 6 = -316.38\ \frac{kg}{m}$

3: $Wx = -40.21 \times 6 = -241.26\ \frac{kg}{m}$

4: $Wx = -37.11 \times 6 = -222.66\ \frac{kg}{m}$

Wx1: Due to G-Cpi

1: $Wx = 41.69 \times 6 = 250.14\ \frac{kg}{m}$

2: $Wx = -5.95 \times 6 = -35.70\ \frac{kg}{m}$

3: $Wx = 6.56 \times 6 = 39.36\ \frac{kg}{m}$

4: $Wx = 9.67 \times 6 = 58.02\ \frac{kg}{m}$

(WxE) Due to G+Cpi :

1: $Wx = 4.44 \times 3 = 13.32\ \frac{kg}{m}$

2: $Wx = -68.88 \times 3 = -206.64\ \frac{kg}{m}$

3: $Wx = -47.51 \times 3 = -142.53\ \frac{kg}{m}$

4: $Wx = -43.76 \times 3 = -131.28\ \frac{kg}{m}$

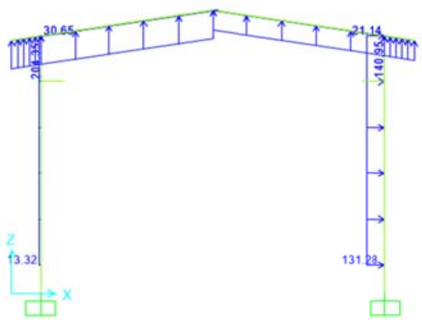
Wx1E: Due to G-Cpi

1: $Wx = 51.24 \times 3 = 153.63\ \frac{kg}{m}$

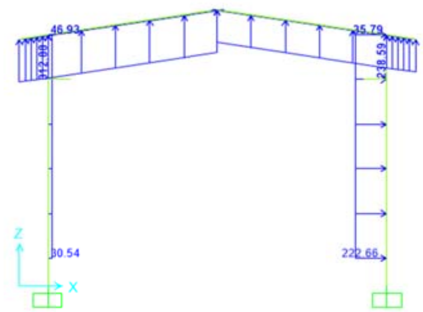
2: $Wx = -22.11 \times 3 = -66.33\ \frac{kg}{m}$

3: $Wx = -0.74 \times 3 = -2.22\ \frac{kg}{m}$

4: $Wx = 3.02 \times 3 = 9.06\ \frac{kg}{m}$



Load on frame (1&13) (kg/m)



Load on frame (2&3&6&7&8&11&12) (kg/m)

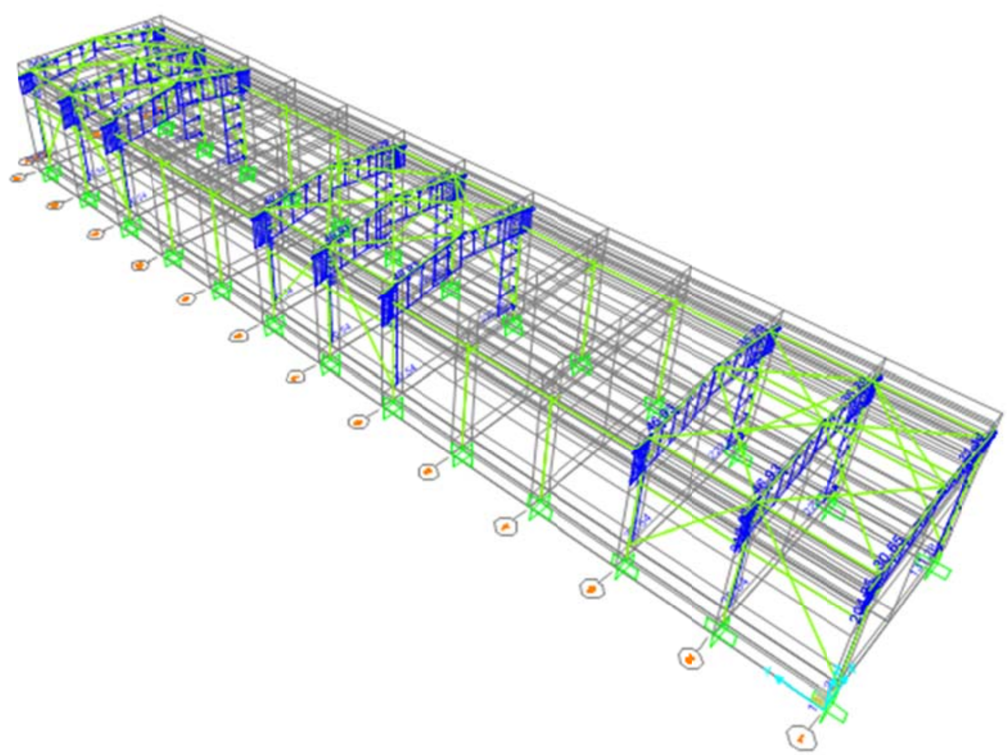
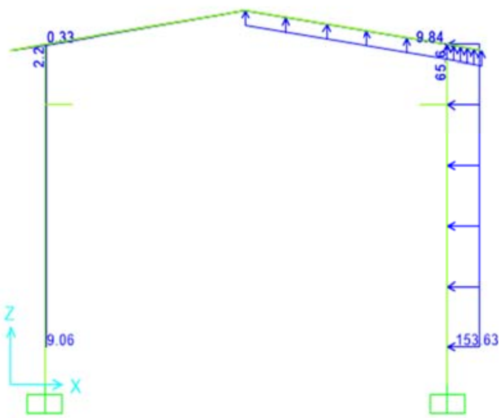
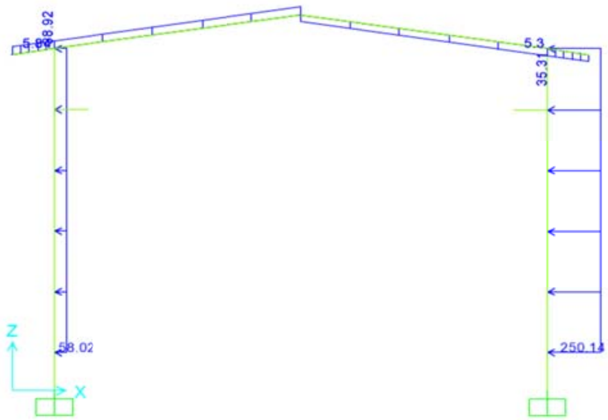


