Smith Monroe Gray
ENGINEERS, INC.

8625 SW CASCADE AVE., SUITE 600
BEAVERTON, OREGON 97008
PHONE 503.643.8595
FAX 503.643.8610

MAPLE VALLEY ASPHALT PLANT

B. STEWART | STRUCTURAL CALCULATIONS FOR | 04/10/19

LAKESIDE INDUSTRIES

18-183B | Rev. 1

CONTENTS:

4-17-19

JOSHUA W. CHRISTIANSEN
STATE OF WASHINGTON
51843
REGISTERED
PROFESSIONAL ENGINEER

DESIGN REFERENCES:

AISC STEEL CONSTRUCTION MANUAL, 14TH EDITION
AISC STEEL SEISMIC DESIGN MANUAL
ASCE 7-14 MINIMUM DESIGN LOADS FOR BUILDINGS
ACI 318-14 BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE
IBC 2015

PROJECT DESCRIPTION:

PROVIDE STRUCTURAL DESIGNS FOR MISCELLANEOUS ASPHALT PLANT STRUCTURES.

ASPHALT PLANT BUILDINGS:

- AGGREGATE STORAGE BUILDING (BUILDING #1 & #2)
- RECYCLED ASHPHALT CRUSHER ENCLOSURE (BUILDING #3)
- EQUIPMENT STORAGE BUILDING

NON-BUILDING STRUCTURES:

- ASPHALT STORAGE TANK SECONDARY CONTAINMENT
- SILO FOUNDATION (*RELOCATED STRUCTURE)
- VAPORIZER FOUNDATION
- LIQUID NATURAL GAS TANK FOUNDATION
- SOUND ATTENUATION WALL

<u>PROJECT LOCATION:</u> MAPLE VALLEY HIGHWAY (RENTON, KING COUNTY, WA)

DESIGN CRITERIA & LOADING:

CONTROLLING STANDARD: 2015 IBC & 2015 WASHINGTON STATE BUILDING CODE

RISK II STRUCTURES
- AGGREGATE STORAGE BUILDING (BUILDING #1 & #2)
- RECYCLED ASHPHALT CRUSHER ENCLOSURE (BUILDING #3)
- EQUIPMENT STORAGE BUILDING
- VAPORIZER FOUNDATION
- SOUND ATTENUATION WALL

RISK III STRUCTURES
- ASPHALT STORAGE TANK SECONDARY CONTAINMENT

CALCULATED AS RISK III IS CONSERVATIVE. RISK II ACTUAL

RISK IV STRUCTURES
- SILO FOUNDATION (*RELOCATED STRUCTURE)
- LIQUID NATURAL GAS TANK FOUNDATION

ASPHALT MATERIAL LOAD

ANGLE OF REPOSE = 37 DEG.
MATERIAL DENSITY = 105 PCF

LIVE LOAD DESIGN CRITERIA

ROOF LIVE LOAD = 20 PSF
EQUIPMENT BUILDING LIVE LOAD = 125 PSF (LIGHT STORAGE)

SNOW

GROUND SNOW LOAD, $p_g := 20\ psf$

EXPOSURE FACTOR, $C_e := 1.0$
THERMAL FACTOR, $C_t := 1.0$

SNOW IMPORTANCE, $I_{sII} := 1.00$ RISK II STRUCTURES
$I_{sIII} := 1.10$ RISK III STRUCTURES
$I_{sIV} := 1.20$ RISK IV STRUCTURES

FLAT ROOF SNOW LOAD, $p_f := 0.7 \cdot C_e \cdot C_t \cdot I_{sII} \cdot p_g = 14\ psf$ RISK II STRUCTURES
$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_{sIII} \cdot p_g = 15.4\ psf$ RISK III STRUCTURES
$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_{sIV} \cdot p_g = 16.8\ psf$ RISK IV STRUCTURES

MINIMUM SNOW LOAD PER CITY REQUIREMENTS = 25 PSF

DESIGN CRITERIA & LOADING:

WIND

EXPOSURE = C

$V_{ult\_II} := 110\ mph$ RISK II STRUCTURES
$V_{ult\_III} := 115\ mph$ RISK III STRUCTURES
$V_{ult\_IV} := 115\ mph$ RISK IV STRUCTURES

INTERNAL PRESSURE COEFFICIENT, $GC_{pi} := 0.55$

COMPONENT & CLADDING DESIGN WIND PRESSURE, $q_z := 26\ psf$
RISK IV STRUCTURE, EXPOSURE C

SEISMIC

SITE DESIGN PARAMETERS:
SITE CLASS D
SEISMIC DESIGN CATEGORY D
ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE

$S_s := 1.325\ g$
$S_1 := 0.495\ g$
$S_{DS} := 0.883\ g$
$S_{D1} := 0.496\ g$

STRUCTURE	I_e	LFRS	R	OVERSTRENGTH	Cs	V
ASPHALT STORAGE (BUILDING #1)	1.00	Ordinary Concentric Braced Frame	3.25	2	0.27	0.27*W
ASPHALT STORAGE (BUILDING #2)	1.00	Ordinary Concentric Braced Frame (Longitudinal)	3.25	2	0.27	0.27*W
		Ordinary Moment Frame (Transverse)	3.5	3	0.25	0.25*W
RAP CRUSHER & STORAGE (BUILDING #3)	1.00	Ordinary Concentric Braced Frame (Longitudinal)	3.25	2	0.27	0.27*W
		Ordinary Moment Frame (Transverse)	3.5	3	0.25	0.25*W
BUILDING #1, #2, #3 BIN WALLS	1.00	Special Reinforced Concrete Shear Walls	5	2.5	0.18	0.18*W
EQUIPMENT STORAGE BUILDING	1.00	Ordinary Concentric Braced Frame (Longitudinal)	3.25	2	0.27	0.27*W
		Ordinary Moment Frame (Transverse)	3.25	2	0.27	0.27*W

NON-BUILDING STRUCTURE	I_e	LFRS	R	Cs	V
SILO 2ND CONTAINMENT STRUCTURE	1.00	FLAT BOTTOM GROUND SUPPORTED TANK	3	0.29	0.29*W
SILO FOUNDATION (RELOCATION)	1.00	ORDINARY MOMENT FRAME	3.5	0.25	0.25*W
VAPORIZER FOUNDATION	1.00	FLAT BOTTOM GROUND SUPPORTED TANK	3	0.29	0.29*W
LNG FOUNDATION	1.50	FLAT BOTTOM GROUND SUPPORTED TANK	3	0.44	0.44*W
SOUND ATTENTUATION WALLS	1.00	MASS CANTILEVER STRUCTURE	3	0.29	0.29*W

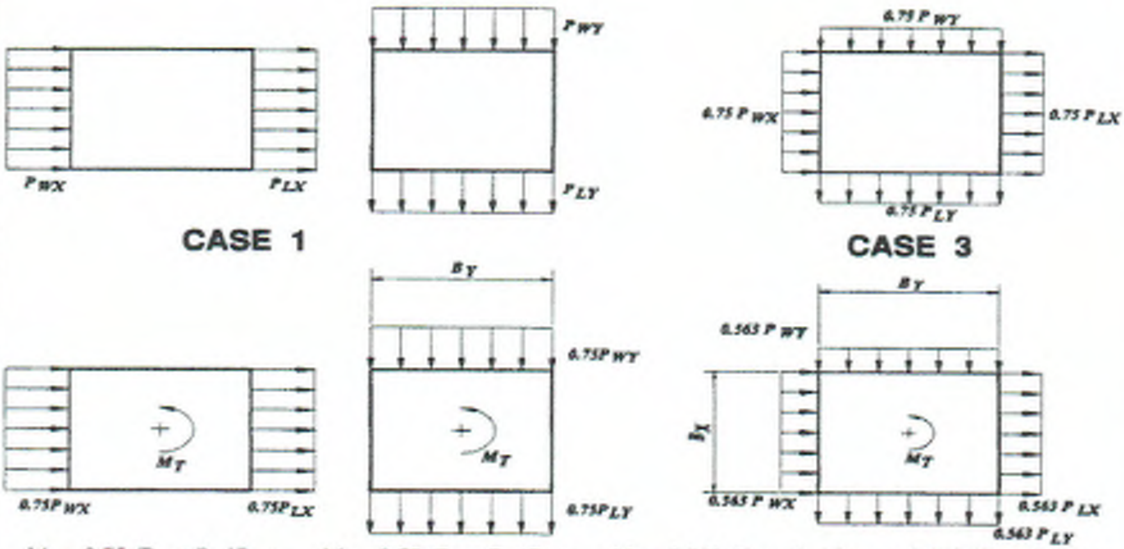
WIND LOAD ON MONOSLOPE ROOF BUILDING (DIRECTIONAL WIND METHOD)

DESIGN PARAMETERS			ASCE 7-10 REF.
EXPOSURE CATEGORY:	C		
RISK CATEGORY:	II		
WIND SPEED, V =	110	MPH	
Kz	1.04		
Kzt	1.00		
Kd	0.85		
GUST FACTOR, G =	0.85		26.9.4
VELOCITY PRESSURE, qz =	27.4	PSF	EQ. (27.3-1)
INTERNAL PRESSURE, (Gcpi)			
PARTIALLY ENCLOSED	0.55	+/-	T26.11.1

WIND CASES	
FULL WIND (TRANS WINDWARD)	CASE 1A
FULL WIND (TRANS LEEWARD)	CASE 1A'
FULL WIND (LONG)	CASE 1B
75% WIND (TRANS) +15% ECC	CASE 2A
75% WIND (LONG) +15% ECC	CASE 2B
75% COMBINED (TRANS)	CASE 3A
75% COMBINED (LONG)	CASE 3B
56.3% COMBINED (TRANS) + 15% ECC	CASE 4A
56.3% COMBINED (LONG) + 15% ECC	CASE 4B

ASCE 7
WIND LOAD CASES

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\ 0.15\ B_Y$

$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\ 0.15\ B_X$ $e_Y = \pm\ 0.15\ B_Y$

CASE 2

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

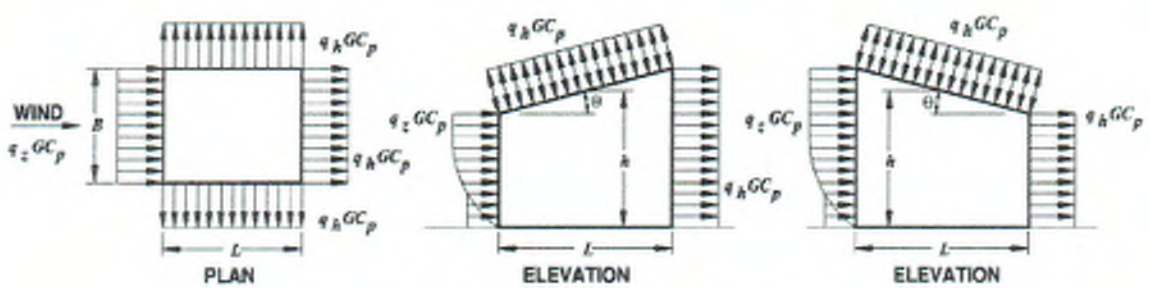
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

		q*G*Cp	+INT	-INT
CASE 1A (TRANS WINDWARD)		PSF	PSF	PSF
ROOF MAX	WINDWARD ROOF	-20.9	-5.9	-36.0
ROOF MIN	MINIMUM ROOF, Cp	-4.2	10.9	-19.3
WINDWARD WALL	WINDWARD WALL	18.6	33.7	3.6
LEEWARD WALL	LEEWARD WALL (TRANS)	-11.6	3.4	-26.7
SIDE WALL	SIDE WALL	-16.3	-1.2	-31.4

CASE 1A' (TRANS LEEWARD)		PSF	PSF	PSF
ROOF MAX	LEEWARD ROOF	-11.6	3.4	-26.7
ROOF MIN	MINIMUM ROOF, Cp	-4.2	10.9	-19.3
WINDWARD WALL	WINDWARD WALL	18.6	33.7	3.6
LEEWARD WALL	LEEWARD WALL (TRANS)	-11.6	3.4	-26.7
SIDE WALL	SIDE WALL	-16.3	-1.2	-31.4

CASE 1B (FULL LONG)		PSF	PSF	PSF
ROOF	PERP OR PARA TO WIND	-7.0	8.1	-22.0
WINDWARD WALL	WINDWARD WALL	18.6	33.7	3.6
LEEWARD WALL	LEEWARD WALL (LONG)	-4.7	10.4	-19.7
SIDE WALL	SIDE WALL	-16.3	-1.2	-31.4



MONOSLOPE ROOF (NOTE 4)

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-6 (con't)	External Pressure Coefficients, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p			
Surface	**L/B**	**C_p**	**Use With**
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h												
Wind Direction	**Windward**									**Leeward**		
	Angle, θ (degrees)									**Angle, θ (degrees)**		
	h/L	**10**	**15**	**20**	**25**	**30**	**35**	**45**	**≥60#**	**10**	**15**	**≥20**
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
200 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B*, *h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 6.5.8.
6. Refer to Figure 6-7 for domes and Figure 6-8 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Design Maps Summary Report

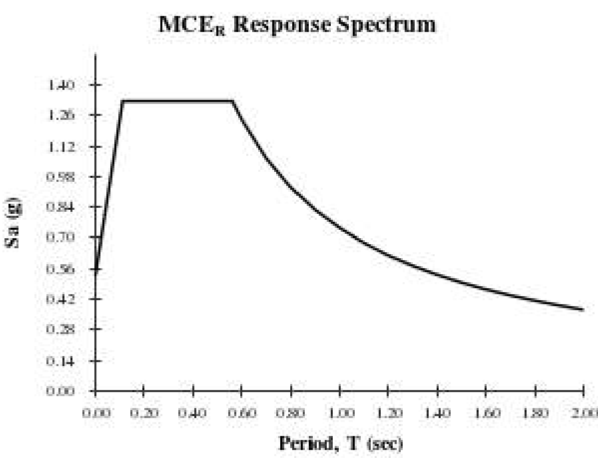
User–Specified Input

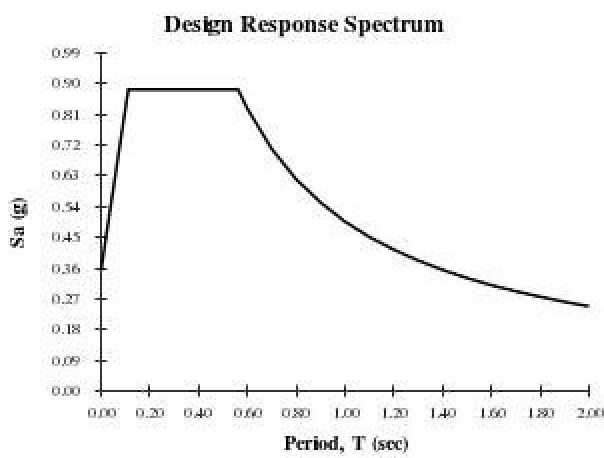
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	47.462°N, 122.09°W
Site Soil Classification	Site Class D – “Stiff Soil”
Risk Category	I/II/III

USGS–Provided Output

S_S =	1.325 g	S_{MS} =	1.325 g	S_{DS} =	0.883 g
S_1 =	0.495 g	S_{M1} =	0.745 g	S_{D1} =	0.496 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.





Design Maps Detailed Report

2012/2015 International Building Code (47.462°N, 122.09°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1]	$S_S = 1.325$ g
From Figure 1613.3.1(2) [2]	$S_1 = 0.495$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.325 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.495 g, F_v = 1.505

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.325 = 1.325$ g

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Equation (16-38): $S_{M1} = F_vS_1 = 1.505 \times 0.495 = 0.745$ g

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Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.325 = 0.883$ g

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Equation (16-40): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.745 = 0.496$ g

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Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	**RISK CATEGORY**		
	I or II	**III**	**IV**
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 0.883 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	**RISK CATEGORY**		
	I or II	**III**	**IV**
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.496 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)” = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

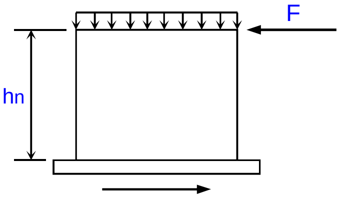
1. *Figure 1613.3.1(1)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

THIS SPREADSHEET IS IN COMPLIANCE WITH IBC 2015

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For the Equivalent Lateral Force Procedure for Regular Single-Level Building/Structural Systems

Input:

Risk Category =	II	ASCE 7-10 Table 1.5-1
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10 Table 20.3-1
Location =	39.95	°N 120.96 °W
Spectral Accel., S_S =	0.976	ASCE 7-10 Chapter 22 Maps
Spectral Accel., S_1 =	0.327	ASCE 7-10 Chapter 22 Maps
Long. Trans. Period, T_L =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16
Structure Height, h_n =	27.000	ft.
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, T_c =	0.000	sec. From independent analysis
Seismic Resist. System =	B3	Steel ordinary concentrically braced frames (ASCE 7-10, Table 12.2-1)



V = Cs*W
Seismic Base Shear

Calculations:

Site Coefficients:

F_a =	1.000	ASCE 7-10 Table 11.4-1	1.110
F_v =	1.510	ASCE 7-10 Table 11.4-2	1.746

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	1.325	$S_{MS} = F_a*S_S$, ASCE 7-10 Eqn. 11.4-1	1.083
S_{M1} =	0.745	$S_{M1} = F_v*S_1$, ASCE 7-10 Eqn. 11.4-2	0.571

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.883	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3	0.722
S_{D1} =	0.496	$S_{D1} = 2*S_{M1}/3$, ASCE 7-10 Eqn. 11.4-4	0.381

Seismic Design Category:

Category(for S_{DS}) =	D	ASCE 7-10 Table 11.6-1
Category(for S_{D1}) =	D	ASCE 7-10 Table 11.6-2
Use Category =	D	The most critical of the two categories.

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approximate Period, T_a =	0.237	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.400	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.332	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.237	sec., T = Ta <= Cu*Ta, ASCE 7-10 Section 12.8.2

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.25	ASCE 7-10 Table 12.2-1
Overstrength Factor, Ω_o =	2	ASCE 7-10 Table 12.2-2
Defl. Amplif. Factor, C_d =	3.25	ASCE 7-10 Table 12.2-3
C_S =	0.272	$C_S = S_{DS}/(R/I)$, ASCE 7-05 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.644	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-05 Eqn. 12.8-3
$C_{S(min)}$ =	0.039	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-10 Eqn. 12.8-5
Use: C_S =	0.272	$C_{S(min)} <= C_S <= C_{S(max)}$

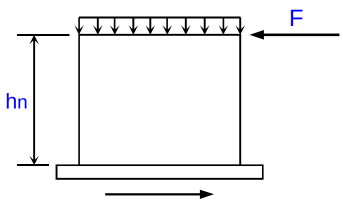
Seismic Base Shear:

V =	0.27	kips, $V = C_S*W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For the Equivalent Lateral Force Procedure for Regular Single-Level Building/Structural Systems

Input:

Risk Category =	II	ASCE 7-10 Table 1.5-1
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10 Table 20.3-1
Location =	39.95	°N 120.96 °W
Spectral Accel., S_S =	0.976	ASCE 7-10 Chapter 22 Maps
Spectral Accel., S_1 =	0.327	ASCE 7-10 Chapter 22 Maps
Long. Trans. Period, T_L =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16
Structure Height, h_n =	43.000	ft.
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, T_c =	0.000	sec. From independent analysis
Seismic Resist. System =	C4	Steel ordinary moment frames (ASCE 7-10, Table 12.2-1)



Seismic Base Shear

ALLOWED FOR ROOF DL < 20 PSF, OK

Calculations:

Warning: Seismic Resisting System which was selected is NOT Permitted!

Site Coefficients:

F_a =	1.000	ASCE 7-10 Table 11.4-1	1.110
F_v =	1.510	ASCE 7-10 Table 11.4-2	1.746

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	1.325	$S_{MS} = F_a*S_S$, ASCE 7-10 Eqn. 11.4-1	1.083
S_{M1} =	0.745	$S_{M1} = F_v*S_1$, ASCE 7-10 Eqn. 11.4-2	0.571

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.883	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3	0.722
S_{D1} =	0.496	$S_{D1} = 2*S_{M1}/3$, ASCE 7-10 Eqn. 11.4-4	0.381

Seismic Design Category:

Category(for S_{DS}) =	D	ASCE 7-10 Table 11.6-1
Category(for S_{D1}) =	D	ASCE 7-10 Table 11.6-2
Use Category =	D	The most critical of the two categories.

Fundamental Period:

Period Coefficient, C_T =	0.028	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.80	ASCE 7-10 Table 12.8-2
Approximate Period, T_a =	0.567	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.400	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.794	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.567	sec., T = Ta <= Cu*Ta, ASCE 7-10 Section 12.8.2

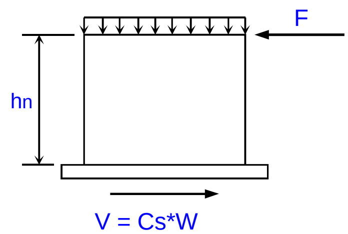
Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-10 Table 12.2-1
Overstrength Factor, Ω_o =	3	ASCE 7-10 Table 12.2-2
Defl. Amplif. Factor, C_d =	3	ASCE 7-10 Table 12.2-3
C_S =	0.252	$C_S = S_{DS}/(R/I)$, ASCE 7-05 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.250	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-05 Eqn. 12.8-3
$C_{S(min)}$ =	0.039	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-10 Eqn. 12.8-5
Use: C_S =	0.250	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	0.25	kips, $V = C_S*W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For the Equivalent Lateral Force Procedure for Regular Single-Level Building/Structural Systems



Seismic Base Shear

Input:

Risk Category =	II	ASCE 7-10 Table 1.5-1
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10 Table 20.3-1
Location =	39.95	°N 120.96 °W
Spectral Accel., S_S =	0.976	ASCE 7-10 Chapter 22 Maps
Spectral Accel., S_1 =	0.327	ASCE 7-10 Chapter 22 Maps
Long. Trans. Period, T_L =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16
Structure Height, h_n =	12.000	ft.
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, Tc =	0.000	sec. From independent analysis
Seismic Resist. System =	A1	Special reinforced concrete shear walls (ASCE 7-10, Table 12.2-1)

Calculations:

Site Coefficients:

F_a =	1.000	ASCE 7-10 Table 11.4-1	1.110
F_v =	1.510	ASCE 7-10 Table 11.4-2	1.746

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	1.325	$S_{MS} = F_a*S_S$, ASCE 7-10 Eqn. 11.4-1	1.083
S_{M1} =	0.745	$S_{M1} = F_v*S_1$, ASCE 7-10 Eqn. 11.4-2	0.571

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.883	$S_{DS} = 2*S_{MS}/3$, ASCE 7-10 Eqn. 11.4-3	0.722
S_{D1} =	0.496	$S_{D1} = 2*S_{M1}/3$, ASCE 7-10 Eqn. 11.4-4	0.381

Seismic Design Category:

Category(for S_{DS}) =	D	ASCE 7-10 Table 11.6-1
Category(for S_{D1}) =	D	ASCE 7-10 Table 11.6-2
Use Category =	D	The most critical of the two categories.

