Client: ***Hubert Nguyen***

Project: ***1-st SINGLE HOUSE USING SHIPPING CONTAINERS***

DESIGN REPORT OF FEA AND STEEL STRUCTURAL DESIGN

FOR 1-st SINGLE HOUSE USING SHIPPING CONTAINERS ACCORDING ASCE 7-10, AISC 360-16, CBC 2014

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2019

TABLE OF CONTENTS

1. DESCRIPTION OF THE CONSTRUCTION

1.1. Overview

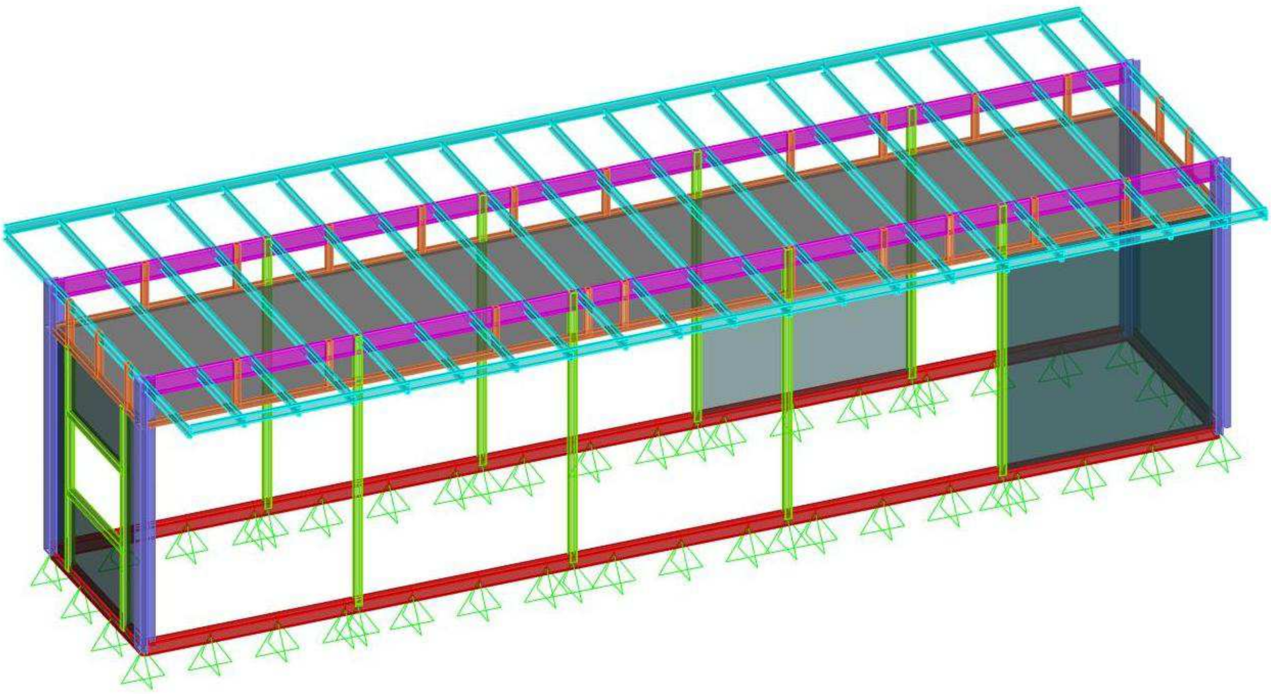
The report presents the results of FEA and steel structures design for for 1-st single house using shipping containers .

1.2. Structural System

The structural system of building consists of blocked shipping containers strengthened system of steel giders/beams/joists and posts according architectural design and requirements for strength, rigidity, buckling and deflections limits .
Additionally calculated option for lift and "dumb waiter" using steel moment frames.

The structural system of the building withstands the all loads , including snow, wind and seismic load.

Load bearing system has the rigidity, stability and strength allowing to transfer gravity and lateral up to the foundation ground consistently and safely.



General view of the 3D model

2. DESIGN CRITERIA

2.1. References, Main Regulations and Standards

This document is prepared in accordance with the applicable regulations, standards and recommendations mentioned below:

1. ASCE 7 American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures
2. CBC 2014, California building code
3. AISC 360-16, Specification for Structural Steel Buildings
4. ASTM A36, Standard Specification for Carbon Structural Steel

2.2. Design Basics

Structural calculations and design details will be established in consideration of the clients demands, in accordance with applicable regulations and standards, and in further compliance with other appropriate disciplines.

In order to ensure adequate reliability, all structures shall be analyzed for ultimate limit state and serviceability limit state. The analyses for ultimate limit state and serviceability limit state shall take into consideration the anticipated loads both during construction work and after its completion.

2.3. Computer Software

The following computer software will be employed in the construction analysis and design:

1. SAP2000 Computers and Structures, Inc. Berkeley, California USA, version 20.2.0
2. Tedds 2017

2.4. Material characteristics

1. Square and rectangular tubing - structural steel ASTM A572Gr50

2. I-beams "W" and C-channels - structural steel ASTM A992Fy50

3. Containers structural members - structural steel ASTM A36

Elasticity Modulus: Es = 29000 Ksi

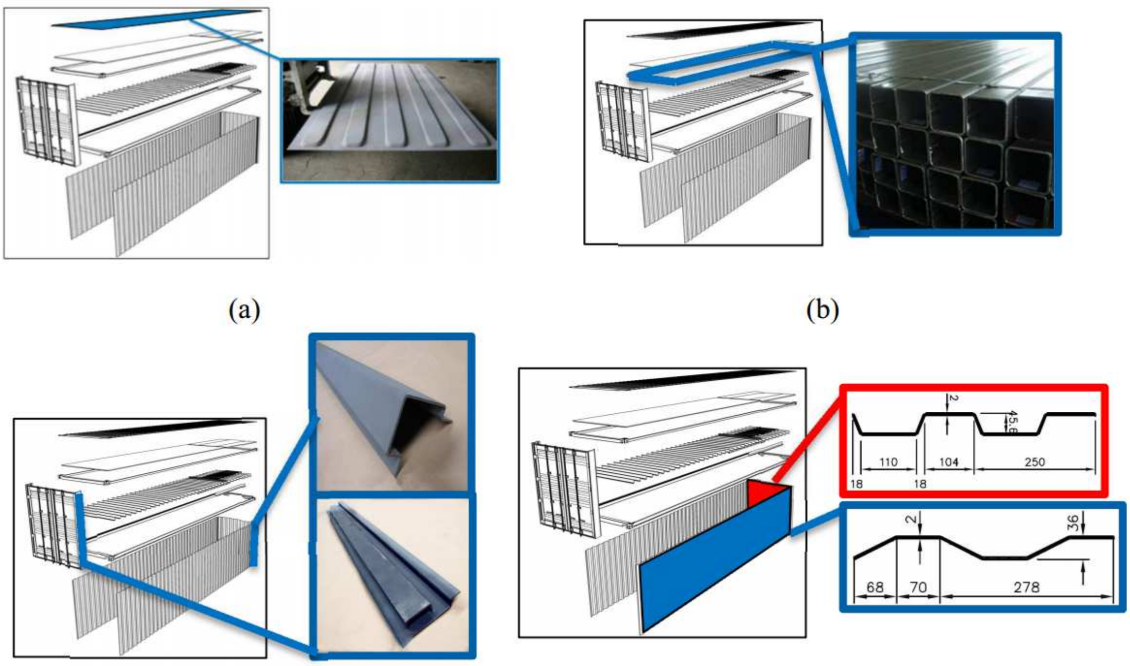
3. DESIGN REQUIREMENTS, LOADS AND LOAD COMBINATIONS

3.1. Introduction

This section describes the characteristic loads, design loads, limits and load combinations employed in structural analysis.