Figure 11-Applied Wind Load on frame-Wx-



Load on frame (1&13) (kg/m)



Load on frame (2&3&6&7&8&11&12) (kg/m)

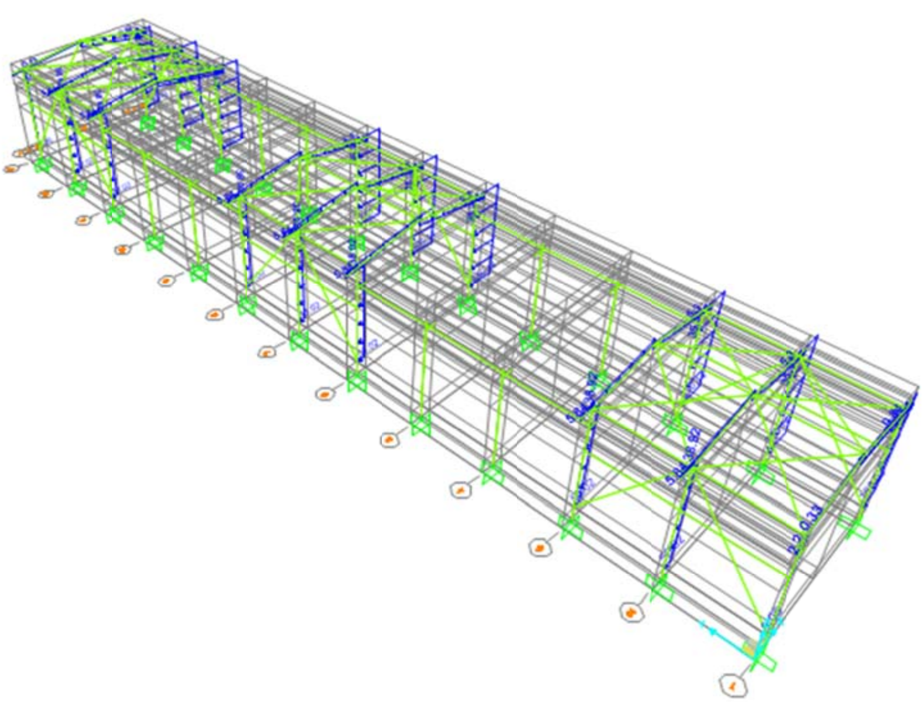


Figure 12-Applied Wind Load on frame-Wx1-

Load Case B

(Wy) Due to G+Cpi :

1: $Wy = -6.38 \times 6 = -38.28\ \frac{kg}{m}$

2: $Wy = -52.73 \times 6 = -316.38\ \frac{kg}{m}$

3: $Wy = -39.12 \times 6 = -234.72\ \frac{kg}{m}$

4: $Wy = -35.72 \times 6 = -214.32\ \frac{kg}{m}$

5: $Wy = -42.53 \times 6 = -255.18\ \frac{kg}{m}$

6: $Wy = -42.53 \times 6 = -255.18\ \frac{kg}{m}$

Wy1: Due to G-Cpi

1: $Wy = 40.40 \times 6 = 242.40\ \frac{kg}{m}$

2: $Wy = -5.95 \times 6 = -35.70\ \frac{kg}{m}$

3: $Wy = 7.65 \times 6 = 45.90\ \frac{kg}{m}$

4: $Wy = 11.06 \times 6 = 66.36\ \frac{kg}{m}$

5: $Wy = 4.25 \times 6 = 25.50\ \frac{kg}{m}$

6: $Wy = 4.25 \times 6 = 25.50\ \frac{kg}{m}$

(WyE) Due to G+Cpi :

1: $Wy = 2.55 \times 3 = 7.65\ \frac{kg}{m}$

2: $Wy = -68.88 \times 3 = -206.64\ \frac{kg}{m}$

3: $Wy = -45.93 \times 3 = -137.79\ \frac{kg}{m}$

4: $Wy = -41.67 \times 3 = -125.01\ \frac{kg}{m}$

5: $Wy = 2.55 \times 3 = 7.65\ \frac{kg}{m}$

6: $Wy = -41.67 \times 3 = -125.01\ \frac{kg}{m}$

Wy1E: Due to G-Cpi

1: $Wy = 49.33 \times 3 = 147.99\ \frac{kg}{m}$

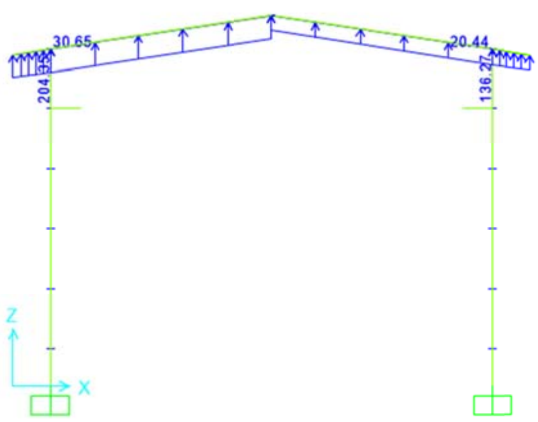
2: $Wy = -22.11 \times 3 = -66.33\ \frac{kg}{m}$