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approximate Period, T_a =	0.129	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.400	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.181	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8.2
Fundamental Period, T =	0.129	sec., T = Ta <= Cu*Ta, ASCE 7-10 Section 12.8.2

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	5	ASCE 7-10 Table 12.2-1
Overstrength Factor, Ω_o =	2.5	ASCE 7-10 Table 12.2-2
Defl. Amplif. Factor, C_d =	5	ASCE 7-10 Table 12.2-3
C_S =	0.177	$C_S = S_{DS}/(R/I)$, ASCE 7-05 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.769	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-05 Eqn. 12.8-3
$C_{S(min)}$ =	0.039	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7-10 Eqn. 12.8-5
Use: C_S =	0.177	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	0.18	kips, $V = C_S*W$, ASCE 7-10 Section 12.8.1, Eqn. 12.8-1

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For Nonbuilding Structures Not Similar to Buildings

Input:

Risk Category =	II	IBC 2012, Table 1604.5
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10 Chapter 20
Location =	39.95	°N 120.96 °W
Spectral Accel., S_S =	0.976	ASCE 7-10 Chapter 22 Maps
Spectral Accel., S_1 =	0.327	ASCE 7-10 Chapter 22 Maps
Long, Trans. Period, T_L =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16
Height, h =	43.000	ft.
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, Tc =	0.000	sec. From independent analysis
Structure Type =	04a	Elevated tanks, vessels, bins or hoppers on symmetrically braced legs (not similar to buildings) (ASCE 7-10 Table 15.4-2)

Results:

Site Coefficients:

F_a =	1.000	IBC 2012 Table 1613.3.3(1)
F_v =	1.510	IBC 2012 Table 1613.3.3(2)

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	1.325	$S_{MS} = F_a*S_s$, IBC 2012 Eqn. 16-37
S_{M1} =	0.745	$S_{M1} = F_v*S_1$, IBC 2012 Eqn. 16-38

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.883	$S_{DS} = 2*S_{MS}/3$, IBC 2012 Eqn. 16-39
S_{D1} =	0.496	$S_{D1} = 2*S_{M1}/3$, IBC 2012 Eqn. 16-40

Seismic Design Category:

Category(for S_{DS}) =	D	IBC 2012 Table 1613.3.5(1)
Category(for S_{D1}) =	D	IBC 2012 Table 1613.3.5(2)
Use Category =	D	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.02	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, T_a =	0.336	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Eqn. 12.8-7
Upper Limit Coef., C_u =	1.4	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.470	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8-2
Fundamental Period, T =	0.470	sec., T = SD1/SDS <= Cu*Ta, ASCE 7-10 Section 12.8.2
Rigid or Flexible?	Flexible	Criteria: If T < 0.06, then Rigid, else if T >= 0.06, then Flexible

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3	ASCE 7-10 Table 15.4-2
Overstrength Factor, Ω_o =	2	ASCE 7-10 Table 15.4-2
Defl. Amplif. Factor, C_d =	2.5	ASCE 7-10 Table 15.4-2
C_S =	0.294	$C_S = S_{DS}/(R/I)$, ASCE 7-10 Eqn. 12.8-2
$C_{S(max)}$ =	0.352	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
$C_{S(min)}$ =	0.039	Cs(min) = 0.044*SDS*I >= 0.03, ASCE 7-10 Eqn. 15.4-1
Use: C_S =	0.294	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	0.29	kips, for Flexible: V = CS*W, ASCE 7-10 Eqn. 12.8-1

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For Nonbuilding Structures Not Similar to Buildings

Input:

Risk Category =	IV	IBC 2012, Table 1604.5	
Importance Factor, I =	1.50	ASCE 7-10 Table 1.5-2	
Soil Site Class =	D	ASCE 7-10 Chapter 20	
Location =	39.95	°N	120.96 °W
Spectral Accel., S_S =	0.976	ASCE 7-10 Chapter 22 Maps	
Spectral Accel., S_1 =	0.327	ASCE 7-10 Chapter 22 Maps	
Long, Trans. Period, T_L =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16	
Height, h =	43.000	ft.	
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2	
Actual Calc. Period, Tc =	0.000	sec. From independent analysis	
Structure Type =	04a	Elevated tanks, vessels, bins or hoppers on symmetrically braced legs (not similar to buildings) (ASCE 7-10 Table 15.4-2)	

Results:

Site Coefficients:

F_a =	1.000	IBC 2012 Table 1613.3.3(1)
F_v =	1.510	IBC 2012 Table 1613.3.3(2)

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	1.325	$S_{MS} = F_a*S_s$, IBC 2012 Eqn. 16-37
S_{M1} =	0.745	$S_{M1} = F_v*S_1$, IBC 2012 Eqn. 16-38

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.883	$S_{DS} = 2*S_{MS}/3$, IBC 2012 Eqn. 16-39
S_{D1} =	0.496	$S_{D1} = 2*S_{M1}/3$, IBC 2012 Eqn. 16-40

Seismic Design Category:

Category(for S_{DS}) =	D	IBC 2012 Table 1613.3.5(1)
Category(for S_{D1}) =	D	IBC 2012 Table 1613.3.5(2)
Use Category =	D	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.02	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-10 Table 12.8-2
Approx. Period, T_a =	0.336	sec., $T_a = C_T*h_n$^(x), ASCE 7-10 Eqn. 12.8-7
Upper Limit Coef., C_u =	1.4	ASCE 7-10 Table 12.8-1
Period max., $T_{(max)}$ =	0.470	sec., $T_{(max)} = C_u*T_a$, ASCE 7-10 Section 12.8-2
Fundamental Period, T =	0.470	sec., T = SD1/SDS <= Cu*Ta, ASCE 7-10 Section 12.8.2
Rigid or Flexible?	Flexible	Criteria: If T < 0.06, then Rigid, else if T >= 0.06, then Flexible

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3	ASCE 7-10 Table 15.4-2
Overstrength Factor, Ω_o =	2	ASCE 7-10 Table 15.4-2
Defl. Amplif. Factor, C_d =	2.5	ASCE 7-10 Table 15.4-2
C_S =	0.442	$C_S = S_{DS}/(R/I)$, ASCE 7-10 Eqn. 12.8-2
$C_{S(max)}$ =	0.527	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
$C_{S(min)}$ =	0.058	Cs(min) = 0.044*SDS*I >= 0.03, ASCE 7-10 Eqn. 15.4-1
Use: C_S =	0.442	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	0.44	kips,	for Flexible: V = CS*W, ASCE 7-10 Eqn. 12.8-1

SEISMIC BASE SHEAR
Using IBC 2012 and ASCE 7-10 Codes
For Nonbuilding Structures Similar to Buildings

Input:

Risk Category =	II	IBC 2012, Table 1604.5
Importance Factor, I =	1.00	ASCE 7-10 Table 1.5-2
Soil Site Class =	D	ASCE 7-10 Chapter 20
Location =	39.95	°N 120.96 °W
Spectral Accel., Ss =	0.976	ASCE 7-10 Chapter 22 Maps
Spectral Accel., S1 =	0.327	ASCE 7-10 Chapter 22 Maps
Long, Trans. Period, TL =	16.000	sec. ASCE 7-10 Fig's. 22-12 to 22-16
Height, h =	12.000	ft.
Total Seismic Weight, W =	1.00	kips ASCE 7-10 Section 12.7.2
Actual Calc. Period, Tc =	0.000	sec. From independent analysis
Structure Type =	03i	Ordinary moment frames of steel (ASCE 7-10 Table 15.4-1)

Results:

Warning: Seismic Resisting System which was selected is NOT Permitted!

ALLOWED FOR ROOF DL < 20 PSF

Site Coefficients:

Fa =	1.000	IBC 2012 Table 1613.3.3(1)
Fv =	1.510	IBC 2012 Table 1613.3.3(2)

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

SMS =	1.325	SMS = Fa*Ss, IBC 2012 Eqn. 16-37
SM1 =	0.745	SM1 = Fv*S1, IBC 2012 Eqn. 16-38

Design Spectral Response Accelerations for Short and 1-Second Periods :

SDS =	0.883	SDS = 2*SMS/3, IBC 2012 Eqn. 16-39
SD1 =	0.496	SD1 = 2*SM1/3, IBC 2012 Eqn. 16-40

Seismic Design Category:

Category(for SDS) =	D	IBC 2012 Table 1613.3.5(1)
Category(for SD1) =	D	IBC 2012 Table 1613.3.5(2)
Use Category =	D	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, CT =	0.028	ASCE 7-10 Table 12.8-2
Period Exponent, x =	0.8	ASCE 7-10 Table 12.8-2
Approx. Period, Ta =	0.204	sec., Ta = CT*hn^(x), ASCE 7-10 Eqn. 12.8-7
Upper Limit Coef., Cu =	1.4	ASCE 7-10 Table 12.8-1
Period max., T(max) =	0.286	sec., T(max) = Cu*Ta, ASCE 7-10 Section 12.8-2
Fundamental Period, T =	0.286	sec., T = SD1/SDS <= Cu*Ta, ASCE 7-10 Section 12.8.2
Rigid or Flexible?	Flexible	Criteria: If T < 0.06, then Rigid, else if T >= 0.06, then Flexible

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	3.5	ASCE 7-10 Table 15.4-1
Overstrength Factor, Ωo =	3	ASCE 7-10 Table 15.4-1
Defl. Amplif. Factor, Cd =	3	ASCE 7-10 Table 15.4-1
Cs =	0.252	Cs = SDS/(R/I), ASCE 7-10 Eqn. 12.8-2
Cs(max) =	0.495	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7-10 Eqn. 12.8-3
Cs(min) =	0.039	Cs(min) = 0.044*SDS*I >= 0.03, ASCE 7-10 Eqn. 15.4-1
Use: Cs =	0.252	Cs(min) <= Cs <= Cs(max)

Seismic Base Shear:

V =	0.25	kips, for Flexible: V = CS*W, ASCE 7-10 Eqn. 12.8-1

DESIGN CRITERIA & LOADING:

GEOTECH PARAMETERS:
REFERENCE GEOTECH CONSULTANTS, INC. REPORT DATED MAY 16, 2018

- ALLOWABLE SOIL BEARING PRESSURE = 3000 PSF
- 1/3 ALLOWABLE INCREASE FOR SHORT TERM (WIND OR SEISMIC LOADS)
- 350 PSF ALLOWABLE PASSIVE RESISTANCE
- COEFFICIENT OF FRICTION = 0.5

CONCRETE REQUIREMENTS

MINIMUM COMPRESSIVE STRENGTH, $f'_c := 4500\ psi$

**STRUCTURAL CALCULATIONS ASSUME f'c = 4000 PSI (CONSERVATIVE)*

EXPOSURE CLASSES	(MINIMUM f'c PER IBC TABLE 19.3.2.1)
FREEZING & THAWING, Fx = F2	4500 psi
SULFATE, Sx = S1	4000 psi
WATER, Wx= W1	4000 psi
CORROSION, Cx = C1	2500 psi

City Amendments to IRC Table R301.2(1), Climatic and Geographic Design Criteria: Table R301.2(1) of the International Residential Code is amended to read as follows:

IRC Table R301.2(1)
Climatic and Geographic Design Criteria

Roof Snow Load[1]	Wind Design[2]		Seismic Design Category [3]	Subject to Damage From:			Outside Design Temp. – Heat/Cool	Ice Barrier Under-layment Required	Flood Hazards[5]	Air Freezing Index	Mean Annual Temp.
	Speed	Topo-graphic Effects		Weathering [4]	Frost Line Depth	Termite Decay					
25 psf	110 mph	See footnote 2	D2	Moderate	12"	Slight to Moderate	24ºF/83ºF	No	N/A	113	50ºF

Footnotes:

1. When using this roof snow load it will be left to the engineer's judgment whether to consider drift or sliding snow. However, rain on snow surcharge of five (5) psf must be considered for roof slopes less than five degrees (5°).

2. Wind exposure category and Topographic effects (Wind Speed-up Kzt factor) shall be determined on a site-specific basis by the Design Professional in Responsible Charge (components and cladding need not consider topographic effects unless otherwise determined by the engineer of record).

3. From IRC Table 301.2(1).

4. Weathering may require a higher strength concrete or grade of masonry than necessary to satisfy the structural requirements of this code. The grade of masonry units shall be determined from ASTM C 34, C 55, C 62, C 73, C 90, C 129, C 145, C 216 or C 652.

5. The City of Renton participates in the National Flood Insurance Program (NFIP) as specified in City of Renton Resolution No. 1984, dated April 21, 1975. The City's Flood Insurance Study is April 19, 2005, and the number and date of current effective Flood Insurance Rate Maps (FIRMS) are as follows:

53033CIND0A 04/19/2005
53033C0664F 05/16/1995
53033C0666F 05/16/1995
53033C0668F 05/16/1995
53033C0669F 05/16/1995

GEOTECH
CONSULTANTS, INC.

2401 10th Ave E
Seattle, Washington 98102
(425) 747-5618

May 16, 2018

JN 18257

Lakeside Industries
P.O. Box 7016
Issaquah, Washington 98027

Attention: Bill Dempsey
via email: bill.dempsey@lakesideindustries.com

Subject: **Geotechnical Engineering Report**
Proposed New Asphalt Plant
18825 SR-169 (Renton-Maple Valley Highway)
King County, Washington

Dear Mr. Dempsey:

This report presents our geotechnical observations and conclusions related to the proposed placement of a new asphalt plant on the subject property, which is located in unincorporated King County between Maple Valley and Renton. We visited the site on May 15, 2018 to observe the existing conditions and to monitor the excavation of test pits in the general area of the proposed asphalt plant. Our services were provided in general accordance with the scope outlined in our **Contract for Professional Services,** which you approved.

The property in question is located on the west side of SR-169. In the past, it was used as the King County Shops. More recently, it has been utilized as a topsoil distribution and fill recycling facility. All previous structures have been removed, and removal of the last of the stockpiled crushed concrete fill as underway at the time of our site visit. We expect that an asphalt plant will be installed on the northern portion of the property, on the relatively flat area along the west side of SR-169. This plant, which will include tall silos used to store and distribute asphalt products, will be installed in the southern portion of the planned development area. This plant will likely sit on a large concrete mat foundation.

During our visit we observed the excavation of five test pits around the anticipated footprint of the foundation. These test pits revealed only 2 to 3 feet of fill below the existing ground surface. This fill was variable in composition, and some of it included coal fragments. Underlying the fill, the test pits revealed medium-dense, medium-grained, gravelly sand to sand that extended to the maximum 7-foot depth of the explorations. This coarse-grained soil has been deposited by fast flowing water since the last glaciers receded from the area. This is typical for the conditions in the Cedar River Valley. No groundwater was encountered in the test pits. We understand that groundwater was found at a depth of approximately 8 feet in previous infiltration test pits conducted further to the south, but these may have been excavated at a lower existing ground elevation.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on our observations, the site is underlain by competent native soils suitable to support the proposed foundation for the new asphalt plant. It will be necessary to excavate through the fill and to reach the competent material below. Once the native sand soils has been reached, its surface should be densified using a hoe-pack. If the excavation to reach suitable bearing soils extends below the planned footing subgrade elevation, the overexcavation could be backfilled with compacted clean crushed rock (quarry spalls or railroad ballast rock). If overexcavation and subsequent placement of structural fill is necessary, it will be important that a geotechnical engineer observes the base of the excavation to verify that competent soil has been encountered before structural fill is placed.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (stiff soils). The soils that underlie the site are coarse-grained, and have a low susceptibility to seismic liquefaction, even under the Maximum Considered Earthquake (MCE), due to their dense, well-drained condition. The MCE has a theoretical return period of once in 2,475 years (2 percent probability of occurring in a 50-year period). Even if liquefaction were to occur, the recommended mat foundation will provide protection against catastrophic foundation collapse due to soil strength loss.

CONVENTIONAL FOUNDATIONS

An allowable bearing pressure of 3,000 pounds per square foot (psf) is appropriate for a mat foundation supported on the native soil underlying the existing fill, or on compacted rock fill placed over the native soils. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be less than one inch, with differential settlements on the order of one-half inch in a distance of 50 feet along a continuous footing with a uniform load.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level, well-compacted fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.50
Passive Earth Pressure	350 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) passive earth pressure is computed using the equivalent fluid density

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. The above values do not include a safety factor.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our field work. This report has been prepared for the exclusive use of Lakeside Industries and its representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with current standards of practice within the scope of our services and within budget and time constraints. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Please contact us if you have any questions regarding this report.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

05/16/18

Marc R. McGinnis, P.E.
Principal

cc: **B & T Design and Engineering** - Jim Trueblood
via email jim@bntengr.com

MRM:kg

IBC CODE REVIEW

CH. 3 - USE AND OCCUPANCY

*ASSUME ALL STRUCTURES FALL UNDER "LOW-HAZARD STORAGE", GROUP S-2

CH. 5 - GENERAL BUILDING HEIGHTS & AREAS

TYPE OF CONSTRUCTION: TYPE II B

MAX ALLOWABLE HEIGHT = 55' (TABLE 504.3) < MAX HEIGHT = 45', OK!!

ALLOWABLE AREA:

$NS := 26000\ ft^2$ (ALLOWABLE AREA FACTOR, W/O SPRINKLER, GROUP S-2)

$A_t := NS = \left(2.6 \cdot 10^4\right)\ ft^2$ TABULAR ALLOWABLE AREA FACTOR

$F := 100\ ft$ $P := 100\ ft \cdot 2 + 50\ ft \cdot 2 = 300\ ft$ $W := 60\ ft$

$I_f := \left(\frac{F}{P} - 0.25\right)\frac{W}{30\ ft} = 0.167$ FRONTAGE AREA FACTOR INCREASE, EQ. 5-5
*AGGREGATE STORAGE BUILD #2 CONTROLS

$A_a := A_t + \left(NS \cdot I_f\right) = 30333\ ft^2$ ALLOWABLE AREA, EQ. 5-1

$A_b := 362\ ft \cdot 79\ ft = 28598\ ft^2$ AGGREGATE STORAGE MAIN BUILDING CONTROLS,
Ab < Aa, OK!!!

AGGREGATE STORAGE (BUILDING #1 & #2) & RAP CRUSHER/STORAGE (BUILDING #3) DESIGN

*SEE PROJECT DESIGN CRITERIA FOR INFORMATION NOT SHOWN HERE

BUILDING #1, #2, & #3 ARE RISK II STRUCTURES

LATERAL FORCE RESISTING SYSTEMS:
BUILDING #1 - ORDINARY CONCENTRIC BRACED FRAME (EA. DIRECTION)
BUILDING #2 & #3 - ORDINARY CONCENTRIC BRACED FRAME (LONGITUDINAL)
ORDINARY MOMENT FRAME (TRANSVERSE)

CONVEYOR LOADING ASSUMPTIONS:

CV CAPACITY PER LAKESIDE = 300 TPH
MATERIAL DENSITY = 105 PCF W/ 20 DEG SURCHARGE ANGLE

TRIPPER BELT SPEED = 300 TO 500 FPM (*ASSUME 300 FPM)

ASSUME 30" BELT @ 300 FPM W/ 20 DEG IDLERS (CONSERVATIVE)

PER CEMA, $A_s := 0.523\ ft^2$

CONVEYOR OUTPUT , $w_{matl} := A_s \cdot \frac{(105\ pcf)}{2000} \cdot 300\ \frac{ft}{min} \cdot 60\ min = 494.235\ lbf$

TYPICAL CONVEYOR LOADING

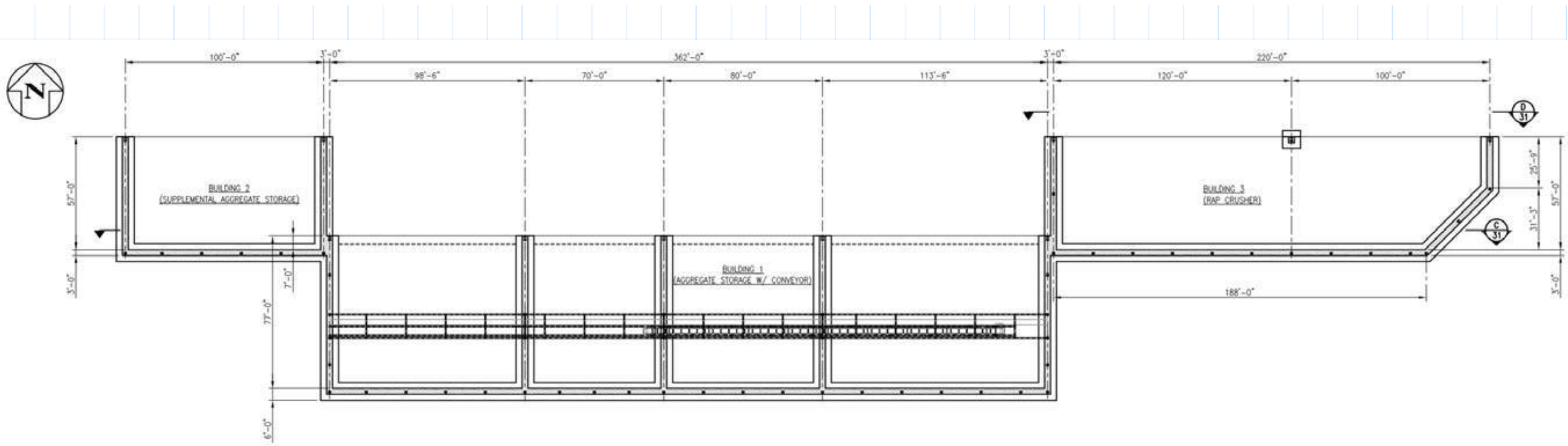
MATERIAL WEIGHT = 60 PLF
30" BELT = 25 PLF
IDLER = 35 PLF
RETURN IDLER = 10 PLF
CONDUIT = 30 PLF
WALKWAY = 20 PLF

TOTAL CONVEYOR LOAD (DL + MATERIAL) = 180 PLF

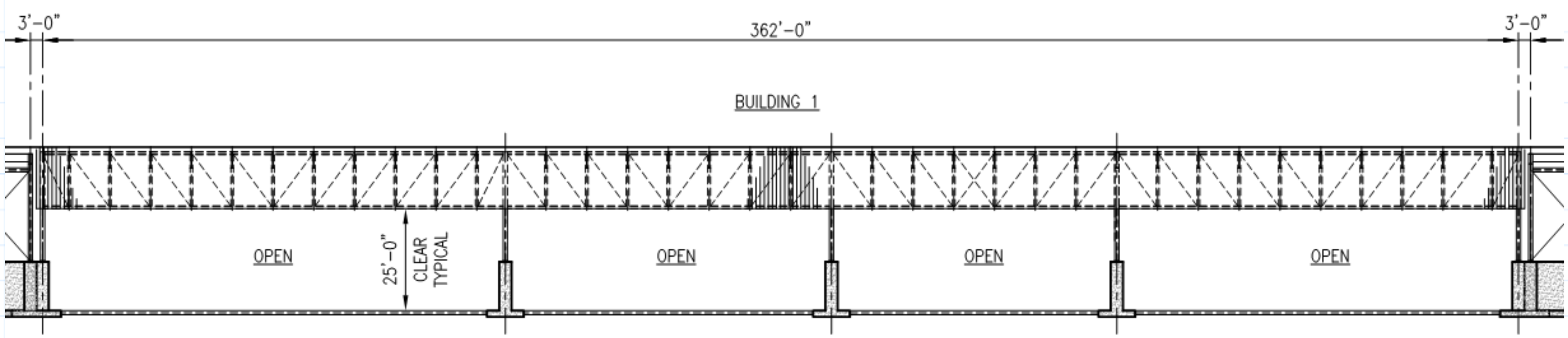
BUILDINGS 1, 2, & 3 HAVE BEEN DESIGNED ASSUMING A STEEL FRAMED BUILDING (OCBF & OMF) SUPPORTED BY REINFORCED CONCRETE BIN WALLS (SPECIAL REINFORCED CONCRETE WALL).