CONTAINER GEOMETRY AND MAIN COMPONENTS

40 foot high-cube:



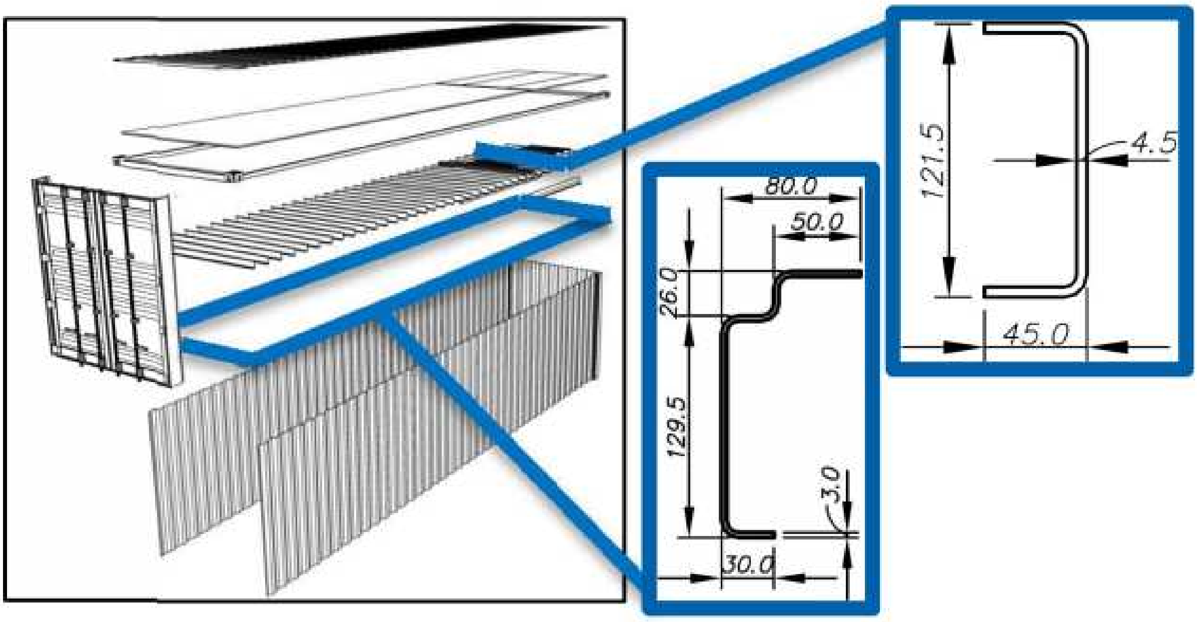
(a)

(b)

(c)

(d)

(a) Roof of shipping container, (b) top rail of shipping container , (c) end posts of shipping container and (d) wall panel section (mm) of 40 foot and 8 foot wall panel.



Floor diaphragm showing bottom rail and cold-formed channel in mm

The roof steel is constructed from cambered, die-stamped, corrugated sheets that are 2 mm thick.

The corrugated wall thickness within 4 feet from end posts is 2 mm and 1.6 mm beyond. The floor diaphragm is composed of 1⅛" marine-grade plywood over cold-formed steel channels spaced at 12" on center and welded to the bottom rail. The corrugated walls are continuously welded to bottom rail within the flat portion of the 50 mm flange.

3.2. Load Definitions

Nominal loads: loads not increased by the safety factor.

Factored loads: loads increased by the safety factor.

Factored loads = nominal loads x safety coefficients.

3.3. Dead loads

Dead loads include self weight of steel structures and weight of mounted equipment .

Net weight assigned automatically by software complex on the basis that density of steel = 490 lb/ft^3.

DL: Roof

Potential Metal Roof	1.5 Psf
Top (Ceiling Steel Plate)	5.1 Psf
5/8" Gypsum Board	2.8 Psf
Mechical/Electrical	2 Psf
Misc Framing	1.6 Psf
TOTAL:	13 Psf

Floor

5/8" Hardwood Flooring (Finish)	2.5 Psf
1 1/4" Marine hardwood flooring	4 Psf
C-Channel Framing	5 Psf
Mechical/Electrical	2 Psf
Misc	1.5 Psf
TOTAL:	15 Psf

Side Wall

Top Rail	3.5 Plf
Perimeter Wall (8'-8")	33.2 Plf
Bottom Rail	3.8 Plf
TOTAL:	40.5 Plf