3: $Wy = 0.85 \times 3 = 2.55\ \frac{kg}{m}$

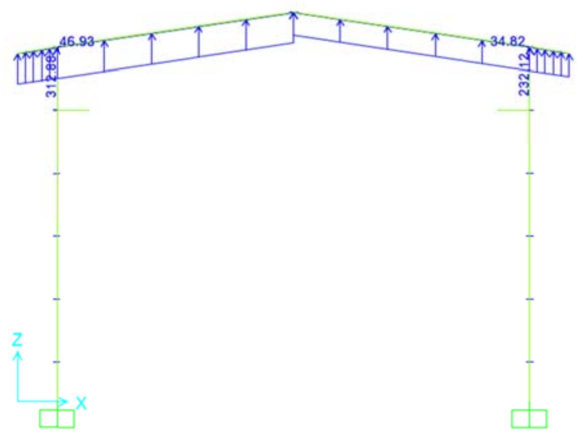
4: $Wy = 5.10 \times 3 = 15.30\ \frac{kg}{m}$

5: $Wy = 49.33 \times 3 = 147.99\ \frac{kg}{m}$

6: $Wy = 5.10 \times 6 = 15.30\ \frac{kg}{m}$



Load on frame (1&13) (kg/m)



Load on frame (2&3&6&7&8&11&12) (kg/m)

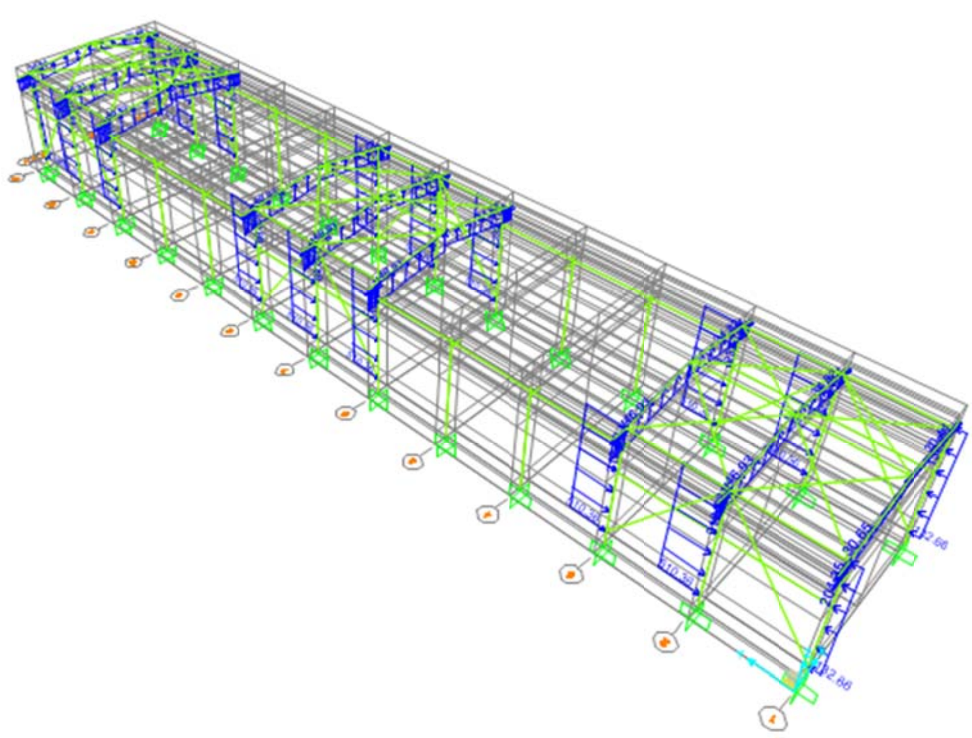
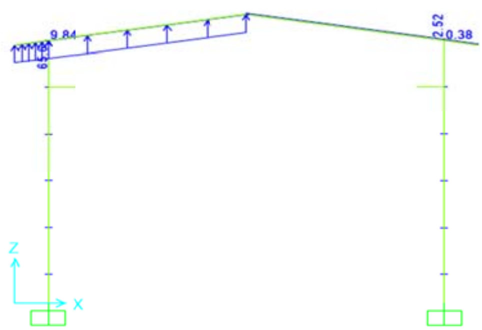
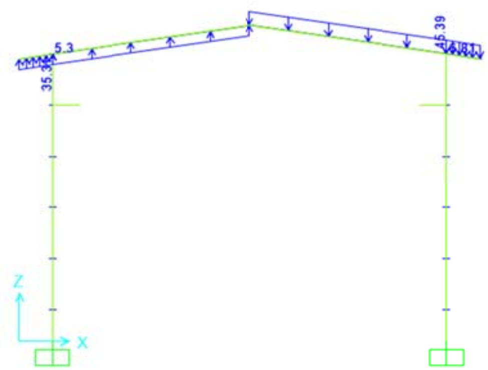