THE BUILDING ROOF WAS ASSUMED FLEXIBLE AND NEGLECTED FOR STRENGTH & STABILITY CALCULATIONS IN THE MODEL. A CONSERVATIVE DIAPHRAGM WAS ADDED TO A SPEARATE MODEL TO DETERMINE CONSERVATIVE OVERALL DEFLECTIONS IN ACCORDANCE WITH IBC DEFLECTION LIMITS.

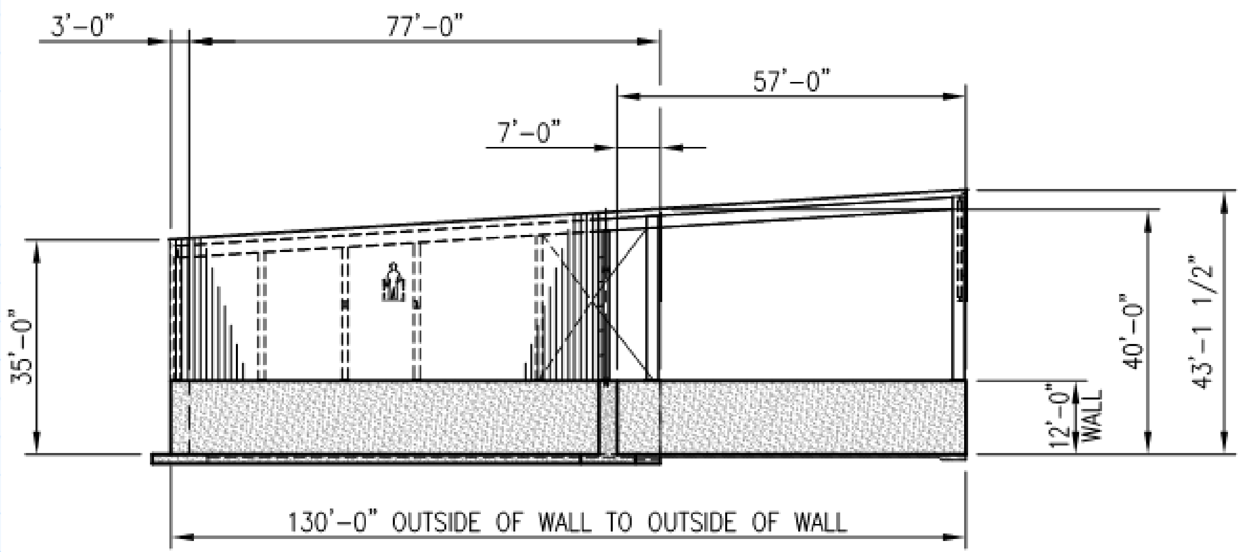
THESE TWO STRUCTURES WERE ANALYZED SEPARATELY WITH THE WORST CASE COLUMN REACTIONS ADDED TO THE BIN WALL DESIGN.



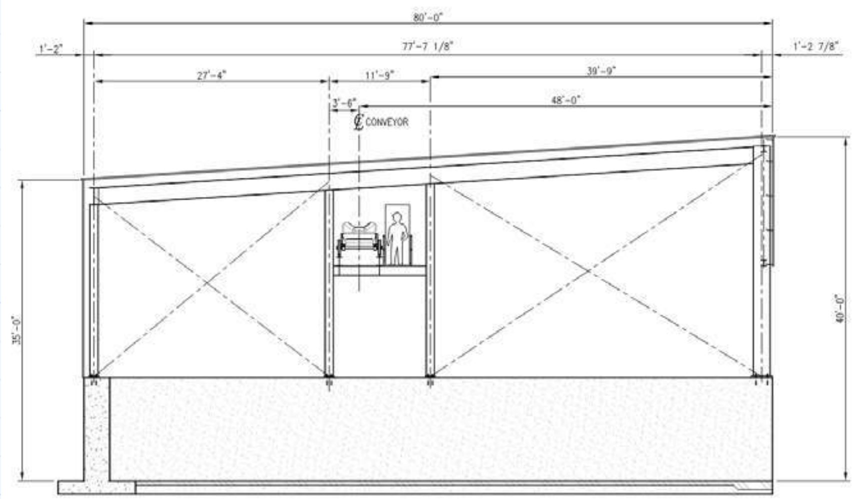
AGGREGATE STORAGE & RAP CRUSHER STRUCTURES (PLAN VIEW)



AGGREGATE STORAGE & RAP CRUSHER STRUCTURES (OPEN FACE ELEVATION VIEW)



AGGREGATE STORAGE & RAP CRUSHER STRUCTURES (ELEVATION SIDE VIEW)



AGGREGATE STORAGE & RAP CRUSHER STRUCTURES (TYPICAL SECTION VIEW)

SHEET B3

Smith Monroe Gray
ENGINEERS, INC.

CLIENT ______
PROJECT ______

BY ______ DATE ______ REV. ______
JOB NO. ______ SHEET ______ OF ______

Aggregate Storage Building Design

Roof (Open Web Joists @ 10' OC, 80' Span) (Bldg #1)

W_{DL} = 5 PSF
Snow = 25 PSF
W_W = 36 PSF (Trans Wind) * Negative Pressure Controls
22 PSF (Long Wind)

MAXIMUM SERVICE LEVEL COMBINED LOADING (ASD) = 30 PSF (32 PSF IS CONSERVATIVE)

Per Model, W_{max} = 32 PSF

@ 10' OC, W = 320 PLF & 80' SPAN

UNIFORM LOAD FOR L/360 = 216 PLF x 360/240 = 324 PLF

Use 40LH14 or Similar
W_{allow} = 324 PLF (L/240)

Roof (Deck & Purlin)

Try Z-Purlins @ 4' OC

W_{max} = 32 PSF x 4' = 128 PLF
Span = 10'

$M_{max} = wl^2/8 = 128 \text{ PLF} (10')^2/8 = 1600^{\#\cdot FT} \rightarrow \underline{19.2^{K\text{-}IN}}$

Use 6" x 16 GA Z Purlin, W_{allow} = 254 PLF

Try Verco PLB 36 Decking (22GA)

$M_{max} = 32 \text{ PLF} \cdot (4')^2/8 = 64^{\#\cdot FT}/FT$

$S_{min} = 0.176 \text{ in}^3 \rightarrow f_b = \dfrac{0.064^{K\cdot FT} \times 12^{IN/FT}}{0.176 \text{ in}^3} = \underline{4.4 \text{ KSI OK}}$

ROOF DESIGN @ BLDG #2 & #3

UNIFORM LOAD FOR L/360 = 227 PLF x 360/240 = 340 PLF

SPAN = 60^{FT} MAX.

W = 32 PSF @ 10' OC → 320 PLF

USE VULCRAFT 32LH11 OPEN WEB JOISTS W_{ALLOW} = 340 PLF @ L/240

*USE SAME Z PURLIN & DECK AS BLDG #1

WALL GIRTS & DECK

MAX. WIND, 0.6W = 0.6·(0.75·34 PSF + 0.75·32 PSF)
= 29.7 PSF → 30 PSF

MAX. COLUMN SP'G/GIRT SP'G = 23^{FT}

GIRT SP'G = 6^{FT} OC MAX

W_{GIRT} = 30 PSF x 6' = 180 PLF

USE 10" Z-GIRTS w/ 3½" FLANGE x 12 GA W_{ALLOW} = 185 PLF

DECK DEMAND: VERCO PLB 36 x 22 GA

$M_{MAX} = 30\ PLF \times (6')^2/8 = 135^{\#\cdot FT}$

$S_{MIN} = 0.176\ in^3 \rightarrow S_b = \dfrac{0.135^{K\cdot FT} \times 12^{IN/FT}}{0.176\ in^3} = 9.2\ ksi$ OK ✓

USE VERCO PLB 36 x 22 GA.

ASD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES

Based on a 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists only)	Depth in inches	Max Load (plf)	SAFELOAD* in Lbs. Between			SPAN IN FEET														
			< 29	29-33			34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	401	11620			342 235	339 226	336 218	323 204	307 188	293 175	279 162	267 152	256 141	244 132	234 124	224 116	215 109	207 102	199 96
24LH04	12	24	491	14240			419 288	398 265	379 246	360 227	343 210	327 196	312 182	298 169	285 158	273 146	262 138	251 130	241 122	231 114	222 107
24LH05	13	24	526	15260			449 308	446 297	440 285	419 264	399 244	380 226	363 210	347 196	331 182	317 171	304 160	291 150	280 141	269 132	258 124
24LH06	16	24	708	20520			604 411	579 382	555 356	530 331	504 306	480 284	457 263	437 245	417 228	399 211	381 197	364 184	348 172	334 161	320 152
24LH07	17	24	777	22540			665 452	638 421	613 393	588 367	565 343	541 320	516 297	491 276	468 257	446 239	426 223	407 208	389 195	373 182	357 171
24LH08	18	24	829	24040			707 480	677 447	649 416	622 388	597 362	572 338	545 314	520 292	497 272	475 254	455 238	435 222	417 208	400 196	384 184
24LH09	21	24	976	28300			832 562	808 530	785 501	764 460	731 424	696 393	663 363	632 337	602 313	574 292	548 272	524 254	501 238	480 223	460 209
24LH10	23	24	1031	29900			882 596	856 559	832 528	809 500	788 474	768 439	737 406	702 378	668 351	637 326	608 304	582 285	556 266	533 249	511 234
24LH11	25	24	1087	31520			927 624	900 588	875 555	851 525	829 498	807 472	787 449	768 418	734 388	701 361	671 337	642 315	616 294	590 276	567 259
			< 34	34-41			42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	415	14120			337 219	323 205	310 192	297 180	286 169	275 159	265 150	255 142	245 133	237 126	228 119	220 113	213 107	206 102	199 97
28LH06	16	28	552	18760			448 289	429 270	412 253	395 238	379 223	364 209	350 197	337 186	324 175	313 166	301 156	291 148	281 140	271 133	262 126
28LH07	17	28	623	21180			505 326	484 305	464 285	445 267	427 251	410 236	394 222	379 209	365 197	352 186	339 176	327 166	316 158	305 150	295 142
28LH08	18	28	667	22680			540 348	517 325	496 305	475 285	456 268	438 252	420 236	403 222	387 209	371 196	357 185	344 175	331 165	319 156	308 148
28LH09	21	28	821	27920			667 428	639 400	612 375	586 351	563 329	540 309	519 291	499 274	481 258	463 243	446 228	430 216	415 204	401 193	387 183
28LH10	23	28	888	30540			729 466	704 439	679 414	651 388	625 364	600 342	576 322	554 303	533 285	513 269	495 255	477 241	460 228	444 215	429 204
28LH11	25	28	984	32760			780 498	762 475	736 448	711 423	682 397	655 373	629 351	605 331	582 312	561 294	540 278	521 263	502 249	485 236	468 223
28LH12	27	28	1058	35980			857 545	837 520	818 496	800 476	782 454	766 435	737 408	709 383	682 361	656 340	632 321	609 303	587 285	566 270	546 256
28LH13	30	28	1103	37500			895 569	874 543	854 518	835 495	816 472	799 452	782 433	766 415	751 396	722 373	694 352	668 332	643 314	620 297	598 281
			< 39	39-46	47-49		50	51	52	53	54	55	56	57	58	59	60	61	62	63	64
32LH06	14	32	431	16820	16820		338 211	326 199	315 189	304 179	294 169	284 161	275 153	266 145	257 138	249 131	242 125	234 119	227 114	220 108	214 104
32LH07	16	32	485	18920	18920		379 235	366 223	353 211	341 200	329 189	318 179	306 170	298 162	288 154	279 146	271 140	262 133	254 127	247 121	240 116
32LH08	17	32	527	20540	20540		411 255	397 242	383 229	369 216	357 205	345 194	333 184	322 175	312 167	302 159	293 151	284 144	275 137	267 131	259 125
32LH09	21	32	661	25780	25780		516 319	498 302	480 285	463 270	447 256	432 243	418 230	404 219	391 208	379 198	367 189	356 180	346 172	335 164	325 157
32LH10	21	32	731	28500	28500		571 [illegible]	560 [illegible]	531 [illegible]	512 [illegible]	495 [illegible]	478 [illegible]	462 [illegible]	445 [illegible]	430 [illegible]	416 [illegible]	402 [illegible]	389 [illegible]	376 [illegible]	364 [illegible]	353 [illegible]
32LH11	24	32	801	31220	31220		625 385	602 383	580 343	560 325	541 308	522 292	505 277	488 263	473 251	458 239	443 227	429 216	416 206	403 196	390 187
32LH12	27	32	[illegible]	[illegible]	[illegible]		[illegible] 450	[illegible] 428	[illegible] 406	[illegible] 384	[illegible] 364	[illegible] 345	[illegible] 327	[illegible] 311	[illegible] 295	[illegible] 281	[illegible] 267	[illegible] 255	[illegible] 243	[illegible] 232	[illegible] 221
32LH13	30	32	1048	40880	40880		817 500	801 480	785 461	771 444	742 420	715 397	690 376	666 354	643 336	621 319	600 304	581 288	562 275	544 262	527 249
32LH14	33	32	1079	42080	42080		843 515	826 495	810 476	795 456	780 440	766 417	738 395	713 374	688 355	665 337	643 321	622 304	602 290	583 276	564 264
32LH15	35	32	1115	43500	43500		870 532	853 511	837 492	821 473	805 454	791 438	776 422	763 407	750 393	725 374	701 355	678 338	656 322	635 306	616 292
			< 43	43-46	47-56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
36LH07	16	36	393	16900	16900		292 177	283 168	274 160	266 153	258 146	251 140	244 134	237 128	230 122	224 117	218 112	212 107	207 103	201 99	196 95
36LH08	18	36	433	18600	18600		321 194	311 185	302 176	293 168	284 160	278 153	268 146	260 140	253 134	246 128	239 123	233 118	227 113	221 109	215 104
36LH09	21	36	554	23840	23840		411 247	398 235	386 224	374 214	363 204	352 195	342 186	333 179	323 171	314 163	306 157	297 150	289 144	282 138	275 133
36LH10	21	36	611	26260	26260		454 273	440 260	426 248	413 236	401 225	389 215	378 206	367 197	357 188	347 180	338 173	328 165	320 159	311 152	303 146
36LH11	23	36	667	28660	28660		495 297	480 283	465 269	451 257	438 246	425 234	412 224	401 214	389 205	378 196	368 188	358 180	348 173	339 166	330 159
36LH12	25	36	798	34300	34300		593 354	575 338	557 322	540 307	523 292	508 279	493 267	478 255	464 243	450 232	437 222	424 213	412 204	400 195	389 187
36LH13	30	36	938	40340	40340		697 415	675 395	654 376	634 359	615 342	596 327	579 312	562 298	546 285	531 273	516 262	502 251	488 240	475 231	463 222
36LH14	36	36	1034	44460	44460		768 456	755 434	729 412	706 392	683 373	661 356	641 339	621 323	602 309	584 295	567 283	551 270	535 259	520 247	505 237
36LH15	36	36	1090	46880	46880		809 480	795 464	781 448	769 434	744 413	721 394	698 375	677 358	656 342	637 327	618 312	600 299	583 286	567 274	551 263

SHEET B6

VULCRAFT REF.

ASD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES

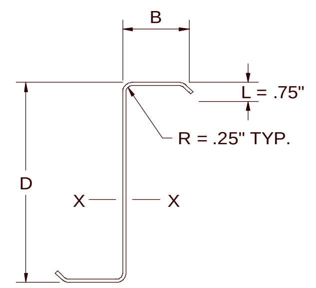
Based on a 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists Only)	Depth in inches	Max Load (plf)	SAFELOAD* in Lbs. Between		SPAN IN FEET														
			< 48	48-59	60-65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
40LH08	16	40	348	16680	16680	254 150	247 144	241 138	234 132	228 127	222 122	217 117	211 112	206 108	201 104	196 100	192 97	187 93	183 90	178 86
40LH09	21	40	457	21920	21920	332 196	323 188	315 180	306 173	298 166	291 160	283 153	276 147	269 141	263 136	256 131	250 126	244 122	239 118	233 113
40LH10	21	40	503	24120	24120	367 216	357 207	347 198	338 190	329 183	321 176	313 169	305 162	297 156	290 150	283 144	276 139	269 134	262 129	255 124
40LH11	22	40	549	26340	26340	399 234	388 224	378 215	368 207	358 198	349 190	340 183	332 176	323 169	315 163	308 157	300 151	293 145	286 140	279 135
40LH12	26	40	668	32060	32060	486 285	472 273	459 261	447 251	435 241	424 231	413 222	402 213	392 205	382 197	373 189	364 182	355 176	346 169	338 163
40LH13	30	40	788	37800	37800	573 [illegible]	557 [illegible]	542 [illegible]	528 [illegible]	514 [illegible]	500 [illegible]	487 [illegible]	475 [illegible]	463 [illegible]	451 [illegible]	440 [illegible]	429 [illegible]	419 [illegible]	409 [illegible]	399 [illegible]
40LH14	35	40	900	43220	43220	656 383	638 367	620 351	603 336	587 323	571 309	556 297	542 285	528 273	515 263	502 252	490 243	478 233	466 225	455 216
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible] 427	[illegible] 408	[illegible] 390	[illegible] 373	[illegible] 357	[illegible] 342	[illegible] 328	[illegible] 315	[illegible] 302	[illegible] 290	[illegible] 279	[illegible] 268	[illegible] 258	[illegible] 248	[illegible] 239
40LH16	42	40	1110	53280	53280	808 469	796 455	784 441	772 428	761 416	751 404	730 387	710 371	691 356	673 342	655 329	638 316	622 304	606 292	591 282
			< 53	53-59	60-73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88
44LH09	19	44	379	20100	20100	272 158	265 152	259 146	253 141	247 136	242 131	236 127	231 122	226 118	221 114	216 110	211 106	207 103	202 99	198 96
44LH10	21	44	419	22200	22200	300 174	293 168	286 162	279 156	272 150	266 144	260 139	254 134	249 130	243 125	238 121	233 117	228 113	223 110	218 106
44LH11	22	44	453	24000	24000	325 186	317 181	310 175	302 168	296 162	289 157	282 151	276 146	269 140	264 136	258 131	252 127	247 123	242 119	238 115
44LH12	25	44	561	29740	29740	402 232	393 224	383 215	374 207	365 200	356 192	347 185	339 179	331 172	323 166	315 160	308 155	300 149	293 144	287 139
44LH13	30	44	665	35260	35260	477 278	466 265	454 254	444 246	433 236	423 228	413 220	404 212	395 205	386 198	377 191	369 185	361 179	353 173	346 167
44LH14	31	44	786	40580	40580	549 315	534 302	520 291	506 279	493 268	481 259	469 249	457 240	446 231	436 223	425 216	415 207	406 200	396 193	387 187
44LH15	36	44	891	47220	47220	639 366	623 352	608 339	593 326	579 314	565 303	551 292	537 281	524 271	512 261	500 252	488 243	476 234	466 227	455 219
44LH16	42	44	1027	54440	54440	737 421	719 405	701 390	684 376	668 362	652 348	637 336	622 324	608 313	594 302	580 291	568 282	555 272	543 263	531 255
44LH17	47	44	1103	58460	58460	790 450	780 438	769 426	759 415	750 405	732 390	715 376	699 363	683 351	667 338	652 327	638 316	624 305	610 295	597 285
			< 57	57-59	60-81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96
48LH10	21	48	352	20080	20080	246 141	241 136	236 132	231 127	226 123	221 119	217 116	212 112	208 108	204 105	200 102	196 99	192 96	188 93	185 90
48LH11	22	48	382	21780	21780	266 152	260 147	255 142	249 137	244 133	239 129	234 125	229 120	225 117	220 113	216 110	212 106	208 103	204 100	200 97
48LH12	25	48	482	27500	27500	336 191	329 185	322 179	315 173	308 167	301 161	295 156	289 151	283 147	277 142	272 138	266 133	261 129	256 126	251 122
48LH13	29	48	578	32940	32940	402 228	393 221	384 213	376 206	368 199	360 193	353 187	345 180	338 175	332 170	325 164	318 160	312 154	306 150	300 145
48LH14	32	48	682	38860	38860	475 269	464 260	454 251	444 243	434 234	425 227	416 220	407 212	399 206	390 199	383 193	375 187	367 181	360 176	353 171
48LH15	36	48	784	44680	44680	545 308	533 298	521 287	510 279	499 269	488 260	478 252	468 244	458 238	448 228	439 221	430 214	422 208	413 201	406 196
48LH16	42	48	904	51500	51500	629 355	615 343	601 331	588 320	576 310	563 299	551 289	540 280	528 271	518 263	507 255	497 247	487 239	477 232	468 226
48LH17	47	48	1015	57840	57840	706 397	690 383	675 371	660 358	646 346	632 335	619 324	606 314	593 304	581 294	569 285	558 276	547 268	536 260	526 252