3.4. Live load

Roof	20 Psf
Floor	15psf + 40psf = 55 Psf

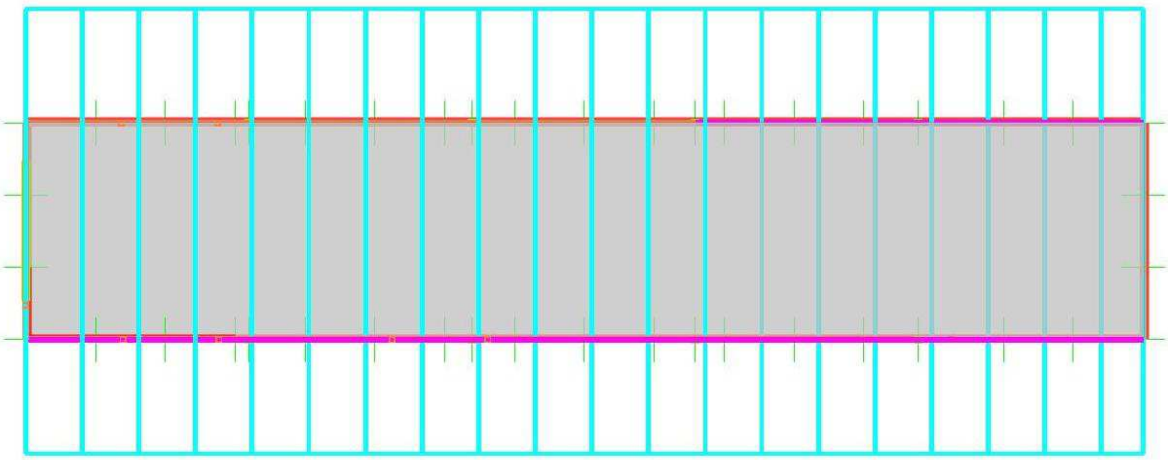
3.5. Snow load

Considered in 150 psf for potentially most heavy snow loads

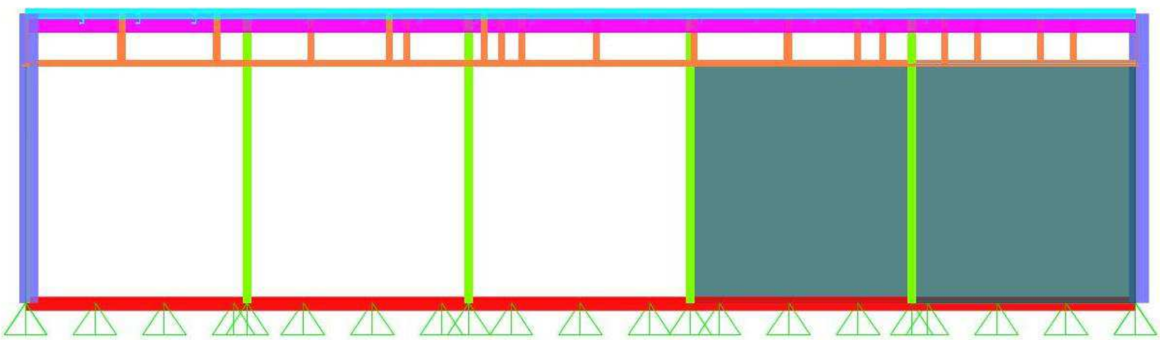
3.6. Wind Loads

Properties of bulding:

Plan



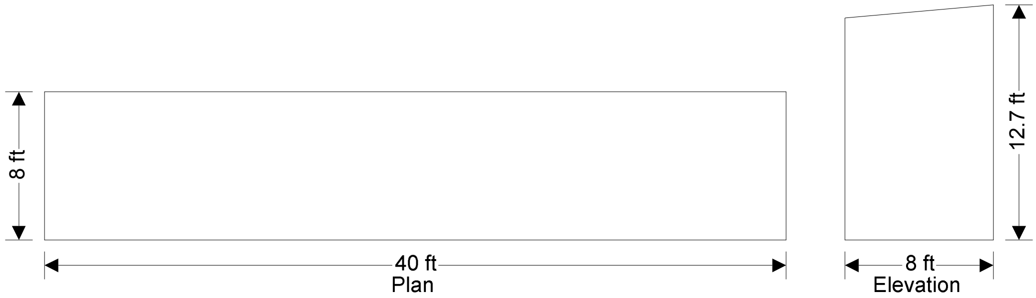
Elevation



WIND LOADING (ASCE7-10)

In accordance with ASCE7-10 incorporating Errata No. 1 and Errata No. 2

Using the directional design method



Building data

Type of roof;	Monoslope
Length of building;	b = **40.00** ft
Width of building;	d = **8.00** ft
Height to eaves;	H = **12.00** ft
Pitch of roof;	α_0 = **5.0** deg
Mean height;	h = **12.00** ft

General wind load requirements

Basic wind speed;	V = **115.0** mph
Risk category;	II
Velocity pressure exponent coeff (Table 26.6-1);	K_d = **0.85**
Exposure category (cl.26.7.3);	B
Enclosure classification (cl.26.10);	Partially enclosed buildings
Internal pressure coef +ve (Table 26.11-1);	GC_{pi_p} = **0.55**
Internal pressure coef –ve (Table 26.11-1);	GC_{pi_n} = **-0.55**

Gust effect factor for rigid structures

Terrain exposure constants (Table 26.9-1)	
Integral length scale factor;	l = **320.0** ft
Turbulence intensity factor;	c = **0.30**
Minimum equivalent height;	z_{min} = **30.0** ft
Peak factor for background response;	g_Q = **3.400**
Peak factor for wind response;	g_v = **3.400**
Integral length scale power law exponent;	$\bar{\varepsilon}$ = **0.333**
Equivalent height of the structure;	$\bar{z} = \max(0.6 \cdot h, z_{min})$ = **30.00** ft
Intensity of turbulence (Eqn. 26.9-7);	$I_{\bar{z}} = c \cdot (33\text{ ft} / \bar{z})^{1/6}$ = **0.30**
Integral length scale of turbulence (Eqn. 26.9-9);	$L_{\bar{z}} = l \cdot (\bar{z} / 33\text{ ft})^{\bar{\varepsilon}}$ = **310.00** ft
Background response (Eqn. 26.9-8);	$Q = O(1 / (1 + 0.63 \cdot ((\min(B, L) + h) / L_{\bar{z}})^{0.63}))$ = **0.948**
Gust effect factor (Eqn. 26.9-6);	$G = G_f = 0.925 \cdot (1 + 1.7 \cdot g_Q \cdot I_{\bar{z}} \cdot Q) / (1 + 1.7 \cdot g_v \cdot I_{\bar{z}})$ = **0.89**

Topography

Topography factor not significant;	K_{zt} = 1.0
Velocity pressure equation;	$q = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot 1\text{psf/mph}^2$;

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
12.00;	0.57;	16.40;
12.70;	0.57;	16.40;

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **16.40** psf

Pressures and forces

Net pressure; $p = q \cdot G_f \cdot C_{pe} - q_i \cdot GC_{pi}$;

Net force; $F_w = p \cdot A_{ref}$;

Roof load case 1 - Wind 0, GC_{pi} 0.55, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	12.00	-1.18	16.40	-26.32	240.92	-6.34
B (-ve)	12.00	-0.70	16.40	-19.29	80.31	-1.55

Total vertical net force; $F_{w,v}$ = **-7.86** kips

Total horizontal net force; $F_{w,h}$ = **-0.69** kips

Walls load case 1 - Wind 0, GC_{pi} 0.55, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	16.40	2.72	480.00	1.30
B	12.00	-0.50	16.40	-16.36	508.00	-8.31
C	12.00	-0.70	16.40	-19.29	98.80	-1.91
D	12.00	-0.70	16.40	-19.29	98.80	-1.91

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \cdot H$ = **480.00** ft^2

Projected vertical area of roof; $A_{vert\_r\_0} = b \cdot d \cdot \tan(\alpha_0)$ = **28.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \cdot A_{vert\_w\_0} + p_{min\_r} \cdot A_{vert\_r\_0}$ = **7.9** kips

Leeward net force; $F_l = F_{w,wB}$ = **-8.3** kips

Windward net force; $F_w = F_{w,wA}$ = **1.3** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **8.9** kips

Roof load case 2 - Wind 0, GC_{pi} -0.55, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	12.00	-0.18	16.40	6.38	240.92	1.54
B (+ve)	12.00	-0.18	16.40	6.38	80.31	0.51