Figure 13-Applied Wind Load on frame-Wy-



Load on frame (1&13) (kg/m)



Load on frame (2&3&6&7&8&11&12) (kg/m)

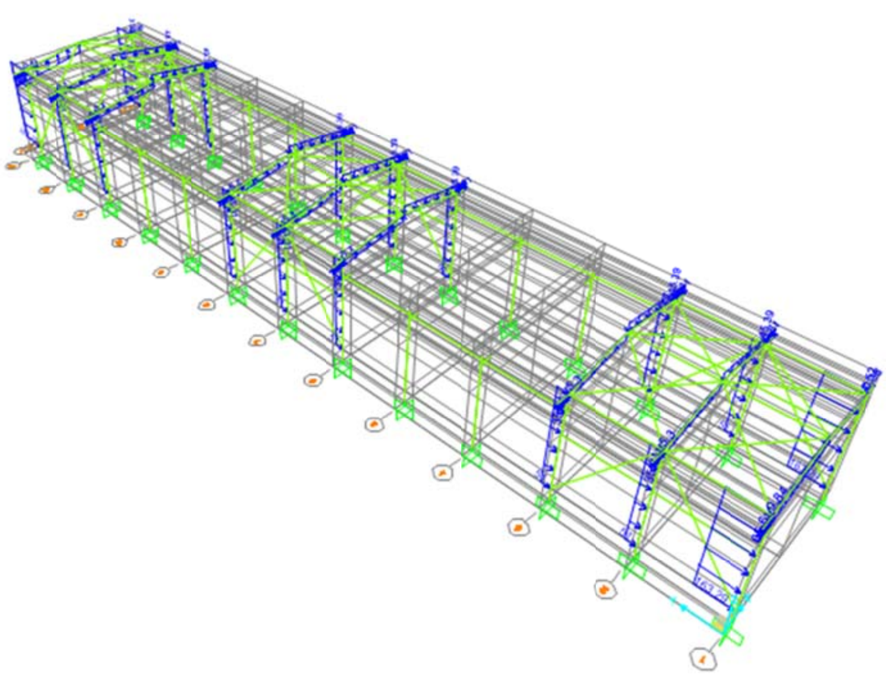


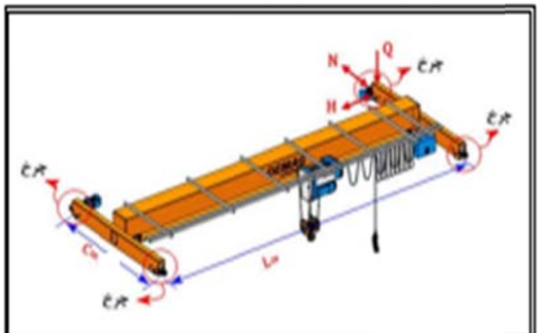
Figure 14-Applied Wind Load on frame-Wy1-

6.1.6. CRAINE LOADS

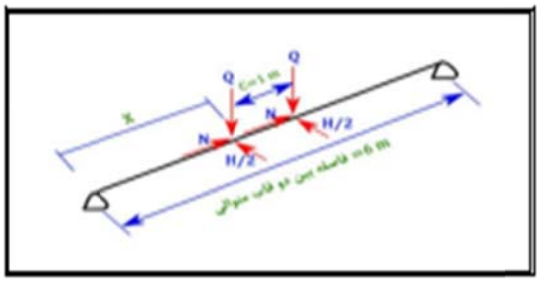
For the shelter structure of the main gas compressors, an electric overhead crane with a capacity of 10 tons is provided in annex 10 (description of works in the contractor's commitment).

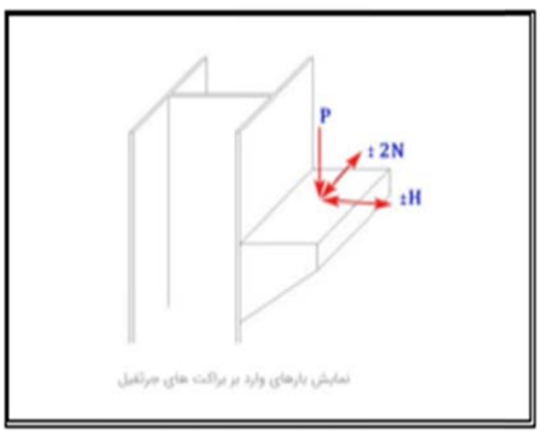
MAXIMUM LOAD OF CRANE BRIDGE WHEELS			
W1	operation load	10,000	Kg
W2	chariot weight	500	Kg
W3	bridge load	50	Kg/m
W4	weight of longitudinal rails and beams	200	Kg/m
L	span length	10.8	m
a	wheel spacing	3	m
n	number of wheels	4	
q	load of each wheel	2625	Kg
R	maximum load of crane bridge wheels	4790.83333	Kg
Q	vertical impact force	5988.54167	Kg
H	lateral load	2100	Kg
N	horizontal load	479.083333	Kg

maximum anchor and shear			
b	distance between frames	6	m
C	distance between wheels	1	m
X	distance from bearing	2.75	m
My	maximum anchor	2647	kg.m
Mz		15996	kg.m
Vy	maximum shear	1925	kg
Vz		11579	kg
P		10781.6667	Kg
H		2100	Kg
2N		958.166667	Kg