SJI STEEL JOIST INSTITUTE

SHEET B7

FLEXOSPAN Z PURLIN AND GIRT SECTION PROPERTIES



SECTION PROPERTIES (ZEES)

SECTION				AXIS X-X			AXIS Y-Y		
D X B	GA	WEIGHT LB/LF	AREA IN^2	Ix IN^4	Sx IN^3	Rx IN	Iy IN^4	Sy IN^3	Ry IN
6 X 2.5 Z	16	2.4534	0.72158	4.1433	1.3811	2.3962	1.237	0.4123	1.3093
	14	3.0522	0.89769	5.1248	1.7083	2.3893	1.5253	0.5096	1.3035
	12	4.2387	1.2467	7.0351	2.345	2.3755	2.0806	0.6987	1.2919
8 X 2.5 Z	16	2.86	0.84118	8.0884	2.0221	3.1009	1.237	0.4123	1.2127
	14	3.5601	1.0471	10.021	2.5053	3.0936	1.525	0.5096	1.207
	12	4.95	1.4559	13.802	3.4506	3.079	2.081	0.6987	1.1955
8 X 3.5 Z	14	4.0681	1.1965	12.367	3.0918	3.215	3.528	0.8836	1.7172
	12	5.6613	1.6651	17.063	4.2657	3.2011	4.843	1.2175	1.7055
10 X 2.5 Z	14	4.0681	1.1965	17.012	3.4023	3.7707	1.525	0.5097	1.1291
	12	5.6613	1.6651	23.481	4.6963	3.7553	2.081	0.6988	1.1179
10 X 3.5 Z	14	4.576	1.3459	20.691	4.1382	3.9209	3.528	0.8836	1.6191
	12	6.3726	1.8743	28.603	5.7205	3.9065	4.844	1.2176	1.6076
12 X 2.5 Z	14	4.576	1.3459	26.395	4.3992	4.4285	1.525	0.5097	1.0646
	12	6.3726	1.8743	36.491	6.0817	4.4124	2.081	0.6989	1.0538
12 X 3.5 Z	14	5.084	1.4953	31.707	5.2845	4.6048	3.528	0.8837	1.5361
	12	7.0839	2.0835	43.891	7.3152	4.5898	4.844	1.2176	1.5247

Flexospan - 253 Railroad St. Sandy Lake, PA 16145 - 1-800-245-0396

FLEXOSPAN - CEE AND ZEE LOAD TABLES

Allowable Uniform Loads In Pounds Per Lineal Foot

CEE		Simple Span					
		16 Gauge		14 Gauge		12 Gauge	
Section	Bay	2 1/2" Flan	3 1/2" Flan	2 1/2" Flan	3 1/2" Flan	2 1/2" Flan	3 1/2" Flan
6" Web	10 ft	251	-	331	-	480	-
	12 ft	174	-	230	-	333	-
	14 ft	128	-	169	-	244	-
	15 ft	111	-	147	-	213	-
	18 ft	77	-	102	-	148	-
	20 ft	62	-	82	-	120	-
	22 ft	51	-	68	-	99	-
	24 ft	43	-	57	-	83	-
	25 ft	40	-	53	-	76	-
	28 ft	32	-	42	-	61	-
8" Web	12 ft	260	269	341	365	493	545
	14 ft	191	198	250	268	362	400
	15 ft	166	172	218	233	315	349
	18 ft	115	119	151	162	219	242
	20 ft	93	97	122	131	177	196
	22 ft	77	80	101	108	146	162
	24 ft	65	67	85	91	123	136
	25 ft	60	62	78	84	113	125
	28 ft	47	49	62	67	90	100
	30 ft	41	43	54	58	78	87
10" Web	20 ft	115	-	168	173	243	266
	22 ft	95	-	139	143	200	220
	24 ft	80	-	116	120	168	185
	25 ft	74	-	107	111	155	170
	28 ft	59	-	85	88	124	136
	30 ft	51	-	74	77	108	118
	32 ft	45	-	65	67	94	104
	34 ft	40	-	58	60	84	92
	35 ft	37	-	54	56	79	87
	38 ft	32	-	46	48	67	73
12" Web	20 ft	-	-	185	206	293	344
	24 ft	-	-	128	143	203	239
	25 ft	-	-	118	132	187	220
	28 ft	-	-	94	105	149	175
	30 ft	-	-	82	91	130	153
	32 ft	-	-	72	80	114	134
	34 ft	-	-	64	71	101	119
	35 ft	-	-	60	67	95	112
	38 ft	-	-	51	57	81	95
	40 ft	-	-	46	51	73	86

ZEE		Simple Span						3 or More Spans, Std. Lap					
		16 Gauge		14 Gauge		12 Gauge		16 Gauge		14 Gauge		12 Gauge	
Section	Bay	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.	2 1/2" Fl.	3 1/2" Fl.
6" Web	10 ft	254	-	331	-	499	-	-	-	-	-	-	-
	12 ft	126	-	230	-	346	-	-	-	-	-	-	-
	14 ft	129	-	169	-	254	-	-	-	-	-	-	-
	15 ft	113	-	147	-	221	-	176	-	235	-	357	-
	18 ft	78	-	102	-	154	-	118	-	157	-	238	-
	20 ft	63	-	82	-	124	-	94	-	124	-	188	-
	22 ft	52	-	68	-	103	-	76	-	101	-	153	-
	24 ft	44	-	57	-	86	-	63	-	84	-	126	-
	25 ft	40	-	53	-	79	-	58	-	76	-	116	-
	28 ft	32	-	42	-	63	-	46	-	60	-	91	-
8" Web	12 ft	260	265	340	364	510	545	-	-	-	-	-	-
	14 ft	191	195	250	267	374	401	-	-	-	-	-	-
	15 ft	166	169	218	233	326	349	-	-	-	-	-	-
	18 ft	115	117	151	161	228	242	156	158	222	235	345	368
	20 ft	93	95	122	131	183	196	127	129	178	188	275	293
	22 ft	77	78	101	108	151	162	105	107	145	154	223	238
	24 ft	65	66	85	91	127	136	88	90	121	129	185	198
	25 ft	59	61	78	83	117	125	81	83	111	118	170	181
	28 ft	47	48	62	66	93	100	65	66	87	93	133	142
	30 ft	41	42	54	58	81	87	56	57	76	81	115	123
10" Web	20 ft	115	119	168	-	250	269	131	133	217	-	365	387
	24 ft	80	82	116	-	173	185	96	97	154	-	248	263
	25 ft	74	76	107	-	160	170	89	91	142	-	228	242
	28 ft	59	60	85	-	127	136	72	74	114	-	179	191
	30 ft	51	52	74	-	111	118	64	65	99	-	155	165
	32 ft	45	46	65	-	97	104	56	58	87	-	135	144
	35 ft	37	38	54	-	81	87	48	49	73	-	112	119
	38 ft	32	33	46	-	69	73	41	42	61	-	94	101
12" Web	20 ft	-	-	183	203	301	345	-	-	210	220	418	463
	24 ft	-	-	127	141	209	239	-	-	153	162	289	324
	25 ft	-	-	117	130	192	220	-	-	142	151	265	299
	28 ft	-	-	93	103	153	176	-	-	116	124	210	238
	30 ft	-	-	81	90	133	153	-	-	102	109	183	207
	32 ft	-	-	71	79	117	134	-	-	90	97	160	181
	35 ft	-	-	59	66	98	112	-	-	76	82	133	151
	37 ft	-	-	53	59	88	100	-	-	68	74	119	135
	40 ft	-	-	45	50	75	86	-	-	59	64	101	115

Notes: 1.The weight of the section has not been subtracted from these values. 2. Both flanges of member must be fully braced. 3. These loads are based on the transfer of the support loads directly to the web of the section by the use of clips or plates. For flanges bearing directly on structural, contact factory for section selection. 4. See back page for weights per lineal foot of memebers shown here. 5. Loads shown are stress governing. When deflection limits are specified, contact factory. 6. These sample calculations are very basic. Many different variables can affect loading. For instance; drift loading, building height, geographic location, etc. Please consult Flexospan if special conditions exist. 7. The selection of sections for your application is subject to final approval by your design professional. 8. Capacity values have been calculated in accordance with the AISI 2001 design manual. 9. Values shown in the load tables for three or more spans are based on uniform bay spacings. If non-uniform bay spacings exist, contact factory. UNCONTROLLED COPY

PLB™-36 or HSB®-36

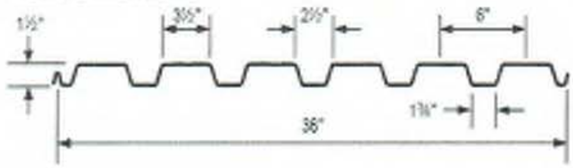
The difference between the PLB™-36 profile and HSB®-36 profile is the method of sidelap attachment; the panels themselves are identical in both geometry and material properties. The prefix "PL" designates a PunchLok ® II version of the B profile, while "HS" (high shear) indicates a top seam weld, button punch, or screw version of the same profile.

Type B profiles are 1.5-inch deep structural roof deck that provide both vertical load and diaphragm shear capacity. The profile contains 6 ribs and is 36 inches wide with male and female edges, creating an interlocking side lap when installed. The wide ribs make the profile an ideal structural substrate to uniformly support roofing systems applied on top of the deck. Type B profiles are typically used for span conditions of 10 feet or less.

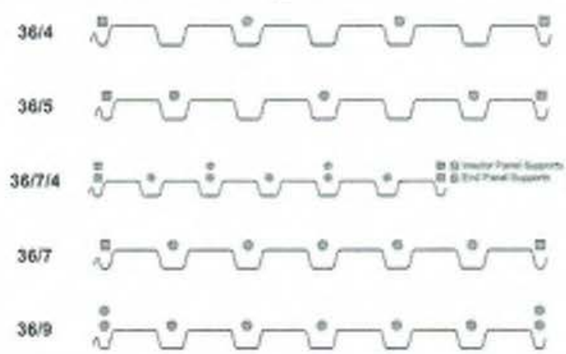
Extensive full scale diaphragm testing is an ongoing effort with B deck to produce a more efficient roof diaphragm in terms of capacity and installation. The current industry use of mechanical fasteners (Hilti and Pneutek), restraining elements (ShearTranz® Systems) and the innovative PunchLok® II side lap attachment system are all direct results of testing.

CAD DRAWINGS

Profile Dimensions



Attachment Patterns to Supports



Section Properties

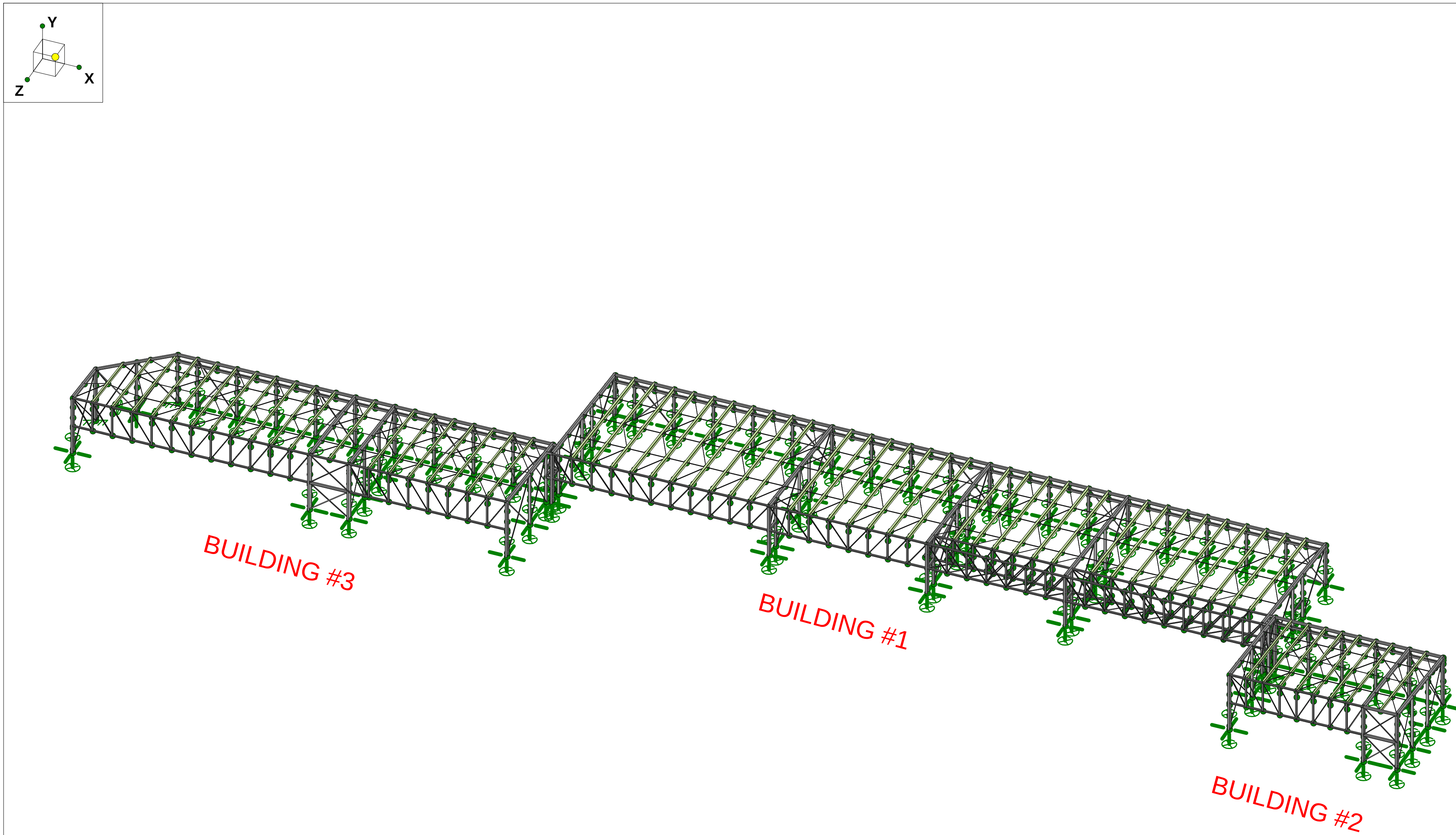
Deck Gage	Deck Weight Galv G60 (psf)	Deck Weight Phos/ Painted (psf)	I_d for Deflection Single Span (in^4/ft)	I_d for Deflection Multiple Spans (in^4/ft)	Moment $+S_{eff}$ (in^3/ft)	Moment $-S_{eff}$ (in^3/ft)
22	1.9	1.8	0.177	0.192	0.176	0.188
20	2.3	2.2	0.219	0.231	0.230	0.237
18	2.9	2.8	0.302	0.306	0.314	0.331
16	3.5	3.4	0.381	0.381	0.399	0.410

NOTE: Section properties based on F_y = 50 ksi.

Allowable Reactions

Deck Gage	End Bearing 2"	End Bearing 3"	End Bearing 4"	Interior Bearing 3"	Interior Bearing 4"
22	935	1076	1163	1559	1671
20	1301	1492	1609	2190	2340
18	2181	2484	2667	3714	3950
16	3265	3699	3955	5607	5938

NOTE: Allowable reactions are in pounds per foot of deck width and are based on F_y = 50 ksi.



ROOF PURLINS & DECKING ASSUMED A "FLEXIBLE"
DIAPHRAGM, NOT MODELED FOR STRUCTURAL DEMANDS

Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 2:31 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

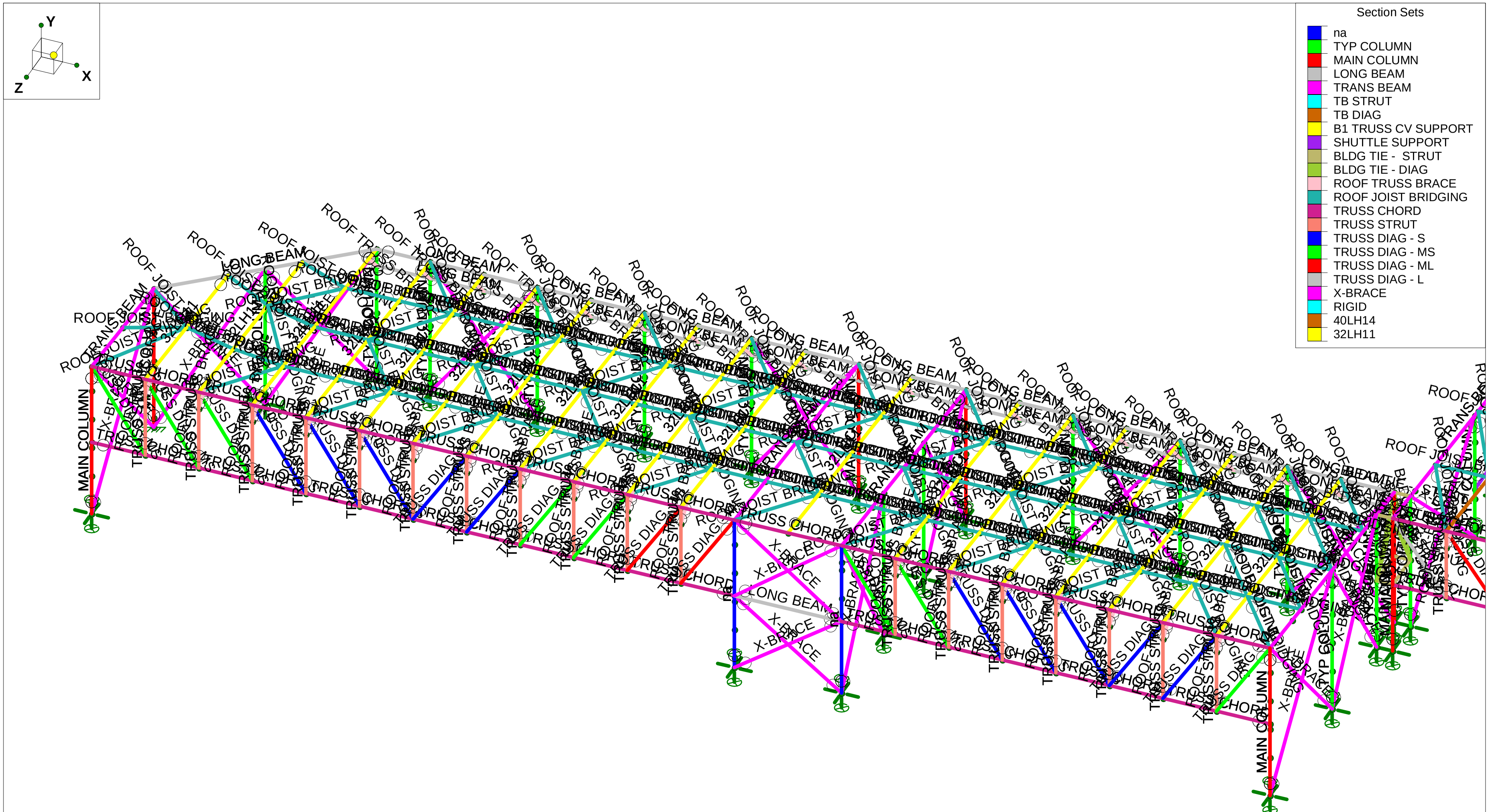
SHEET B11

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design ...	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	TYP COLUMN	W12X65	Column	Wide Flange	A572 Gr....	Typical	19.1	174	533	2.18
2	MAIN COLUMN	W24X104	Column	Wide Flange	A572 Gr....	Typical	30.7	259	3100	4.72
3	LONG BEAM	W12X65	Beam	Wide Flange	A572 Gr....	Typical	19.1	174	533	2.18
4	TRANS BEAM	W24X104	Beam	Wide Flange	A572 Gr....	Typical	30.7	259	3100	4.72
5	TB STRUT	L4X3X4	HBrace	Single Angle	A36 Gr.36	Typical	1.69	1.33	2.75	.039
6	TB DIAG	L4X3X5	HBrace	Single Angle	A36 Gr.36	Typical	2.09	1.62	3.36	.073
7	B1 TRUSS CV SUPPO...	W6X12	Beam	Wide Flange	A572 Gr....	Typical	3.55	2.99	22.1	.09
8	SHUTTLE SUPPORT	W12X40	Beam	Wide Flange	A572 Gr....	Typical	11.7	44.1	307	.906
9	B3 TALL COLUMN	W36X194	Column	Wide Flange	A572 Gr....	Typical	57	375	12100	22.2
10	BLDG TIE - STRUT	W12X40	Beam	Wide Flange	A572 Gr....	Typical	11.7	44.1	307	.906
11	BLDG TIE - DIAG	L4X3X4	VBrace	Single Angle	A36 Gr.36	Typical	1.69	1.33	2.75	.039
12	ROOF TRUSS BRACE	LL4x3x4x6	VBrace	Double Angle (3/...	A36 Gr.36	Typical	3.38	6.72	5.5	.077
13	ROOF JOIST BRIDGING	L2x2x3	HBrace	Single Angle	A36 WT...	Typical	.722	.271	.271	.009
14	TRUSS CHORD	W8X31	Beam	Wide Flange	A572 Gr....	Typical	9.13	37.1	110	.536
15	TRUSS STRUT	W8X31	Column	Wide Flange	A572 Gr....	Typical	9.13	37.1	110	.536
16	TRUSS DIAG - S	LL3x2x4x6	VBrace	Double Angle (3/...	A36 Gr.36	Typical	2.4	2.55	2.18	.054
17	TRUSS DIAG - MS	LL4x3x4x6	VBrace	Double Angle (3/...	A36 Gr.36	Typical	3.38	6.72	5.5	.077
18	TRUSS DIAG - ML	LL4x3x5x6	VBrace	Double Angle (3/...	A36 Gr.36	Typical	4.18	8.55	6.72	.146
19	TRUSS DIAG - L	LL4x3.5x6x6	VBrace	Double Angle (3/...	A36 Gr.36	Typical	5.36	15.3	8.3	.264
20	X-BRACE	L4X3X4	VBrace	Single Angle	A36 WT...	Typical	1.69	1.33	2.75	.039