Total vertical net force; $F_{w,v}$ = **2.04** kips

Total horizontal net force; $F_{w,h}$ = **0.18** kips

Walls load case 2 - Wind 0, GC_{pi} -0.55, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	16.40	20.76	480.00	9.96
B	12.00	-0.50	16.40	1.69	508.00	0.86
C	12.00	-0.70	16.40	-1.25	98.80	-0.12

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
D	12.00	-0.70	16.40	-1.25	98.80	-0.12

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \cdot H =$ **480.00** ft^2

Projected vertical area of roof; $A_{vert\_r\_0} = b \cdot d \cdot \tan(\alpha_0) =$ **28.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \cdot A_{vert\_w\_0} + p_{min\_r} \cdot A_{vert\_r\_0} =$ **7.9** kips

Leeward net force; $F_l = F_{w,wB} =$ **0.9** kips

Windward net force; $F_w = F_{w,wA} =$ **10.0** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **9.3** kips

Roof load case 3 - Wind 90, GC_{pi} 0.55, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	12.00	-0.90	16.40	-22.23	48.18	-1.07
B (-ve)	12.00	-0.90	16.40	-22.23	48.18	-1.07
C (-ve)	12.00	-0.50	16.40	-16.36	96.37	-1.58
D (-ve)	12.00	-0.30	16.40	-13.42	128.49	-1.72

Total vertical net force; $F_{w,v} =$ **-5.42** kips

Total horizontal net force; $F_{w,h} =$ **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.55, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.70	0.80	16.40	2.72	98.80	0.27
B	12.00	-0.20	16.40	-11.96	98.80	-1.18
C	12.00	-0.70	16.40	-19.29	480.00	-9.26
D	12.00	-0.70	16.40	-19.29	508.00	-9.80

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \cdot (H + d \cdot \tan(\alpha_0) / 2) =$ **98.80** ft^2

Projected vertical area of roof; $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \cdot A_{vert\_w\_90} + p_{min\_r} \cdot A_{vert\_r\_90} =$ **1.6** kips

Leeward net force; $F_l = F_{w,wB} =$ **-1.2** kips

Windward net force; $F_w = F_{w,wA} =$ **0.3** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **1.6** kips

Roof load case 4 - Wind 90, GC_{pi} -0.55, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	12.00	-0.18	16.40	6.38	48.18	0.31
B (+ve)	12.00	-0.18	16.40	6.38	48.18	0.31
C (+ve)	12.00	-0.18	16.40	6.38	96.37	0.61
D (+ve)	12.00	-0.18	16.40	6.38	128.49	0.82

Total vertical net force; $F_{w,v} =$ **2.04** kips

Total horizontal net force; $F_{w,h} =$ **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.55, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.70	0.80	16.40	20.76	98.80	2.05
B	12.00	-0.20	16.40	6.09	98.80	0.60
C	12.00	-0.70	16.40	-1.25	480.00	-0.60
D	12.00	-0.70	16.40	-1.25	508.00	-0.63

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \bullet (H + d \bullet \tan(\alpha_0) / 2) = \mathbf{98.80}$ ft^2

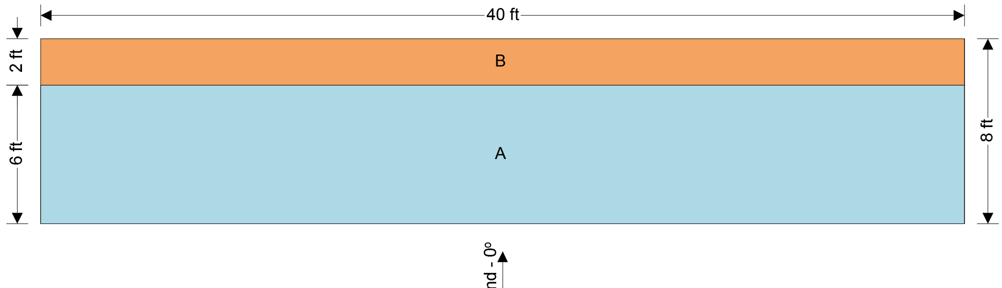
Projected vertical area of roof; $A_{vert\_r\_90} = \mathbf{0.00}$ ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \bullet A_{vert\_w\_90} + p_{min\_r} \bullet A_{vert\_r\_90} = \mathbf{1.6}$ kips

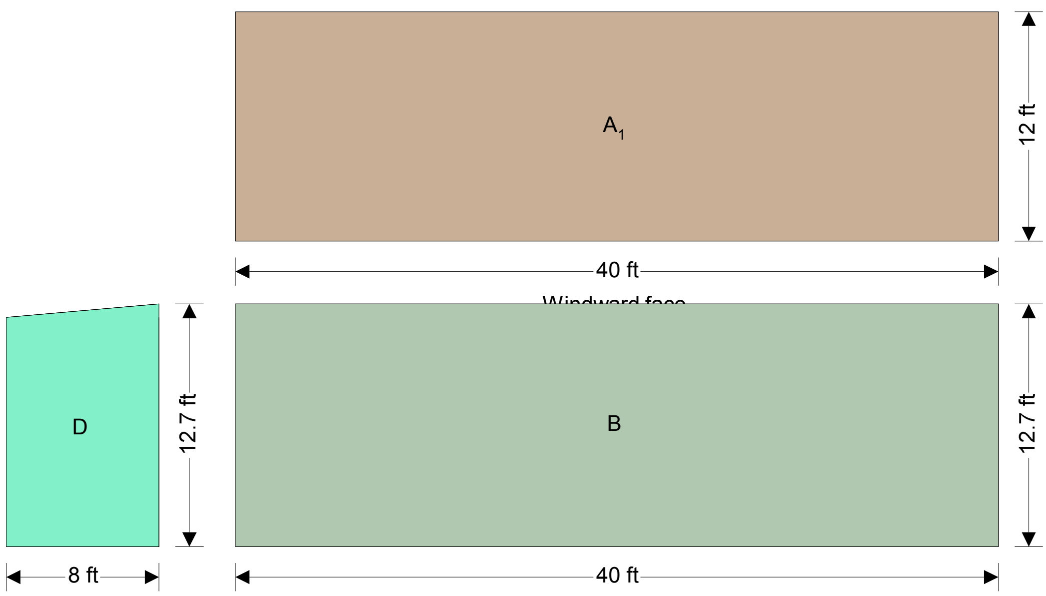
Leeward net force; $F_l = F_{w,wB} = \mathbf{0.6}$ kips

Windward net force; $F_w = F_{w,wA} = \mathbf{2.1}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{1.6}$ kips