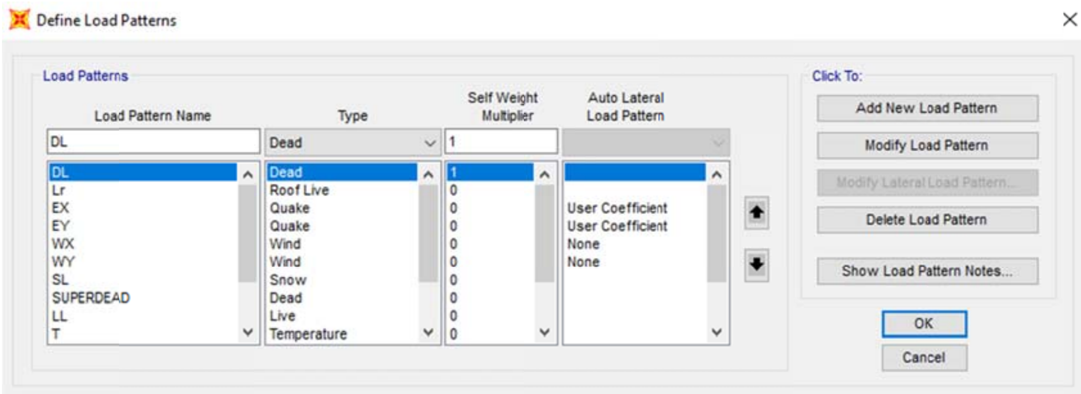


نیروهای وارد بر هر چرخ





6.2 SAP loading table



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

7.0 STRUCTURE ANALYSIS AND DESIGN

7.1 ANALYSIS

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

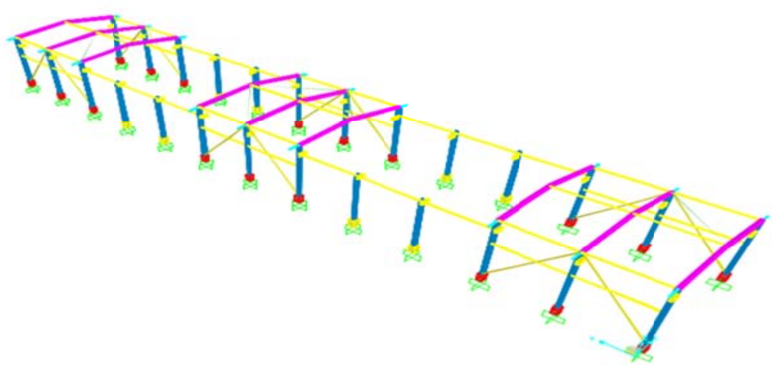


Figure 15 - 3D view of SAP model

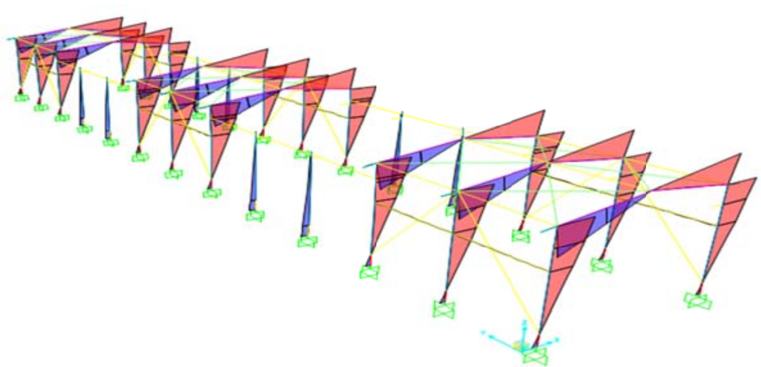


Figure 16 - Moment 3-3 under Ex load

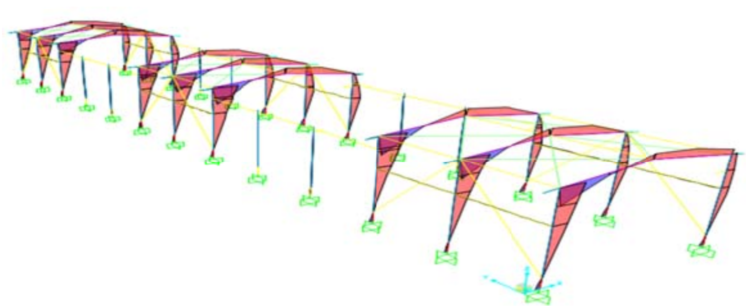


Figure 17 - Moment 3-3 under Wx load

7.2 Control of Columns Required Axial Strength

According to AISC 341-10 section D1.4a. columns axial strength shall be checked with load combination including amplified seismic load.

Omega X = 3

Omega Y = 2

Load combination for amplified seismic control:

1- D+0.75L+0.75S±2EY (compression)

2- 0.6D±2EY (tension)

According to design file (sap model) axial load in columns in above combination checked an all columns are acceptable .