General Section Sets

	Label	Shape	Type	Material	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	RIGID		None	RIGID	1e+6	1e+6	1e+6	1e+6
2	40LH14	40LH14	Beam	gen_Steel	10.3	10	2570	10
3	32LH11	32LH11	Beam	gen_Steel	7.1	10	1142	10



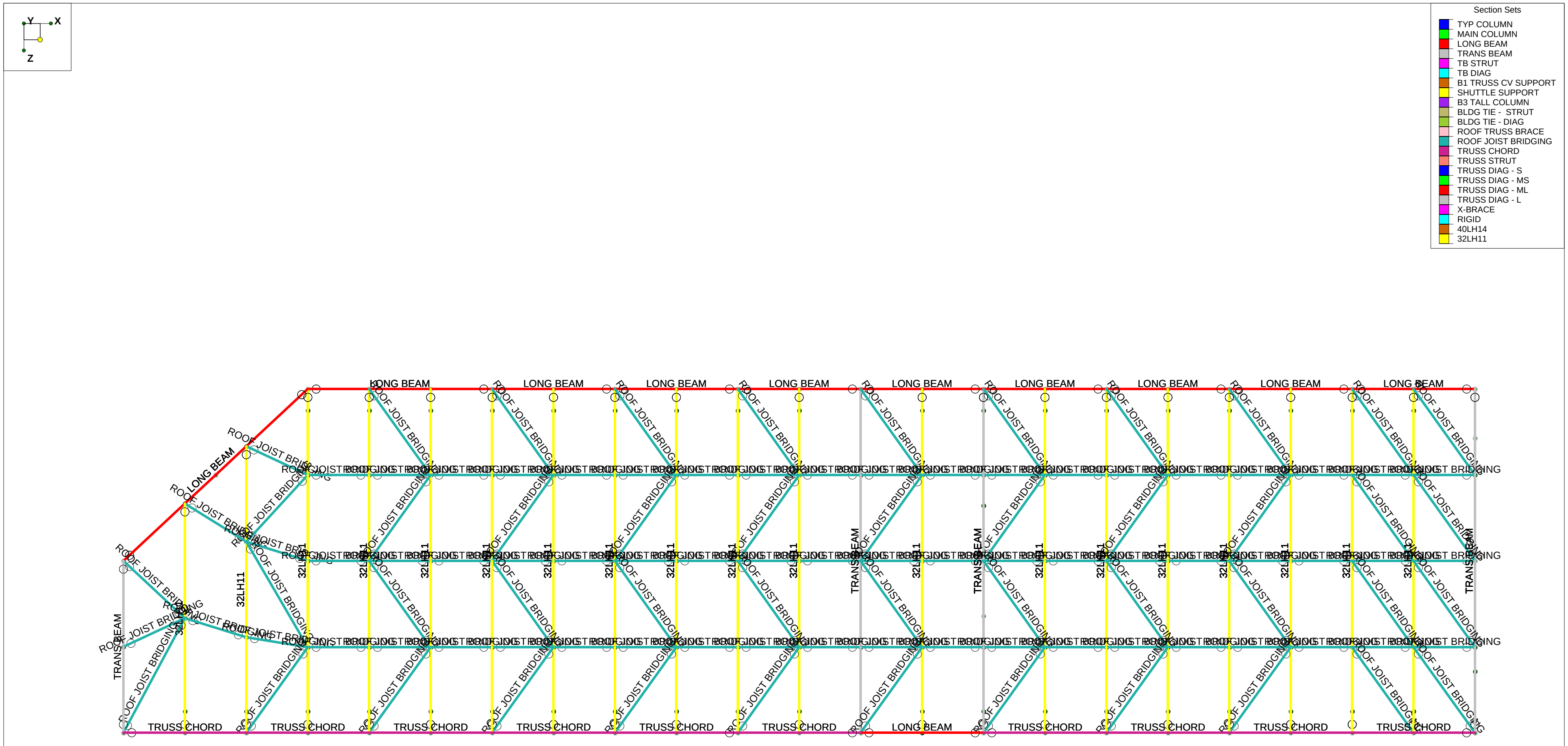
Section Sets

- na
- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 3:00 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B14



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
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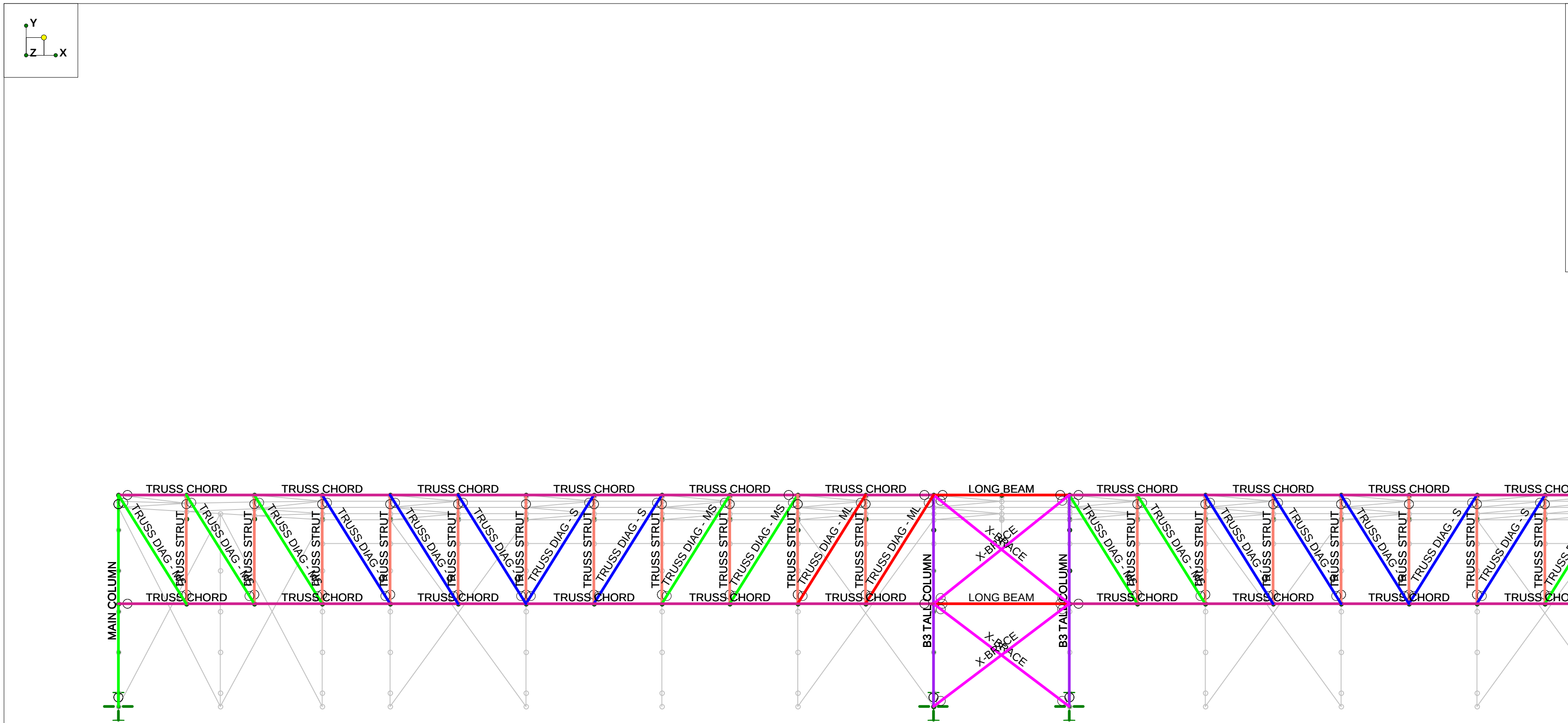
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B15



Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
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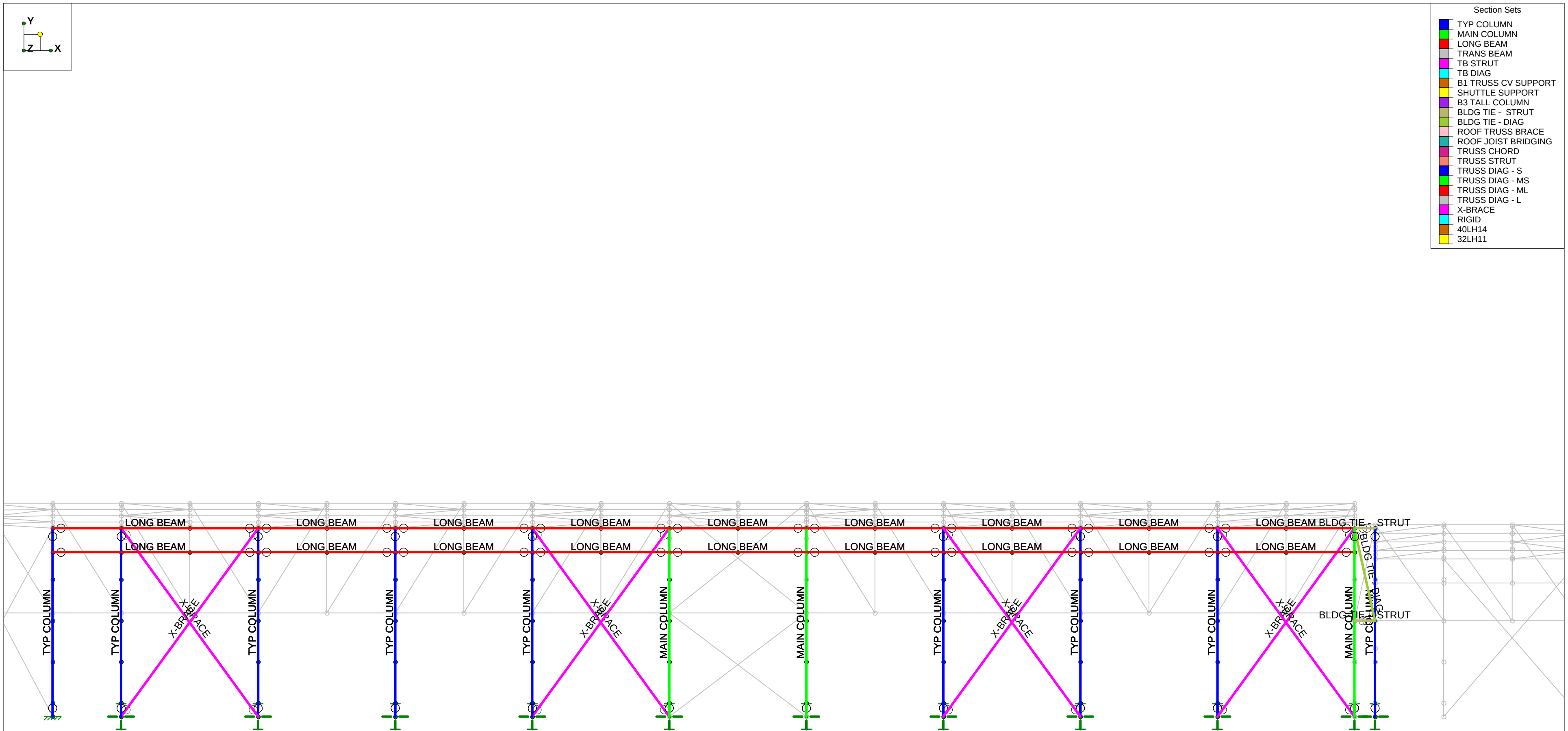
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



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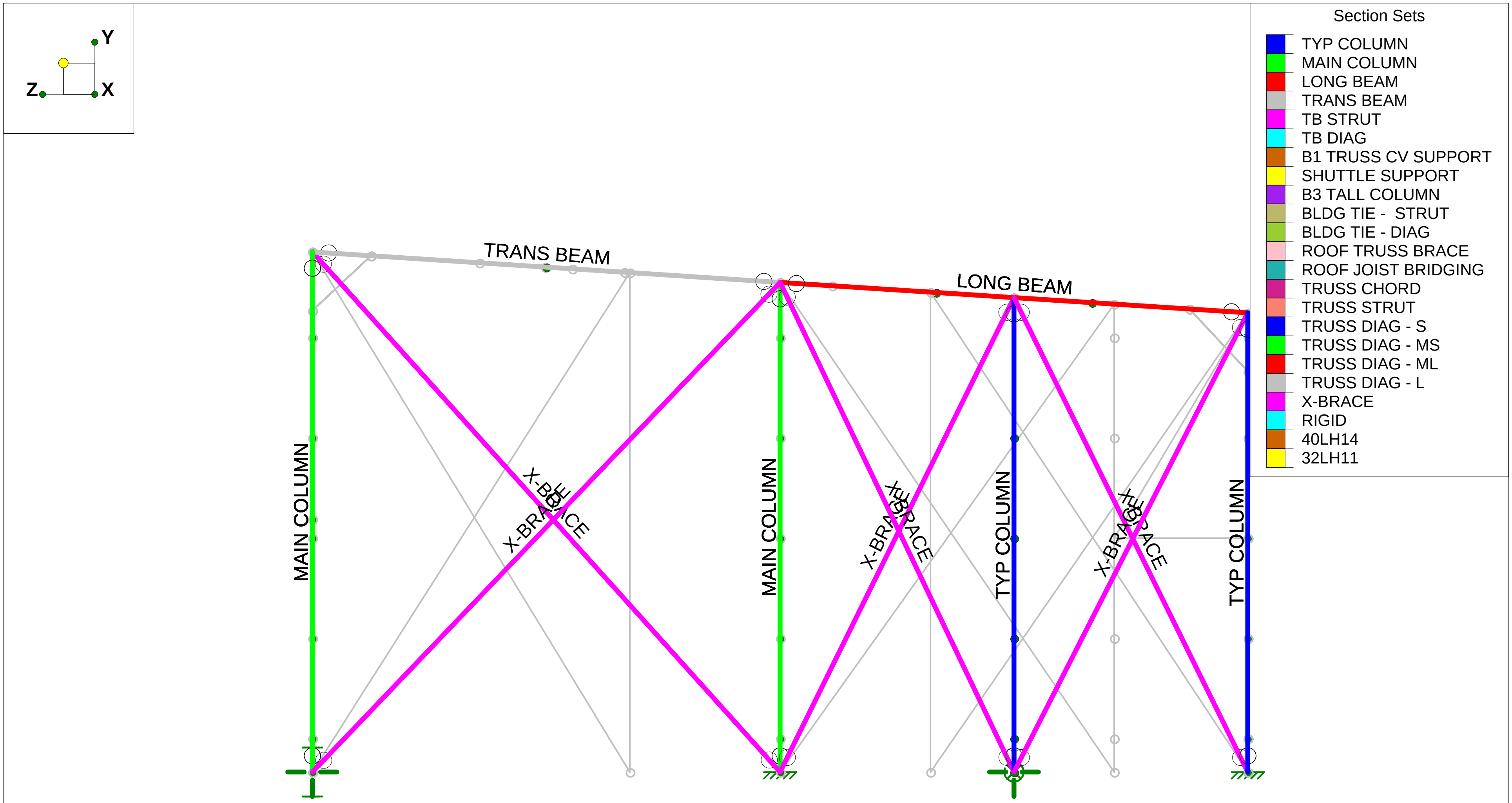
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B17



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
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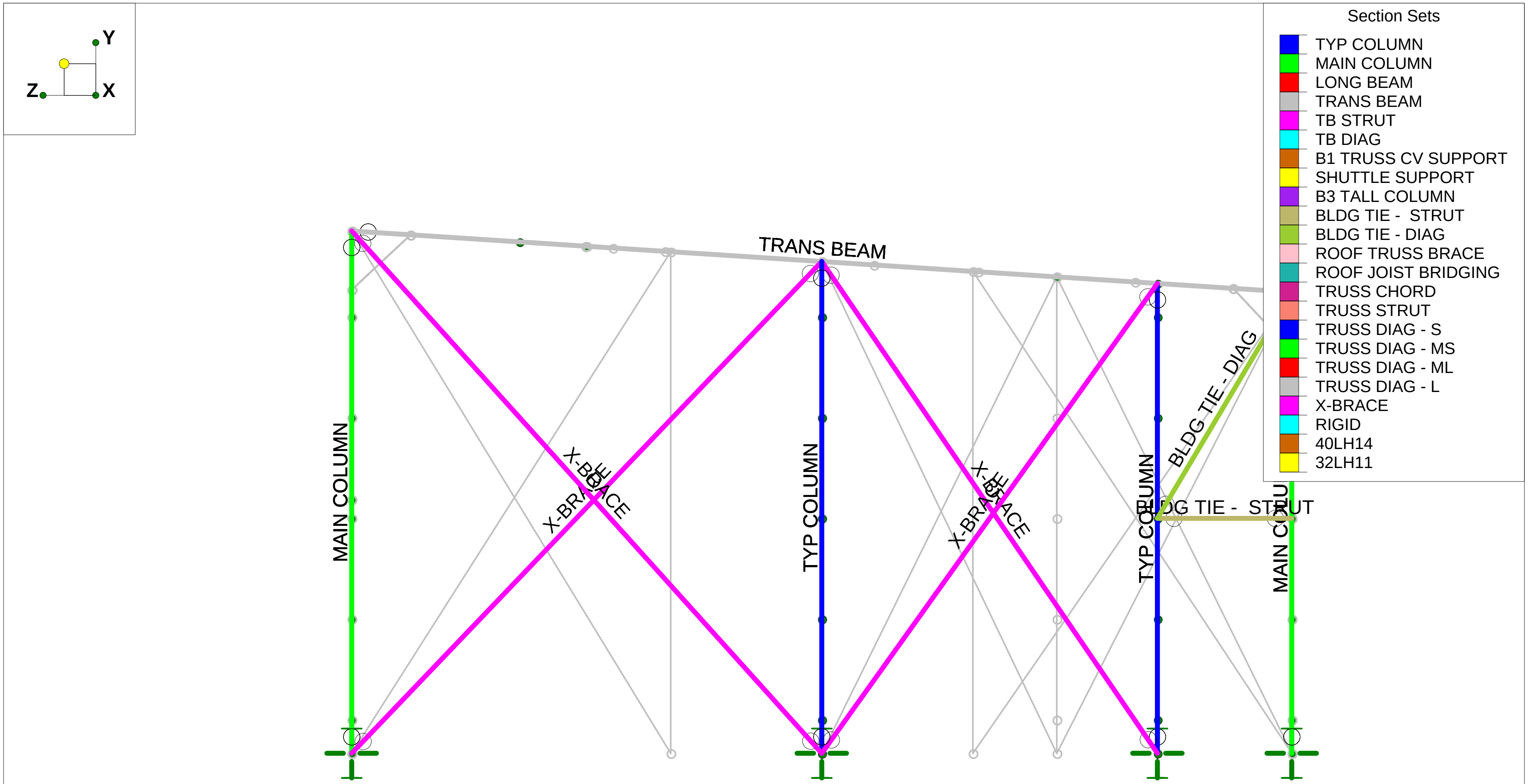
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B18



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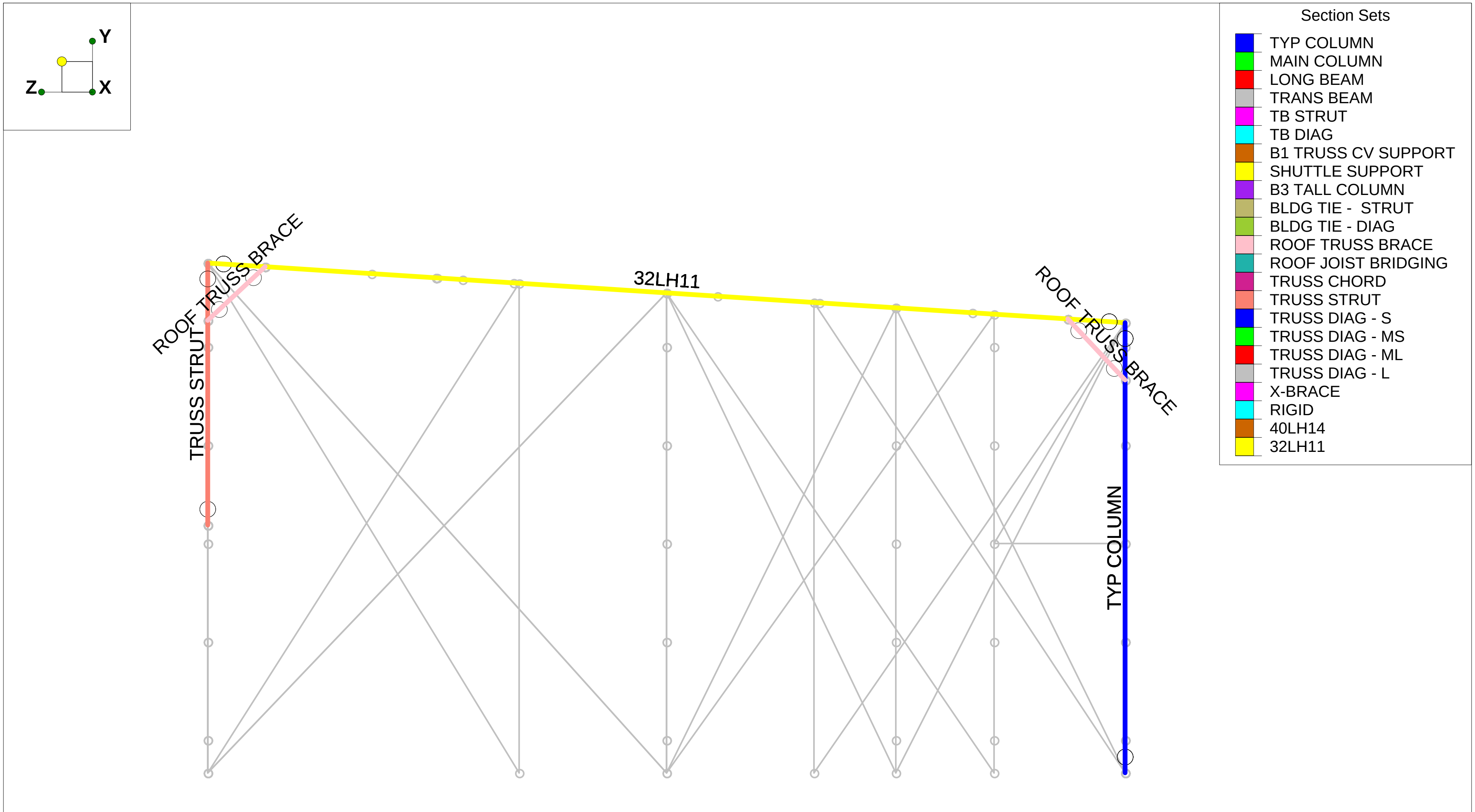
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B19