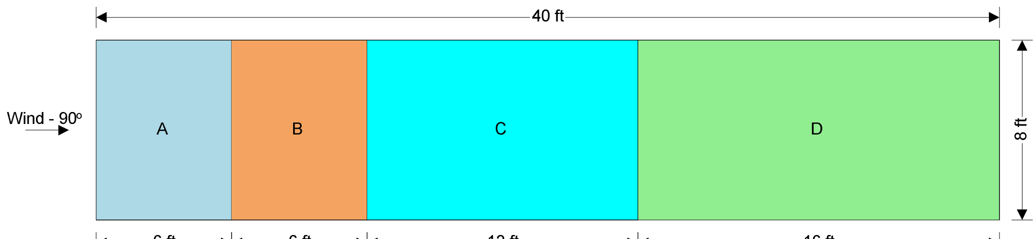


Plan view - Monoslope roof

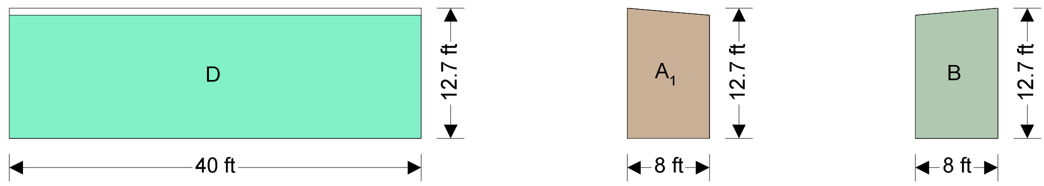


Side face

Leeward face



Plan view - Monoslope roof



Side face Windward face Leeward face

3.7. Seismic loads

<u>SEISMIC FORCES (ASCE 7-10)</u>

Site parameters

Site class; D

Mapped acceleration parameters (Section 11.4.1)

at short period; S_S = **1.5**

at 1 sec period; S_1 = **0.6**

Site coefficientat short period (Table 11.4-1); F_a = **1.000**

at 1 sec period (Table 11.4-2); F_v = **1.500**

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \cdot S_S =$ **1.500**

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \cdot S_1 =$ **0.900**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \cdot S_{MS} =$ **1.000**

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \cdot S_{M1} =$ **0.600**

Seismic design category

Risk category (Table 1.5-1); II

Seismic design category based on short period response acceleration (Table 11.6-1)

D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

D

Seismic design category; D

Approximate fundamental period

Height above base to highest level of building; h_n = **12** ft

From Table 12.8-2:

Structure type; All other systems

Building period parameter C_t; C_t = **0.02**

Building period parameter x; x = **0.75**

Approximate fundamental period (Eq 12.8-7); $T_a = C_t \cdot (h_n)^x \cdot 1\text{sec} / (1\text{ft})^x =$ **0.129** sec

Building fundamental period (Sect 12.8.2); $T = T_a =$ **0.129** sec

Long-period transition period; T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1); H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED

1. Steel system not specially detailed for seismic resistance,

ex

Response modification factor (Table 12.2-1); R = **3**

Seismic importance factor (Table 1.5-2); I_e = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2); $C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.3333**

Maximum (Eq 12.8-3); $C_{s\_max} = S_{D1} / (T \cdot (R / I_e))$ = **1.5510**

Minimum:

Eq 12.8-5; $C_{s\_min1} = \max(0.044 \cdot S_{DS} \cdot I_e, 0.01)$ = **0.0440**

Eq 12.8-6 (where S_1 >= 0.6); $C_{s\_min2} = (0.5 \cdot S_1) / (R / I_e)$ = **0.1000**

C_{s_min} = **0.1000**

Seismic response coefficient; C_s = **0.3333**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure; W = **17.2** kips

Seismic response coefficient; C_s = **0.3333**

Seismic base shear (Eq 12.8-1); $V = C_s \cdot W$ = **5.7** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12); $C_{vx} = w_x \cdot h_x^k / \Sigma(w_i \cdot h_i^k)$

Lateral force induced at level i (Eq 12.8-11); $F_x = C_{vx} \cdot V$

Minimum diaphragm forces (Section 12.10.1.1)

Calculated min. diaphragm force (Eq 12.10-1); $F_{px} = \Sigma F_i \cdot w_{px} / \Sigma w_i$,(i=x to n)

$F_{pxmin} = 0.2 \cdot S_{DS} \cdot I_e \cdot w_{px}$

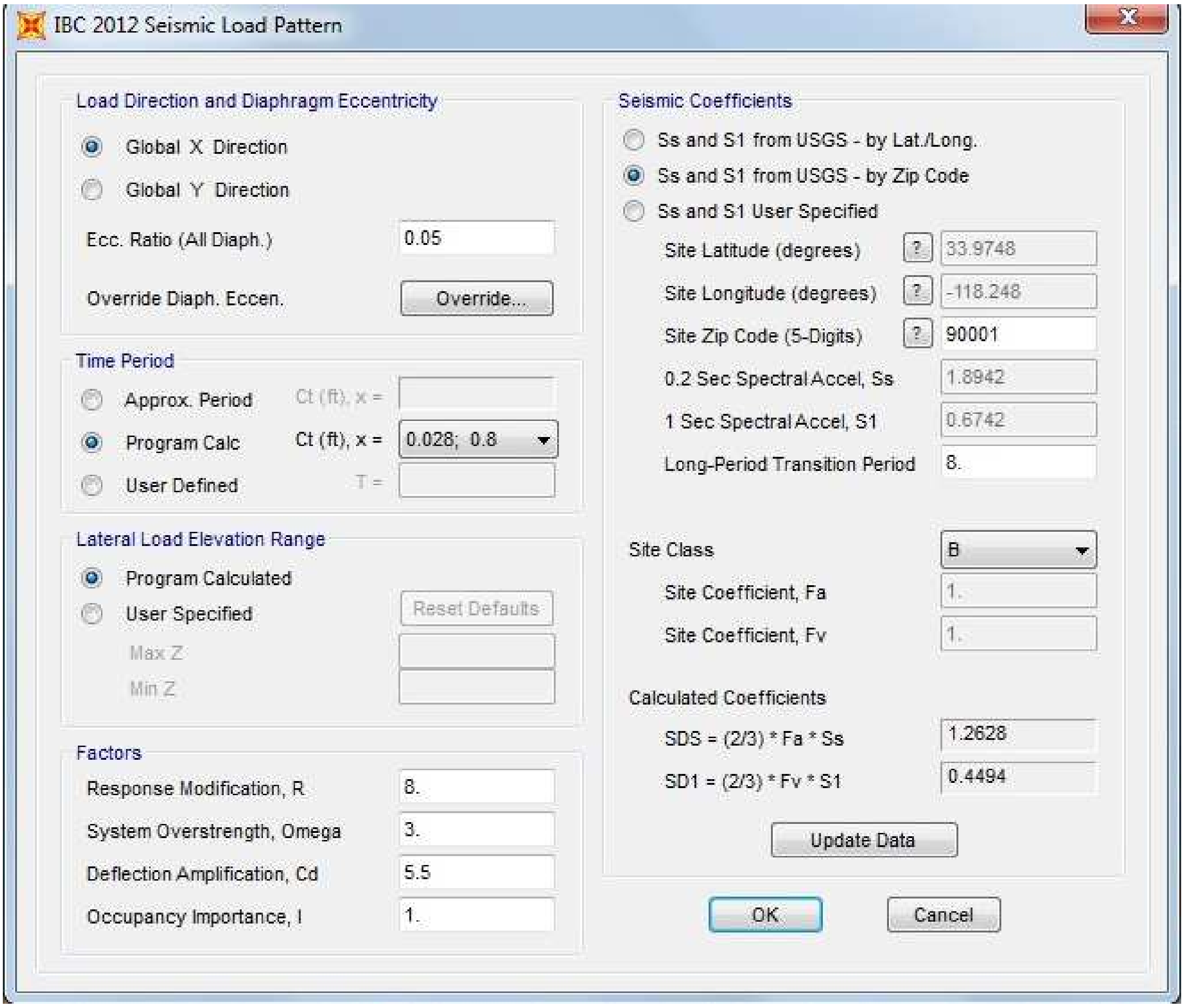
$F_{pxmax} = 0.4 \cdot S_{DS} \cdot I_e \cdot w_{px}$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x;	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	0.0;	16.0;	1.00;	0.000;	0.0;	30.1	10.0
2	12.0;	1.2;	1.00;	1.000;	5.7;	30.1	12.0