8.0 STRUCTURAL DESIGN RESULTS

8.1 Graphical output

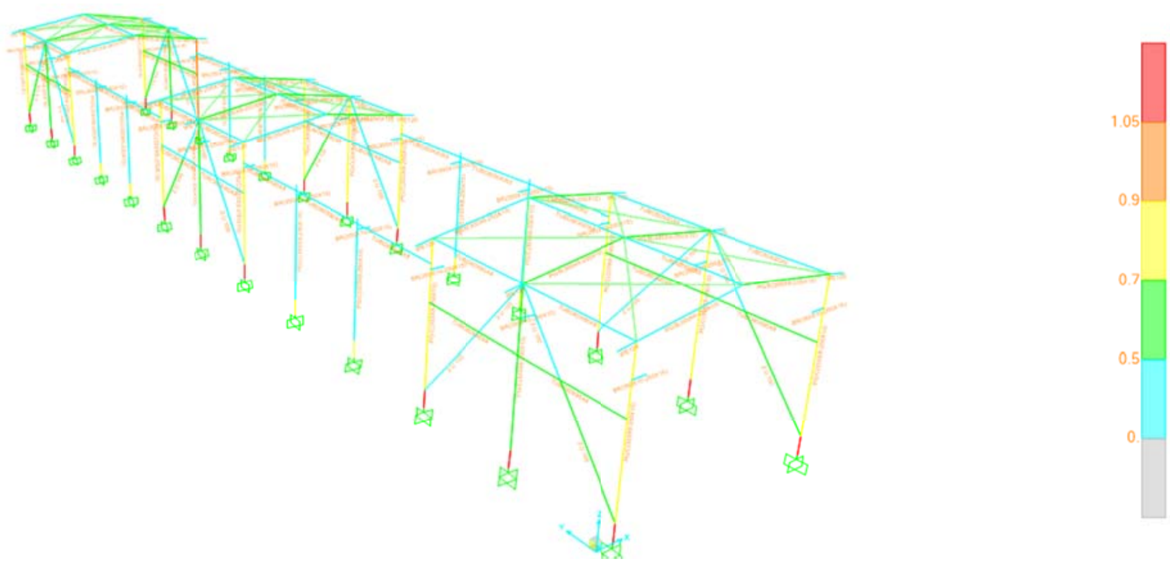
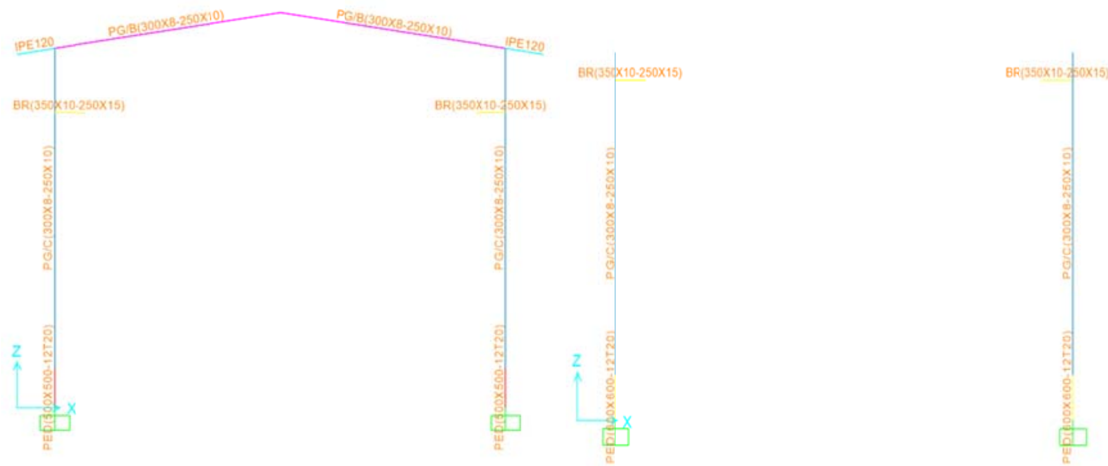


Figure 18 – Design Results

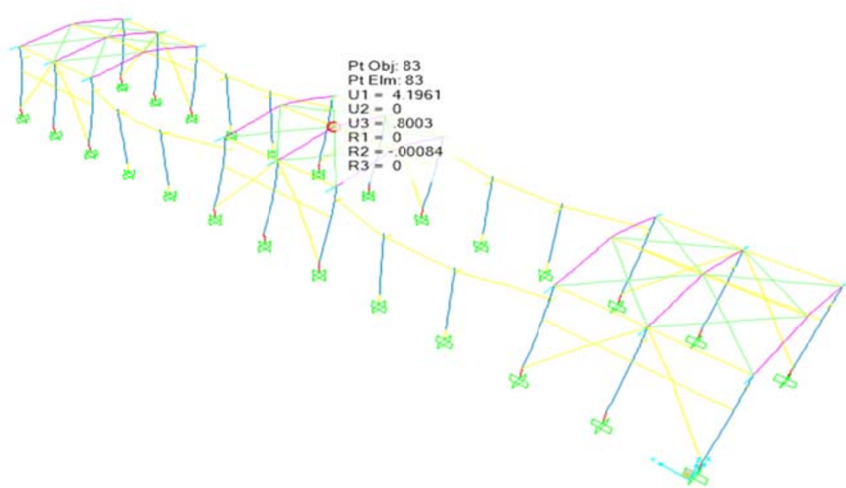


Section of frame (1&2&3&6&7&8&11&12&13)

Section of frame (4&5&9&10)

Figure 19 : Frames

8.2 DRIFT CONTROL



Maximum displacement according to above output from sap model under critical service load combination is about 4.19 cm which is less than allowable drift.

allowable drift is $\frac{h}{180} = 5.5cm$

9.0 STRUCTURAL CONNECTIONS

9.1 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	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$	
	Flanges of H-pile sections per Section D4	b/t	$0.45\sqrt{E/F_y}$	not applicable	
	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]	
	Flanges of boxed I-shaped sections and built-up box sections	b/t			
	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t			
	Webs of rolled or built-up I-shaped sections used as diagonal braces	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$	

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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 $C_a \le 0.125$ $3.76\sqrt{E/F_y}(1-2.75C_a)$	
	Side plates of boxed I-shaped sections used as beams or columns	h/t	For $C_a > 0.125$ $0.77\sqrt{E/F_y}(2.93-C_a) \ge 1.49\sqrt{E/F_y}$ where	For $C_a > 0.125$ $1.12\sqrt{E/F_y}(2.33-C_a) \ge 1.49\sqrt{E/F_y}$ where	
	Webs of built-up box sections used as beams or columns	h/t	$C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	$C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$	not applicable	
	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	$1.4\sqrt{E/F_y}$	$2.26\sqrt{E/F_y}$	
	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 [illegible]
[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 $0.07E/F_y$.