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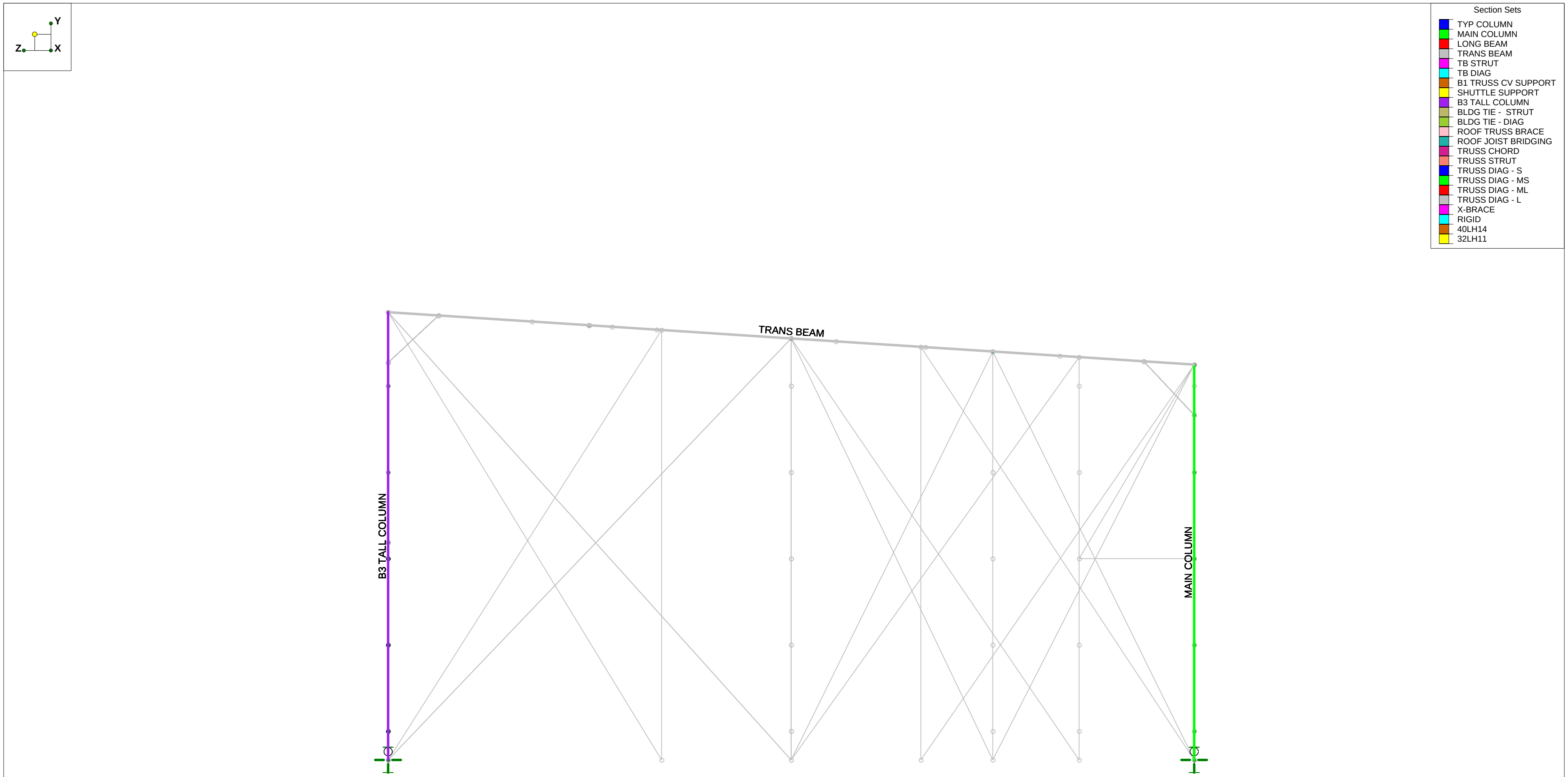
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B20



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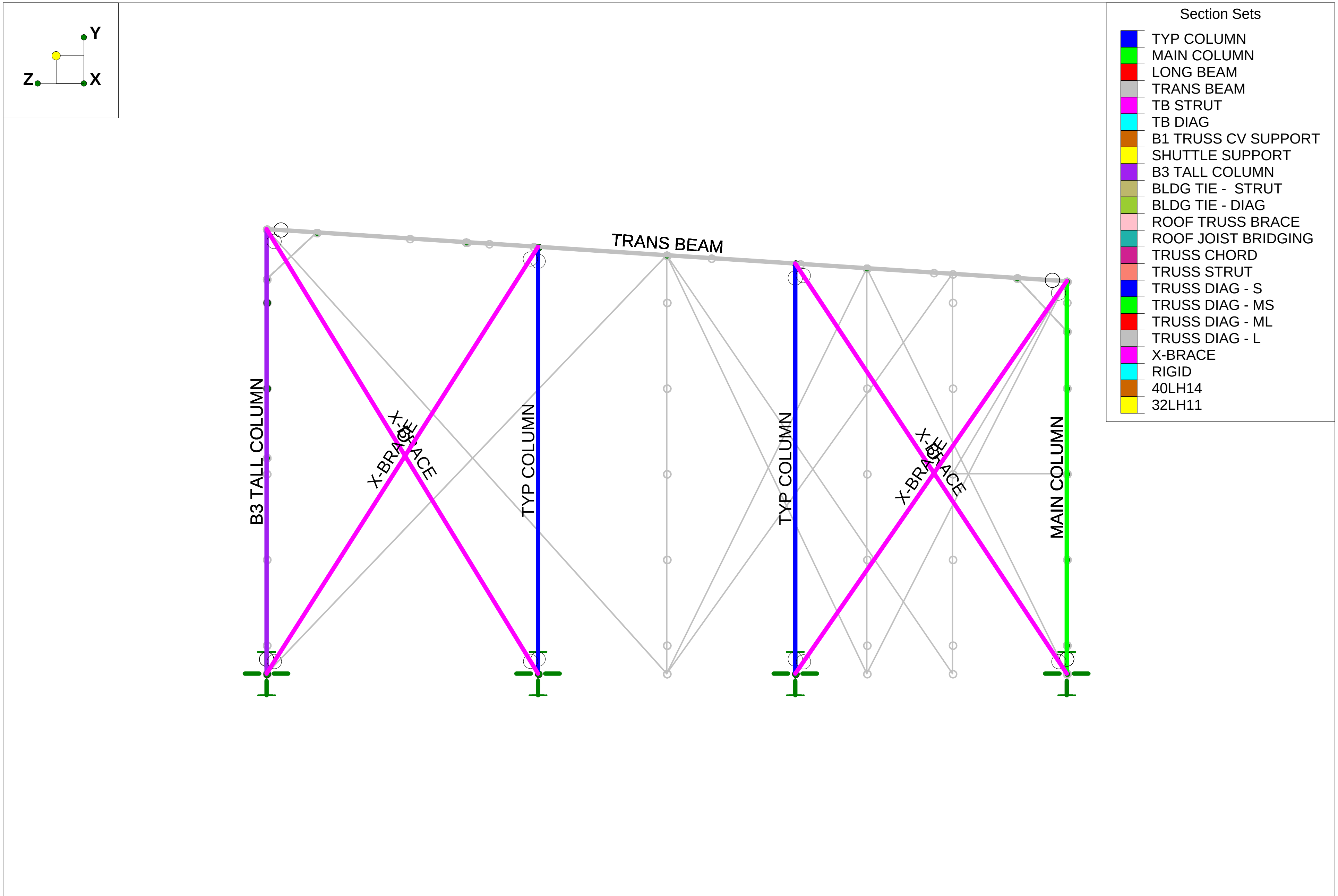

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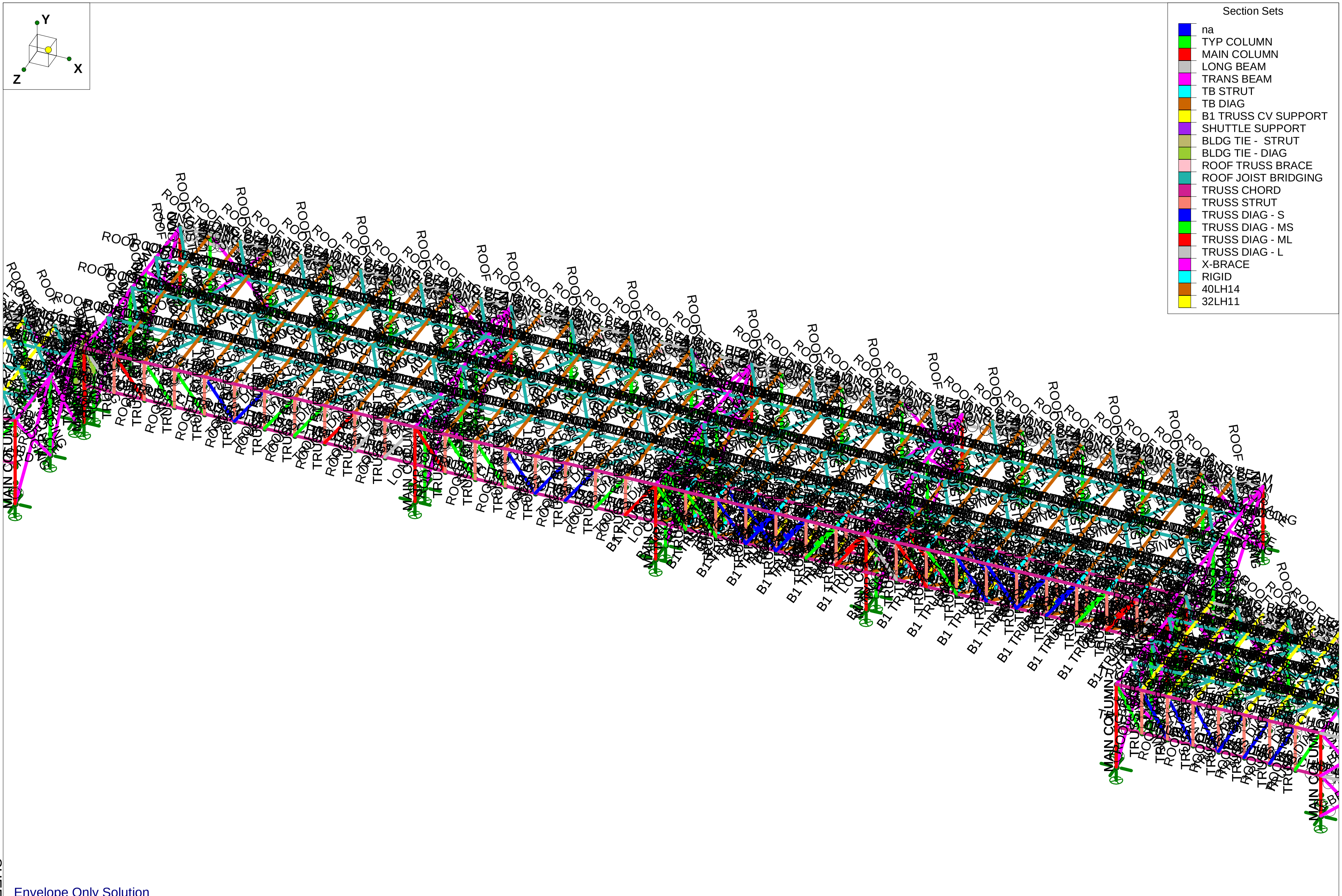
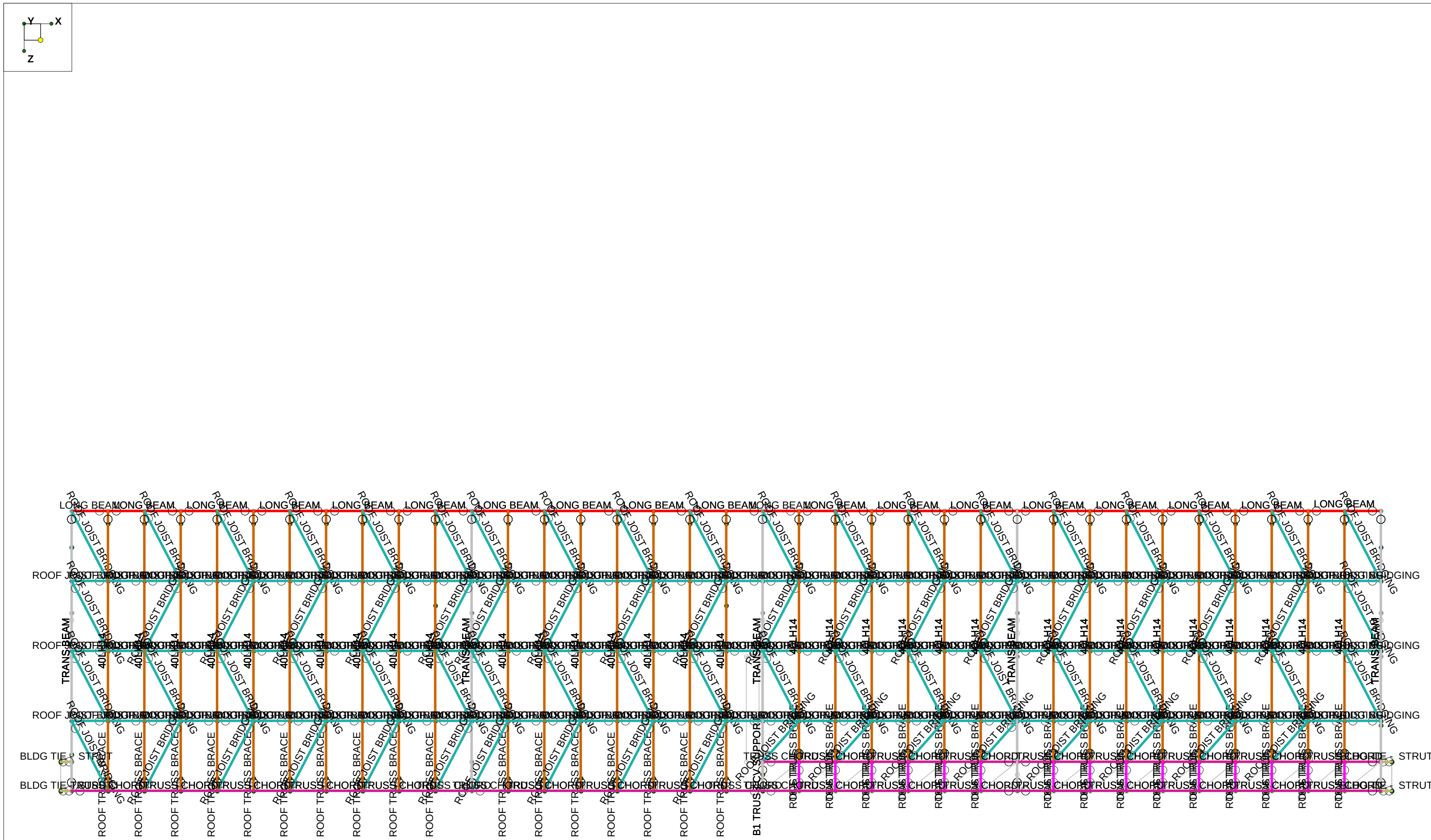
SHEET B22



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 3:00 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
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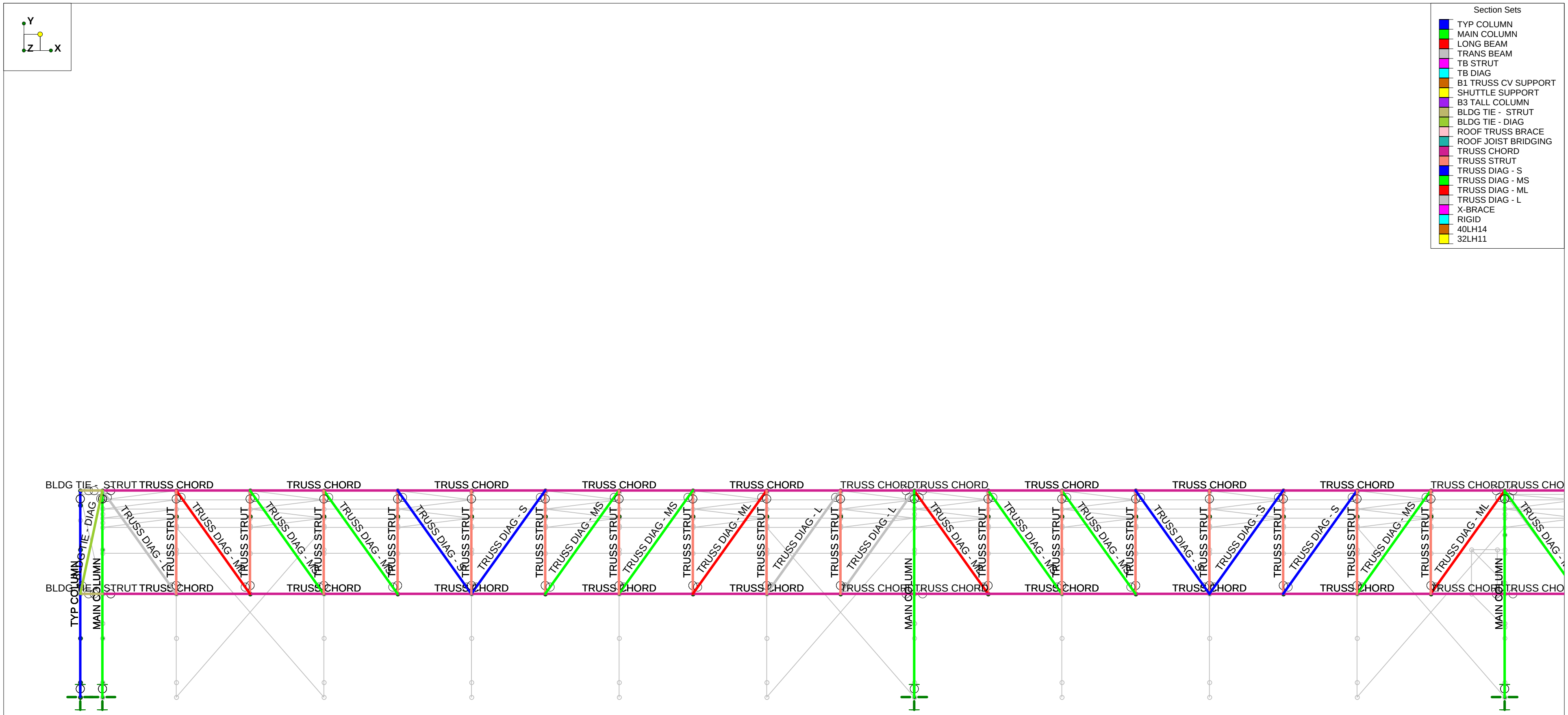
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