;

Alternative seismic loads calculations using SAP2000



3.8. Load Combinations

D - dead loads

L - live load

SN - snow load

WX+ - wind positive load in direction X

WX- - wind negative load in direction X

WY+ - wind positive load in direction Y

WY- - wind negative load in direction Y

EX - seismic load along X

EY- seismic load along Y

Included in the calculation load combinations:

UDSTL1 = 1.4D

UDSTL2 = 1.2D+1.6L

UDSTL3 = 1.2D+L

UDSTL4= 1.2D+L+(WX+)

UDSTL5 =1.2D+L+(WX-)

UDSTL6 =1.2D+L+(WY+)

UDSTL7 =1.2D+L+(WY-)

UDSTL8 =1.2D+1.6SN+0.5(WX+)

UDSTL9 = 1.2D+1.6SN+0.5(WX-)

UDSTL10 = 1.2D+1.6SN+0.5(WY+)

UDSTL11 = 1.2D+1.6SN+0.5(WY-)

UDSTL12 = 0.9D+(WX+)

UDSTL13 = 0.9D+(WX-)

UDSTL14 = 0.9D+(WY+)

UDSTL15 = 0.9D+(WY-)

UDSTL16 = 1.3D+L+EX

UDSTL17 = 1.3D+L-EX

UDSTL18 =1.3D+L+EY

UDSTL19 = 1.3D+L-EY

UDSTL20 = 0.8D+EX

UDSTL21 = 0.8D-EX

UDSTL22 = 0.8D+EY

UDSTL23 =0.8D-EY

UDSTL24 = D

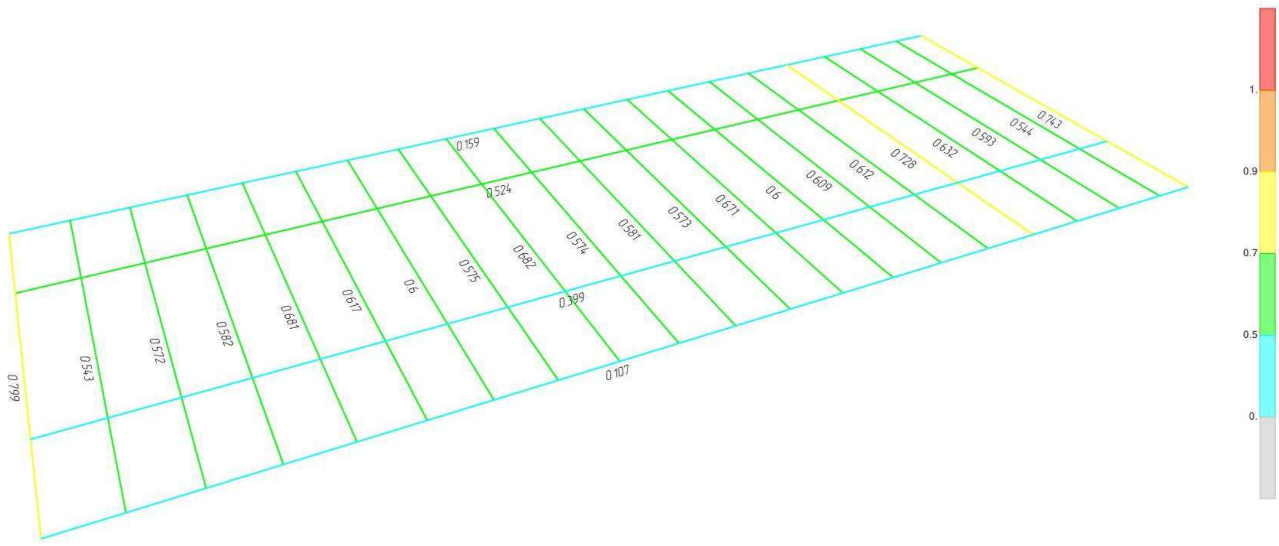
UDSTL25 = D+L

UDSTL26 = D+SN

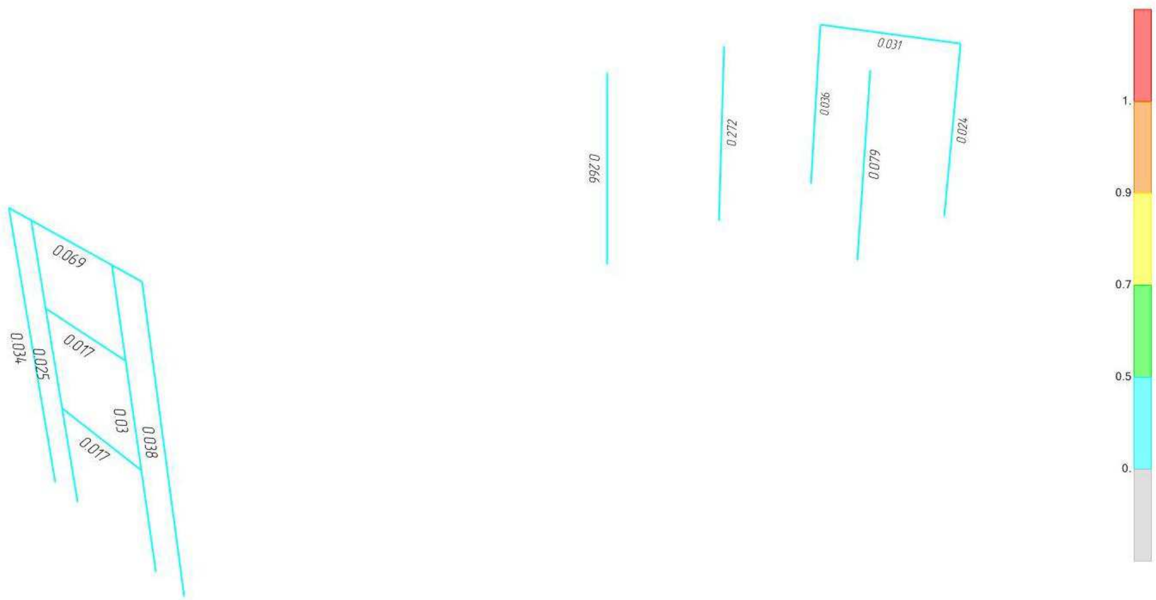
4. RESULTS OF ANALYSIS

4.1. Steel members analysis and check

Presented results of structural checks for steel structural members according AISC 360-16 design codes.