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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)

For eight-bolt connections (8ES):

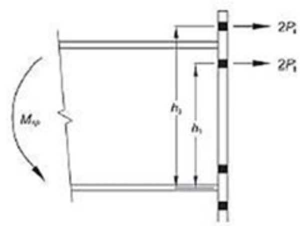
$$d_{b\ \text{req'd}} = \sqrt{\frac{2\,M_f}{\pi\phi_n F_{nt}\left(h_1 + h_2 + h_3 + h_4\right)}} \tag{6.10-4}$$

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)



9.2 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.

ϕ_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).

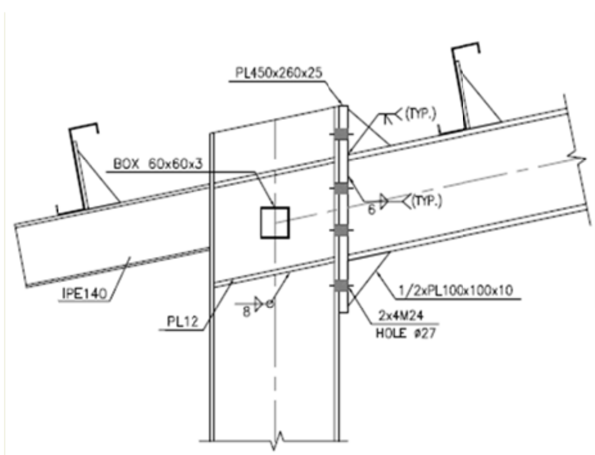
phi Rn cripp. = unstiffened column web crippling strength at beam compression flange (according to AISC358-10).

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).



9.3 CONNECTION FORCE CALCULATION:

$$S_h = \min\left(\frac{d}{2}\ , 3b_{bf}\right) = 16\ \text{cm}$$

$$L_h = L - 2Sh = \ 11.68\ \text{m}$$

$$Wu = 1.2D + L + 0.2s = 0.506\ \text{t/m}$$

9.4 BEAM SPECIFICATION:

$$b_f(\text{cm}) = 25\ , t_f(\text{cm}) = 1\ , h_w(\text{cm}) = \ 30\ , t_w(\text{cm}) = 0.8$$

$$Z = 955\ \ \text{cm3}$$

$$M_p = Z * Fy = \ \ 955 * 2400 = 2292000\ \ \text{Kg.cm}$$

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

$$M_{pr} = C_{pr}R_yM_p = 1.2 \times 1.15 \times 2292000 = 3162960\ \text{kg.cm} = 31.62\ \text{t.m}$$

$$R_y = 1.15$$

$$V^H{}_{gravity} = \frac{W_uL_h}{2} = 2.96\ \text{ton}$$

$$V_{pr} = \frac{2M_{pr}}{L_h} = 5.41\ \text{ton}$$

$$M_u = M_{pr} + \frac{W_uSh^2}{2} = 31.62\ \ \text{ton.m}$$

9.5 END PLATE THICKNESS CALCULATION:

Mu=31.62 ton.m

$$\emptyset_d = 1.0$$

$$F_{yp} = 2400\ \text{kg/cm2}$$

$$Y_P = \frac{b_p}{2}[h_1\left(\frac{1}{P_{fi}} + \frac{1}{S}\right) + h0\left(\frac{1}{P_{fo}}\right) - \frac{1}{2}] + \frac{2}{g}[h1(P_{fi} + S)]$$

$$t_{p\ req} = \sqrt{\frac{1.11M_u}{\varphi_d F_{yp} Y_p}} = 2.5\ \text{cm}$$

10.0 PURLIN DESIGN

10.1 SOIL PRESSURE

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.

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 :

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

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

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

$$\text{middle of span}: \ M_y = \frac{w.L^2}{8} = \frac{76 \times 1.0 \times 6^2}{8} = 342 \text{ kg.m}$$

$$\text{middle of span}: \ M_x = \frac{w.L^2}{360} = \frac{14.5 \times 1 \times 6^2}{360} = 1.45\text{kg.m}$$

$$f_b = \frac{M_y}{s_y} + 2\frac{M_x}{s_x} = \frac{342 \times 100}{42.305} + 2\frac{1.45 \times 100}{7.024} = 808.42 + 20.65 = 829 < 1440 \text{ ok}$$

$$\text{moment on sagrod support}: \ M_y = \frac{w.L^2}{9} = \frac{76 \times 1.0 \times 6^2}{9} = 304 \text{ kg.m}$$

$$\text{moment on sagrod support}: \ M_x = \frac{w.L^2}{90} = \frac{14.5 \times 1.0 \times 6^2}{90} = 5.8 \text{ kg.m}$$

$$f_b = \frac{M_y}{s_y} + 2\frac{M_x}{s_x} = \frac{304 \times 100}{42.305} + 2\frac{5.8 \times 100}{7.024} = 498.99 + 114.46 = 884 < \ 1440 \quad \text{ok}$$