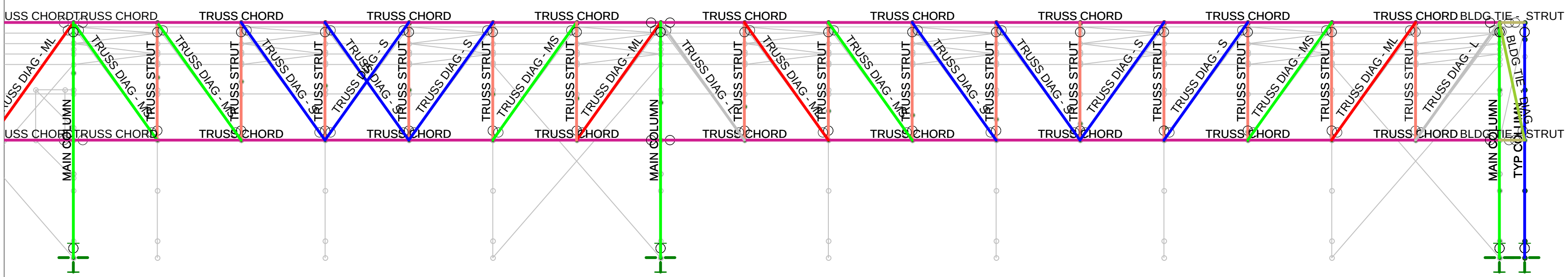


Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

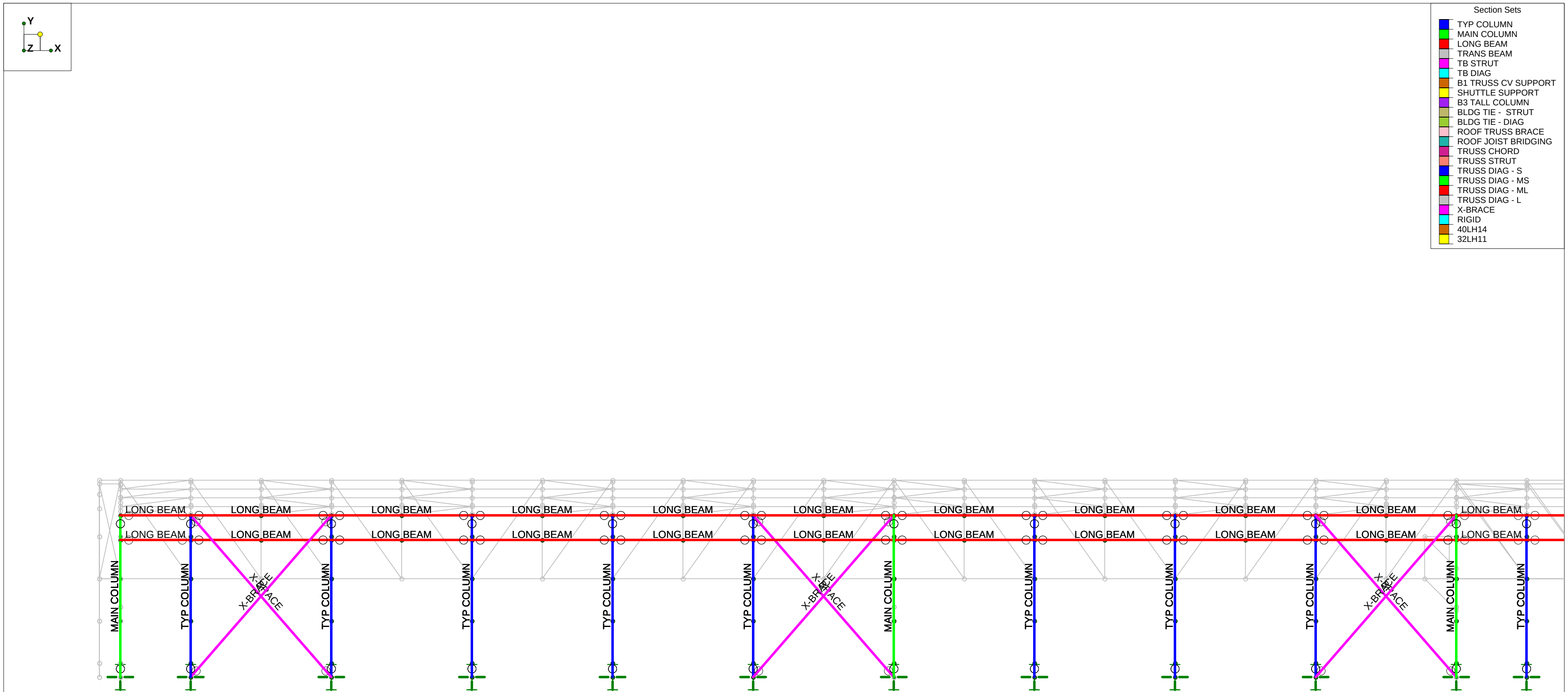
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:21 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B25

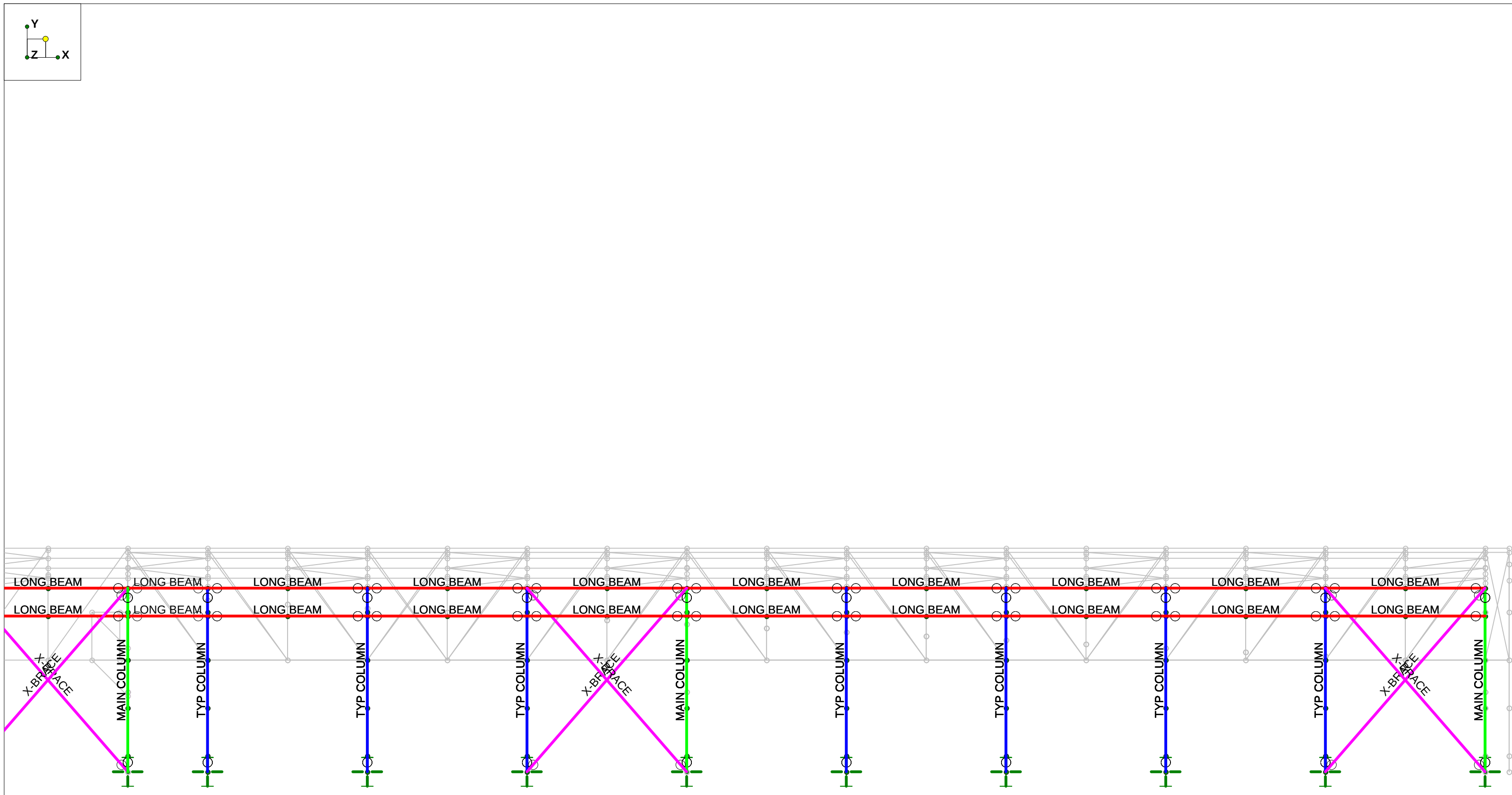


Section Sets
- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:22 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:23 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

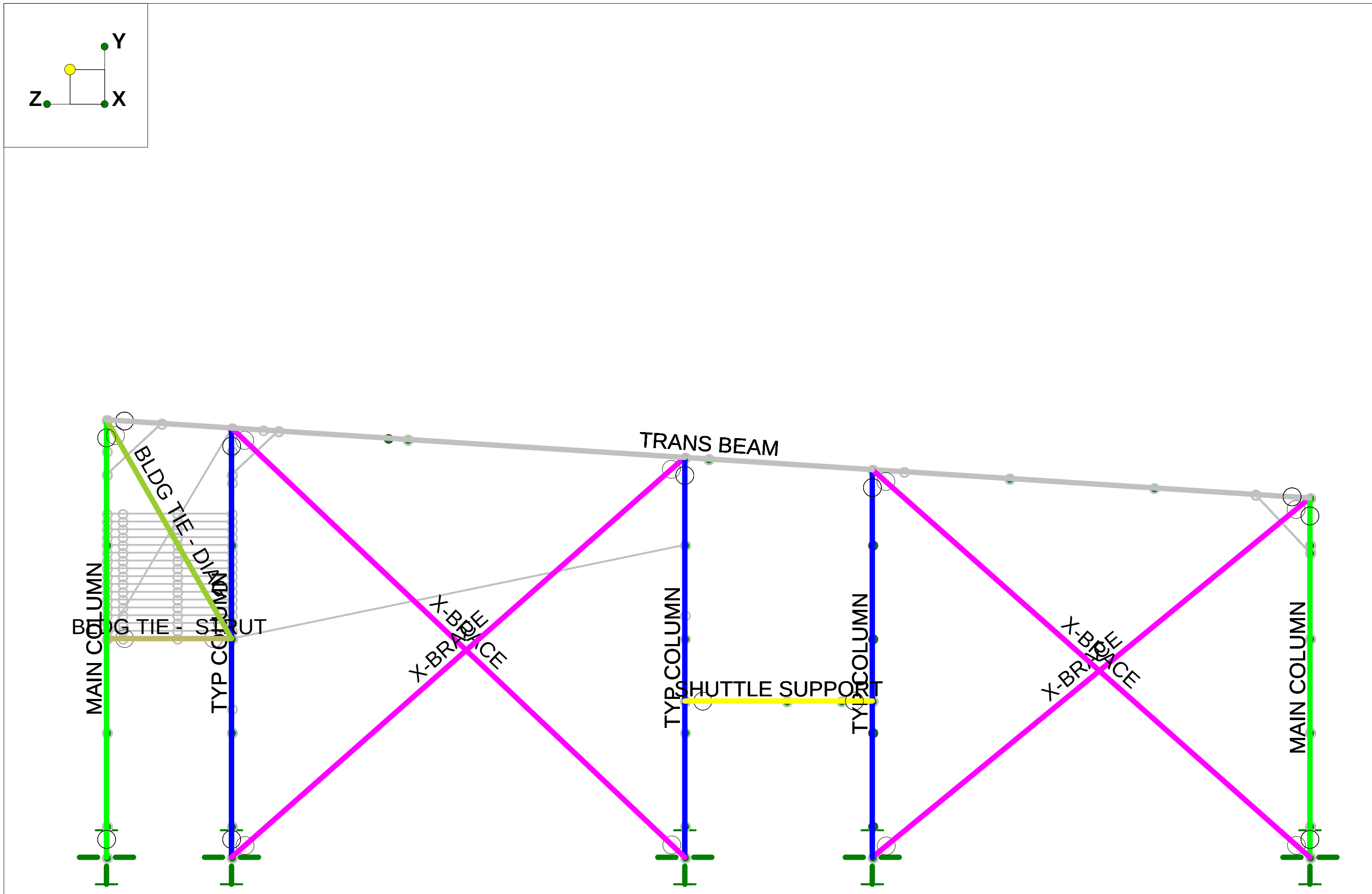


Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:23 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B28

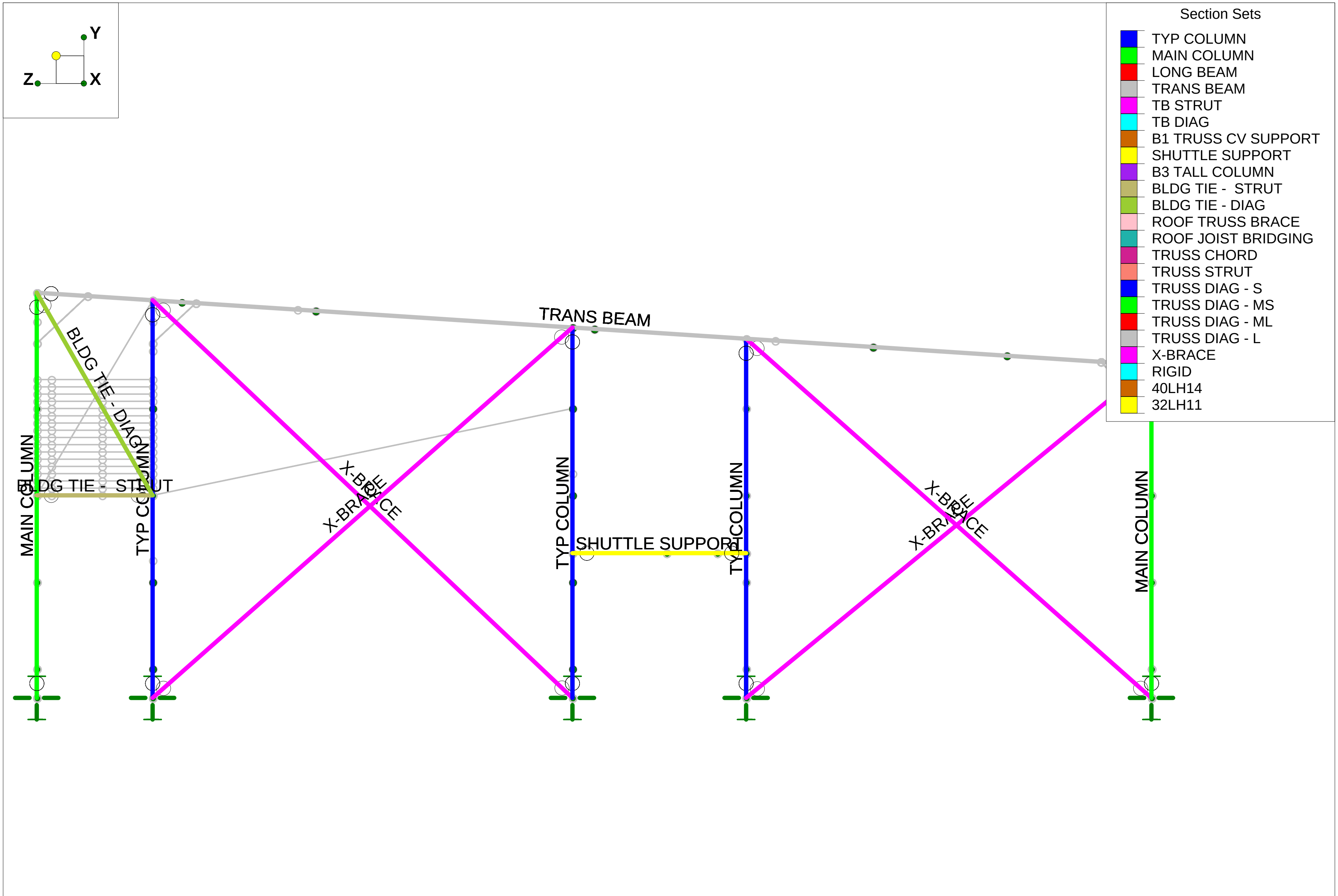


Section Sets

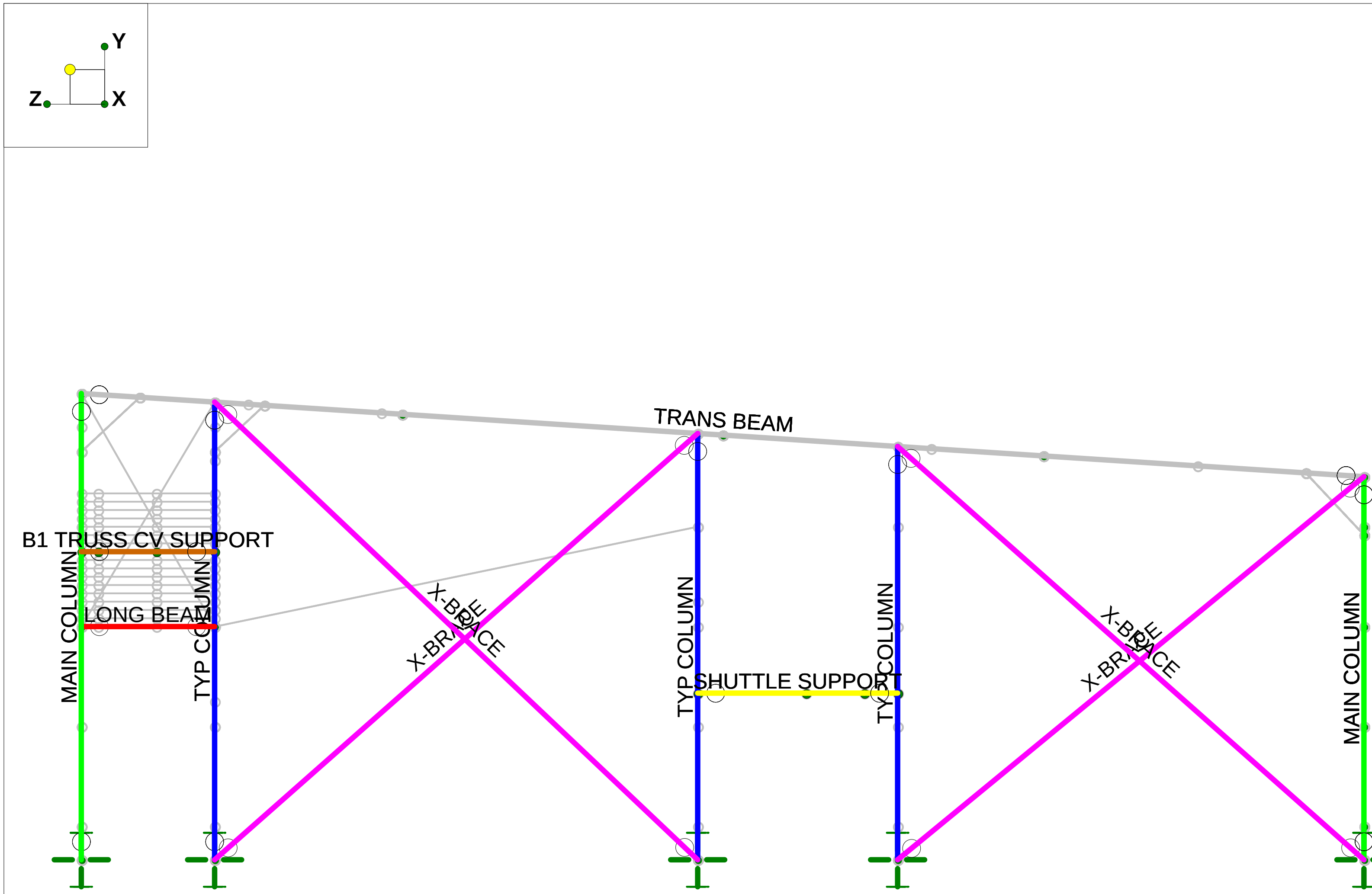
- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:24 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B29



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:29 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

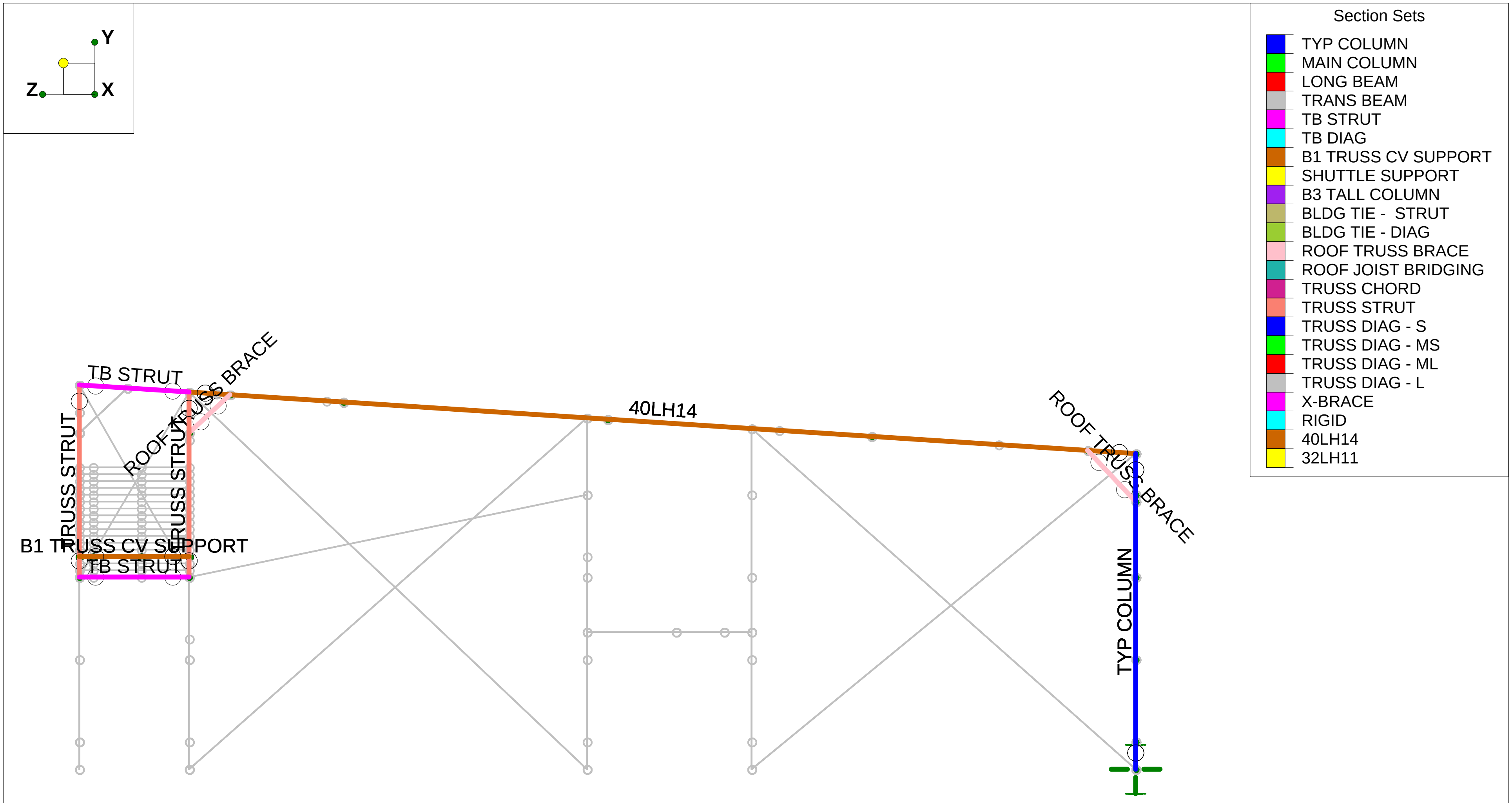


Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

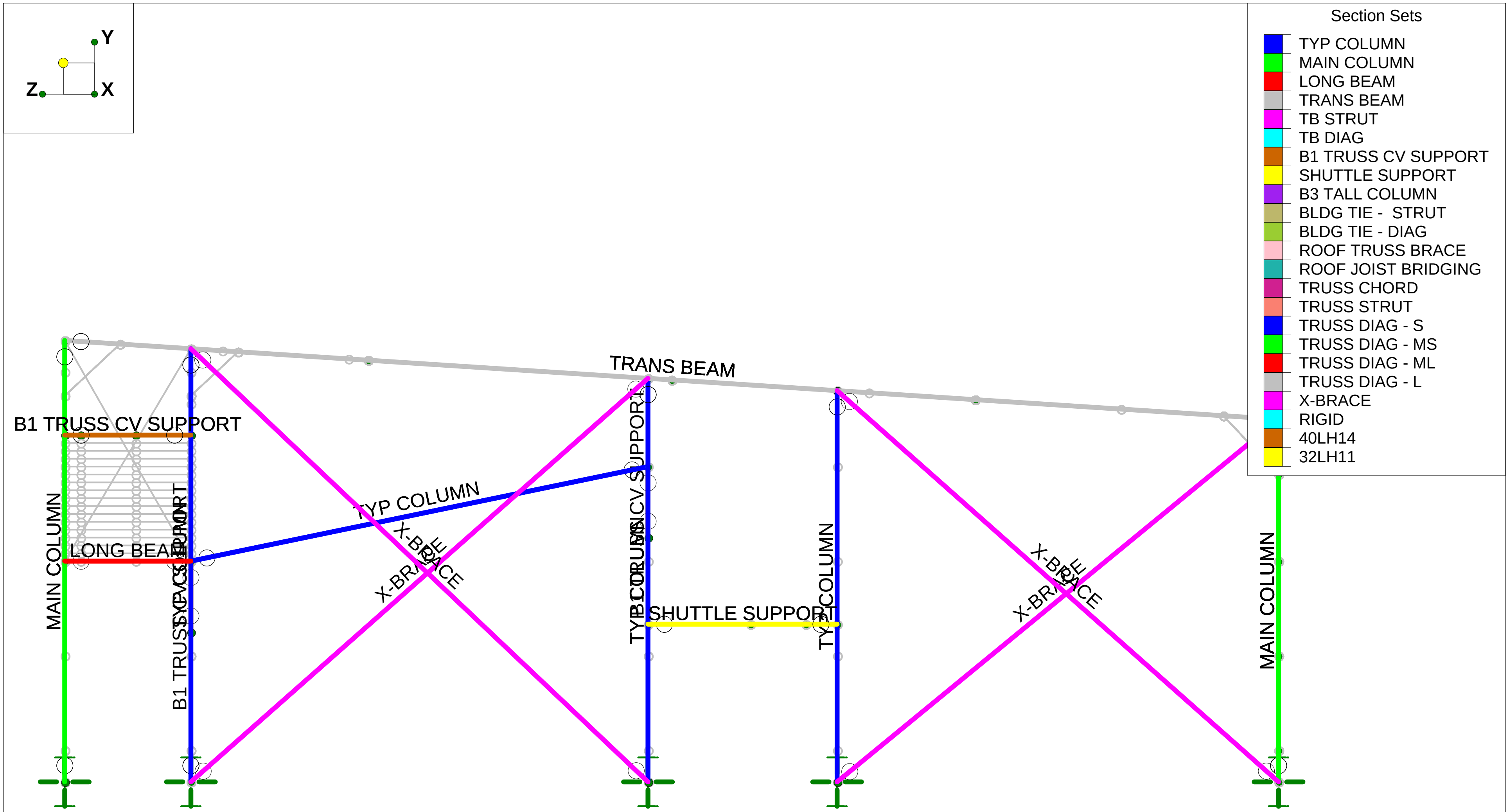
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:25 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B31