I-BEAMS AND C-CHANNELS (RATIO VALUES)



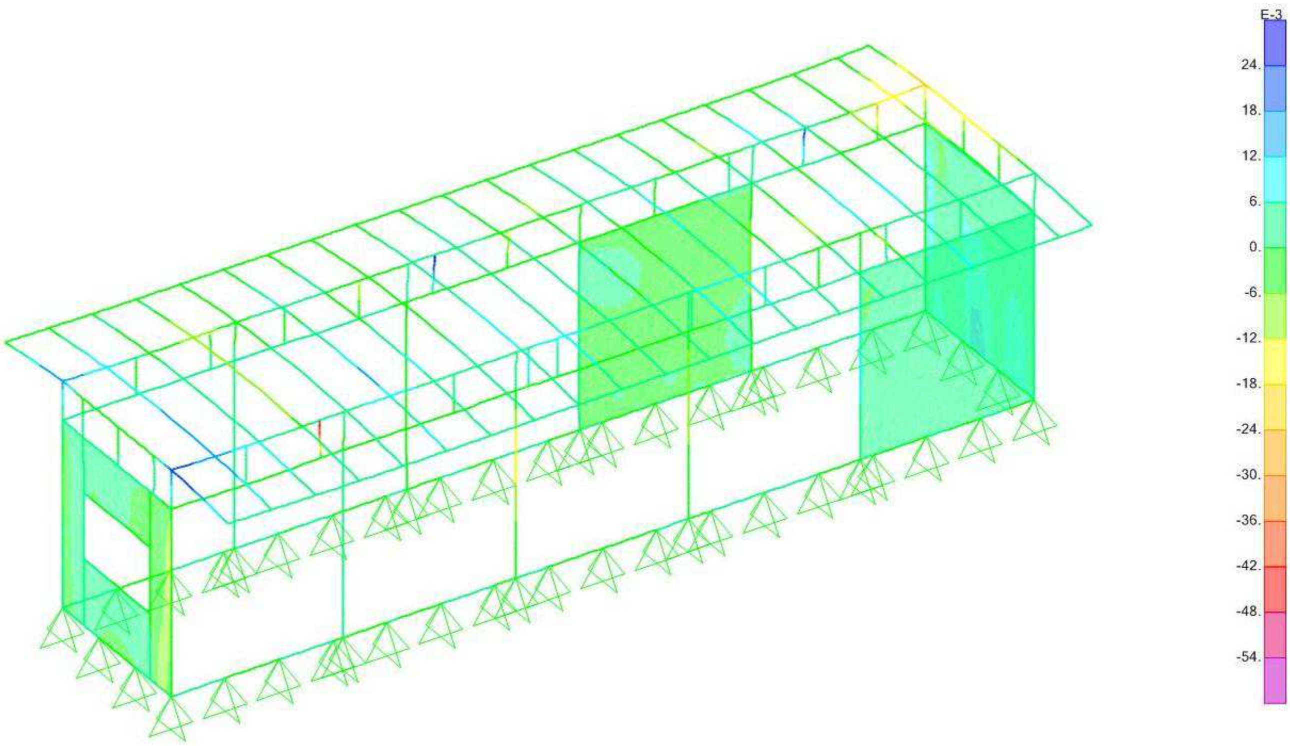
POSTS (RATIO VALUES)

4.2. Deflections analysis

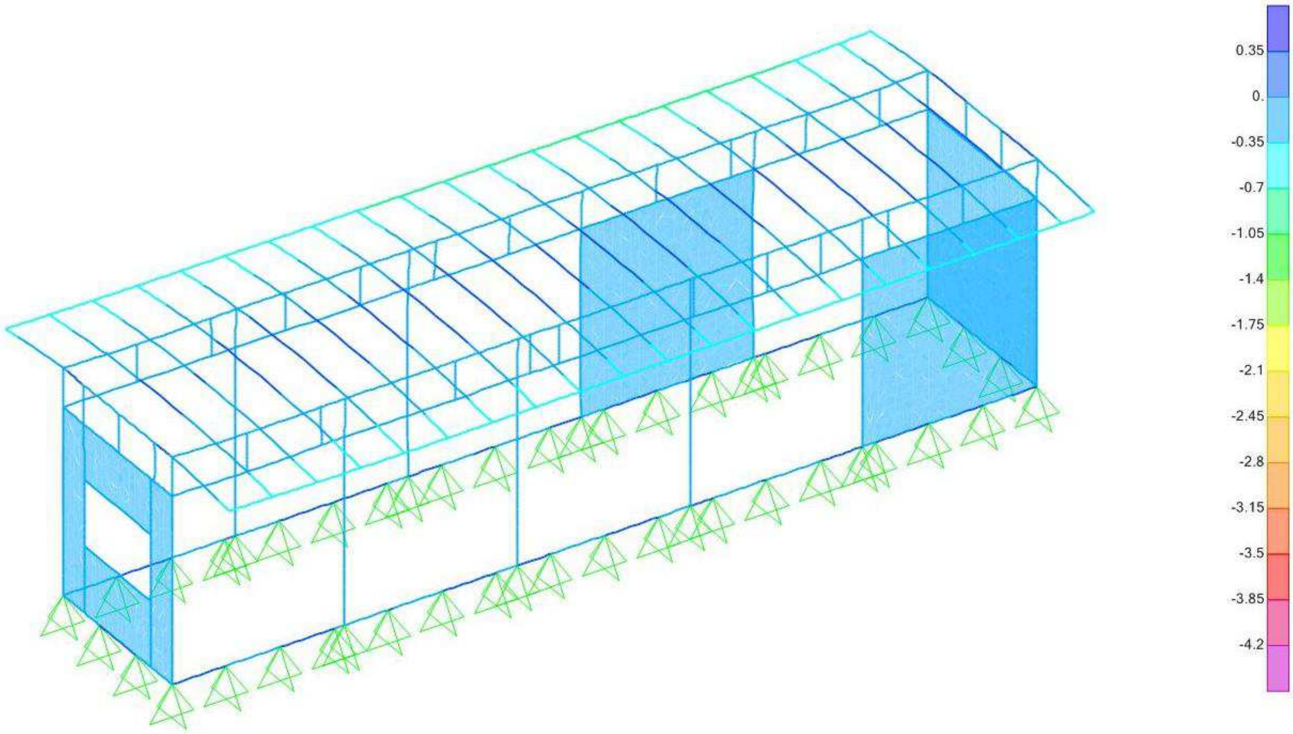
Considered combinations for deflections analysis:

Combination1 = 1.0D+1.0SN+1.0(WX+)

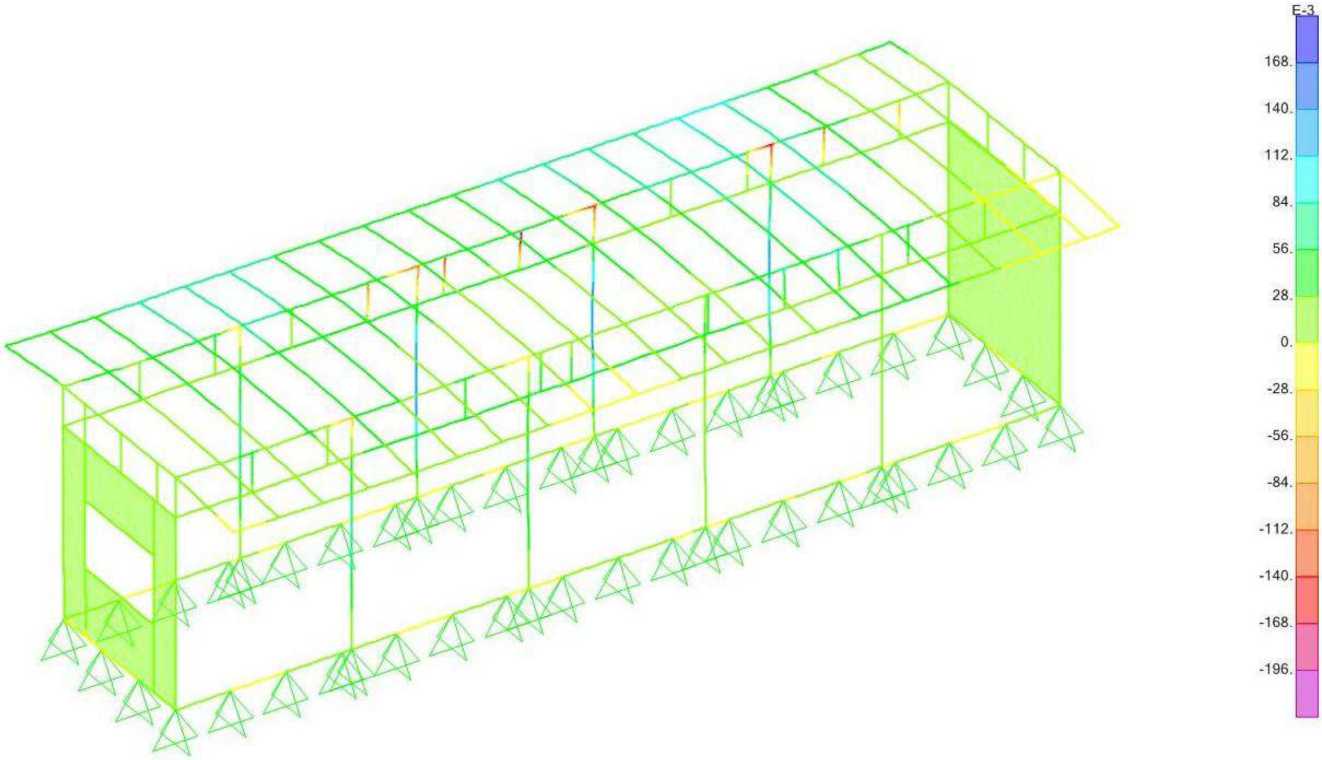
Combination2 = 1.0D+1.0SN+1.0(WX-)



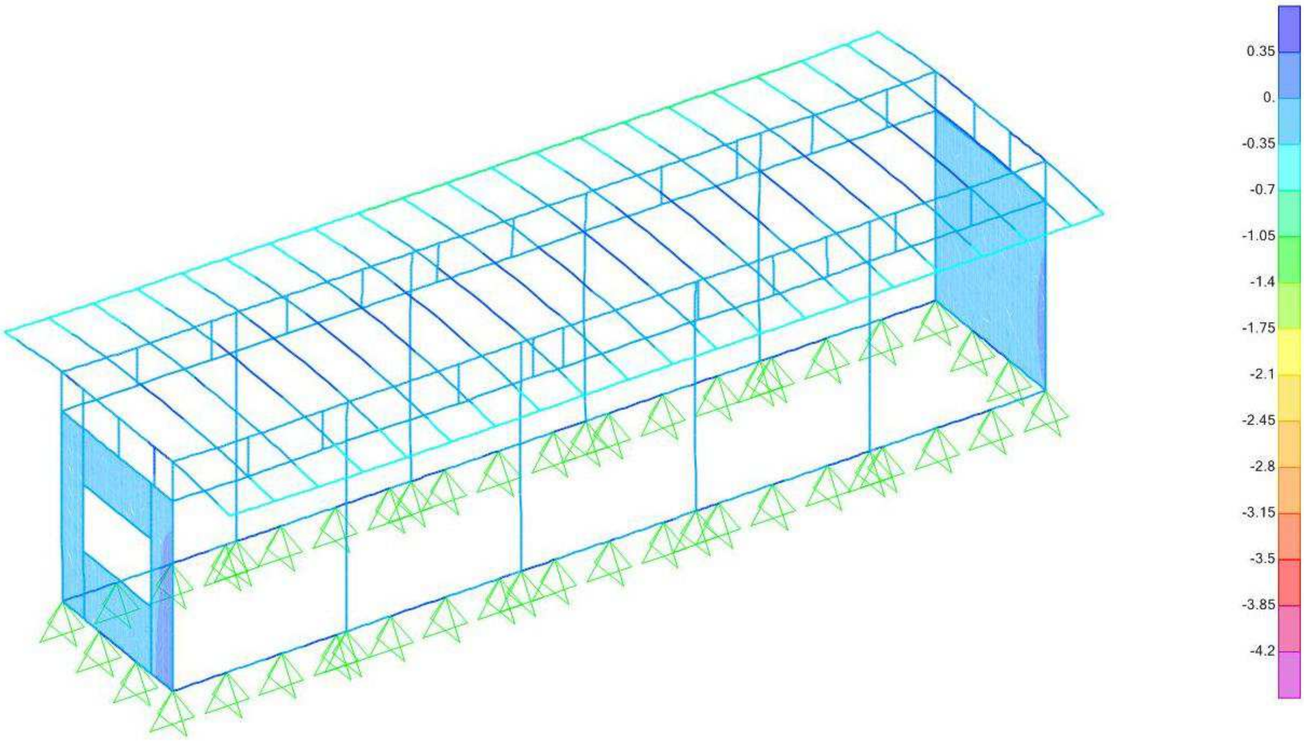
COMBINATION1 (DEFLECTION ALONG X, in)



COMBINATION1 (DEFLECTION ALONG Z,in)



COMBINATION2 (DEFLECTION ALONG Y, in)



COMBINATION2 (DEFLECTION ALONG Z,in)