11.13.2.Un deformed shape CONTROL:

$$\text{dead + live loads}: \ \Delta = \frac{5 \times q \times L^4}{384 \times E \times I} = \frac{5 \times 0.76 \times 1.0 \times 600^4}{384 \times 2.04 \times 10^6 \times 1350} = 0.46\text{cm} \ < \ \frac{L}{240} = 2.0\text{cm}$$

$$\text{for live loads}: \ \Delta = \frac{5 \times q \times L^4}{384 \times E \times I} = \frac{5 \times 0.5 \times 1.0 \times 600^4}{384 \times 2.04 \times 10^6 \times 1350} = 0.31\text{cm} \ < \ \frac{L}{360} = 1.3 \text{ cm}$$

11.0 FOUNDATION DESIGN

11.1 SOIL PRESSURE

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.

11.2 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.

$f'_c = 30\text{Mpa}$ $\qquad\qquad$ $f_y = 400\text{Ma}$

11.3 FOUNDATION DESIGN CONTROL

10.3.1 CHECK OF STRESS FOR FOUNDATION

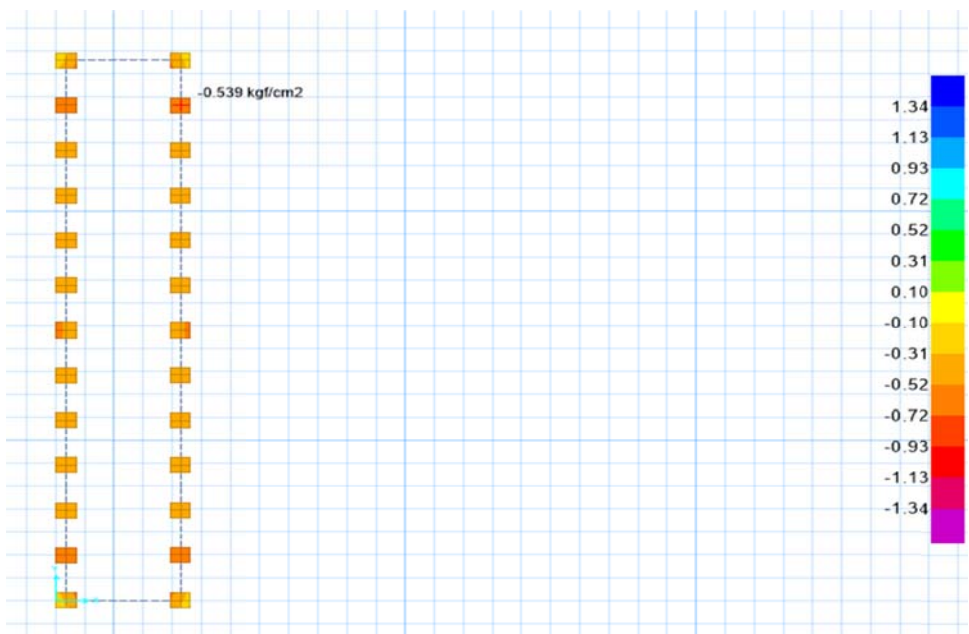


Figure 20 : Check of Stress for Foundation (kg/cm2)

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

$q_n = 0.539\text{kg/cm}^2 \ < \ 2 \ \text{kg/cm}^2 \ \ \text{ok}$

10.3.2 REINFORCING CONTROL

Minimum rebar for foundation:

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

$$A_{smin} = \frac{1}{2}0.0018\ bh = \frac{1}{2}0.0018 \times 100 \times 60 = 5.4 \quad cm^2/m$$

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

Slab Strip Design - Layers A, B - Top and Bottom Reinforcement (Enveloping Flexural) - Additional to 16 @ 20 cm (Top), 16 @ 20 cm (Bot)

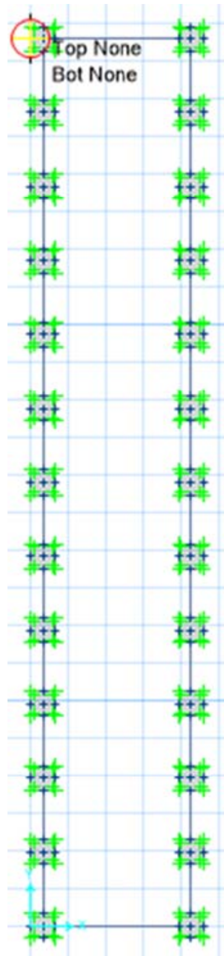


Figure 21 : additional reinforement

Punching Shear Capacity Ratios/Shear Reinforcement

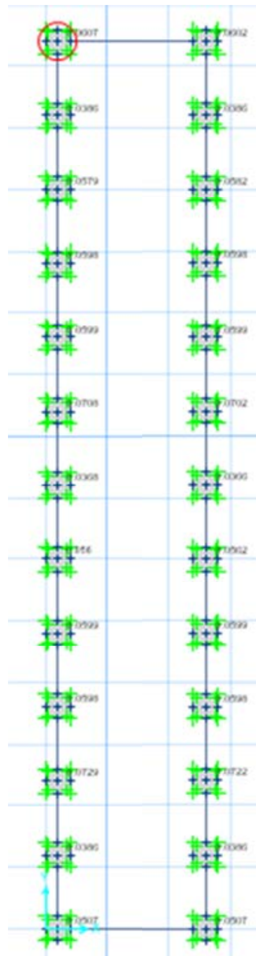


Figure 22: punch shear control