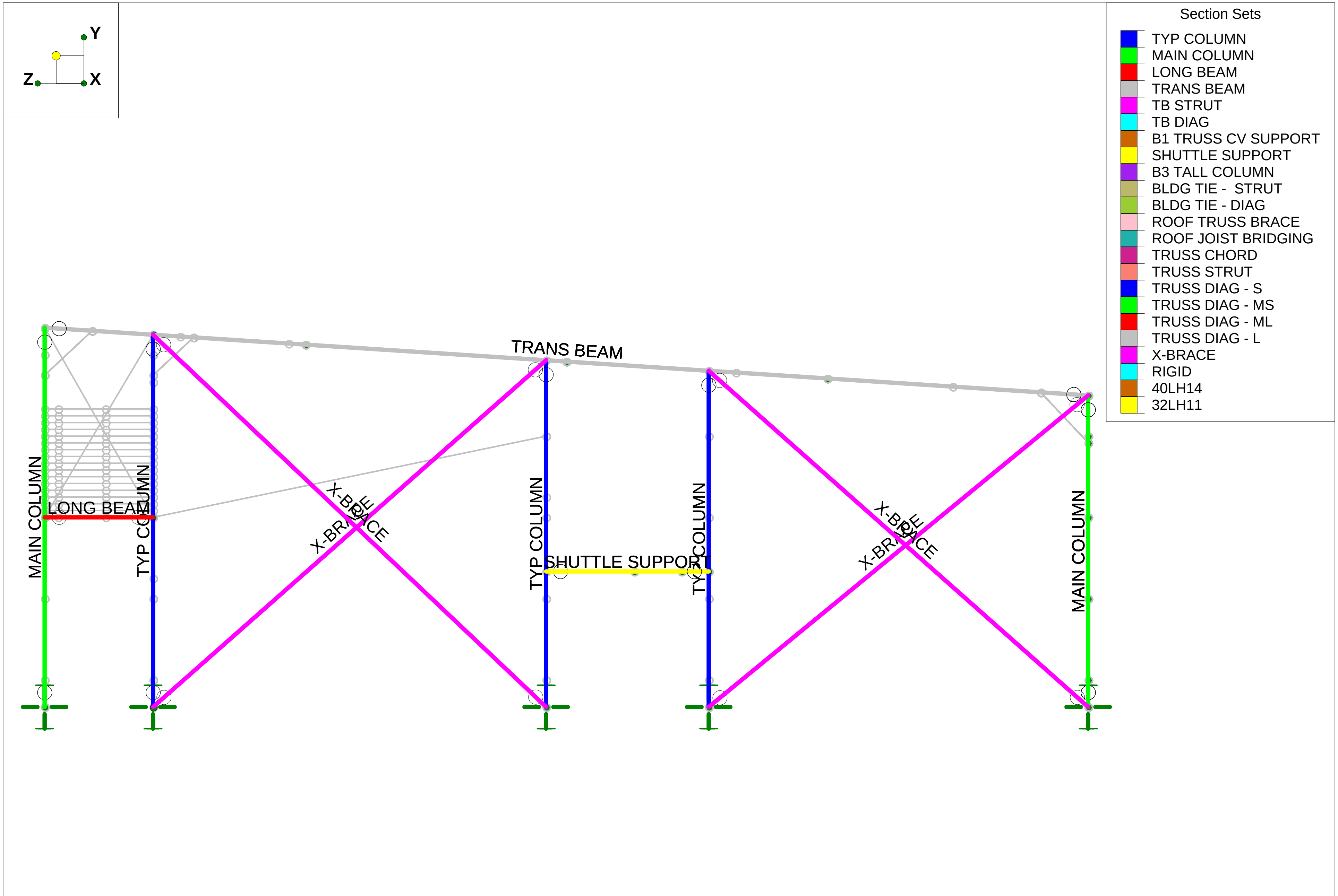
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:26 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B32



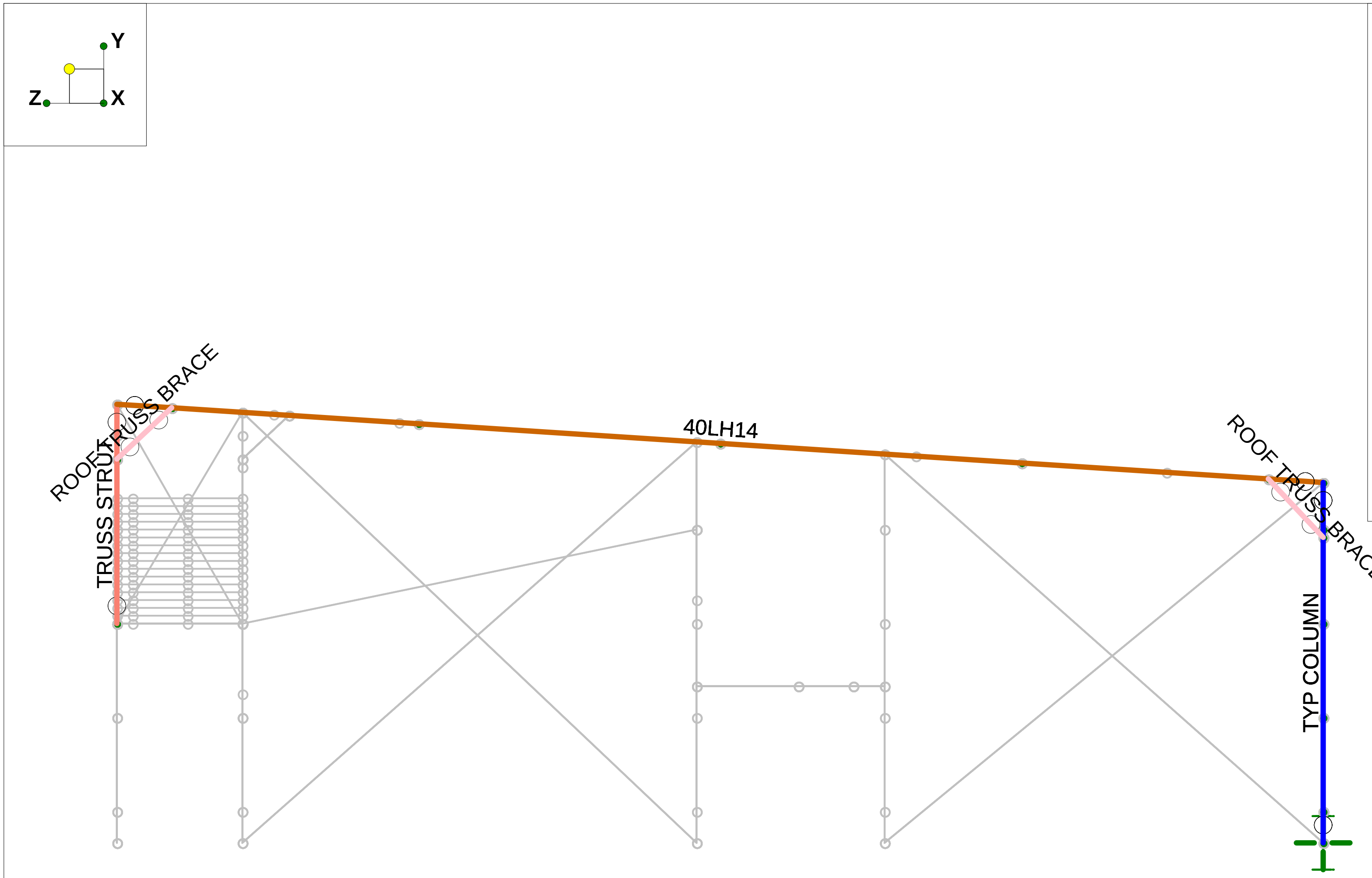
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:27 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B33



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:28 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

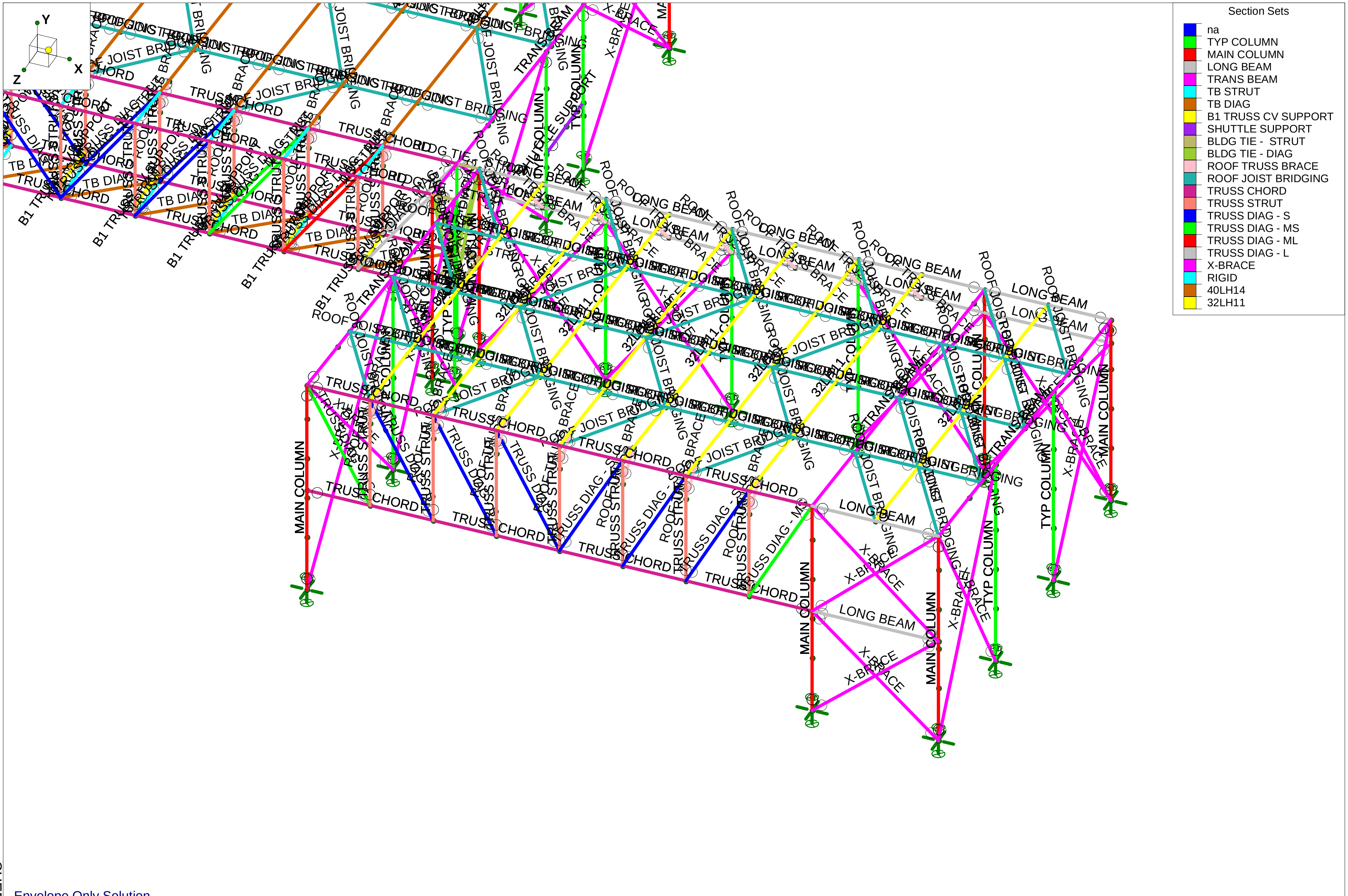
SHEET B34



Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

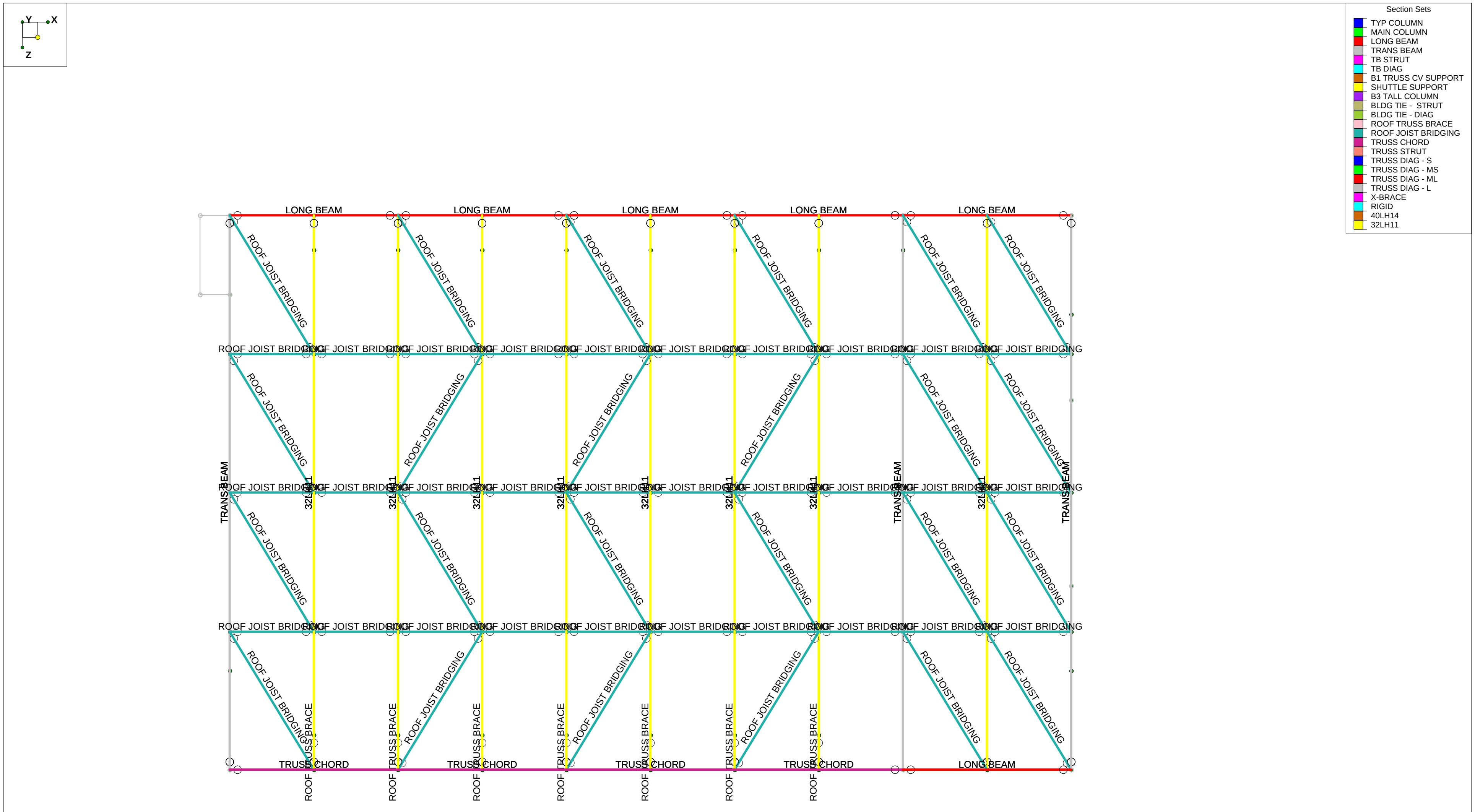
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:29 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



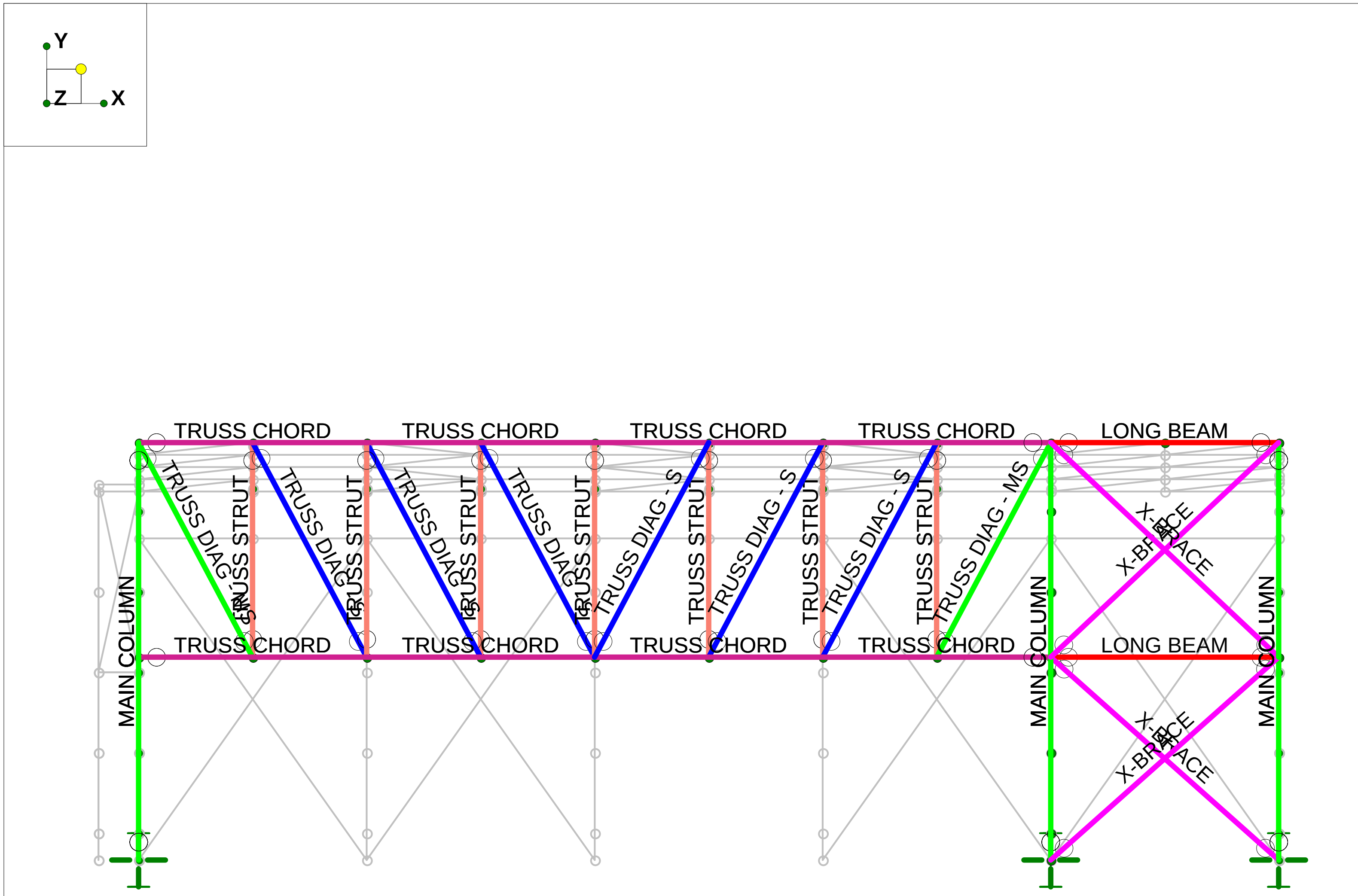
Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 2:59 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B36



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:33 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

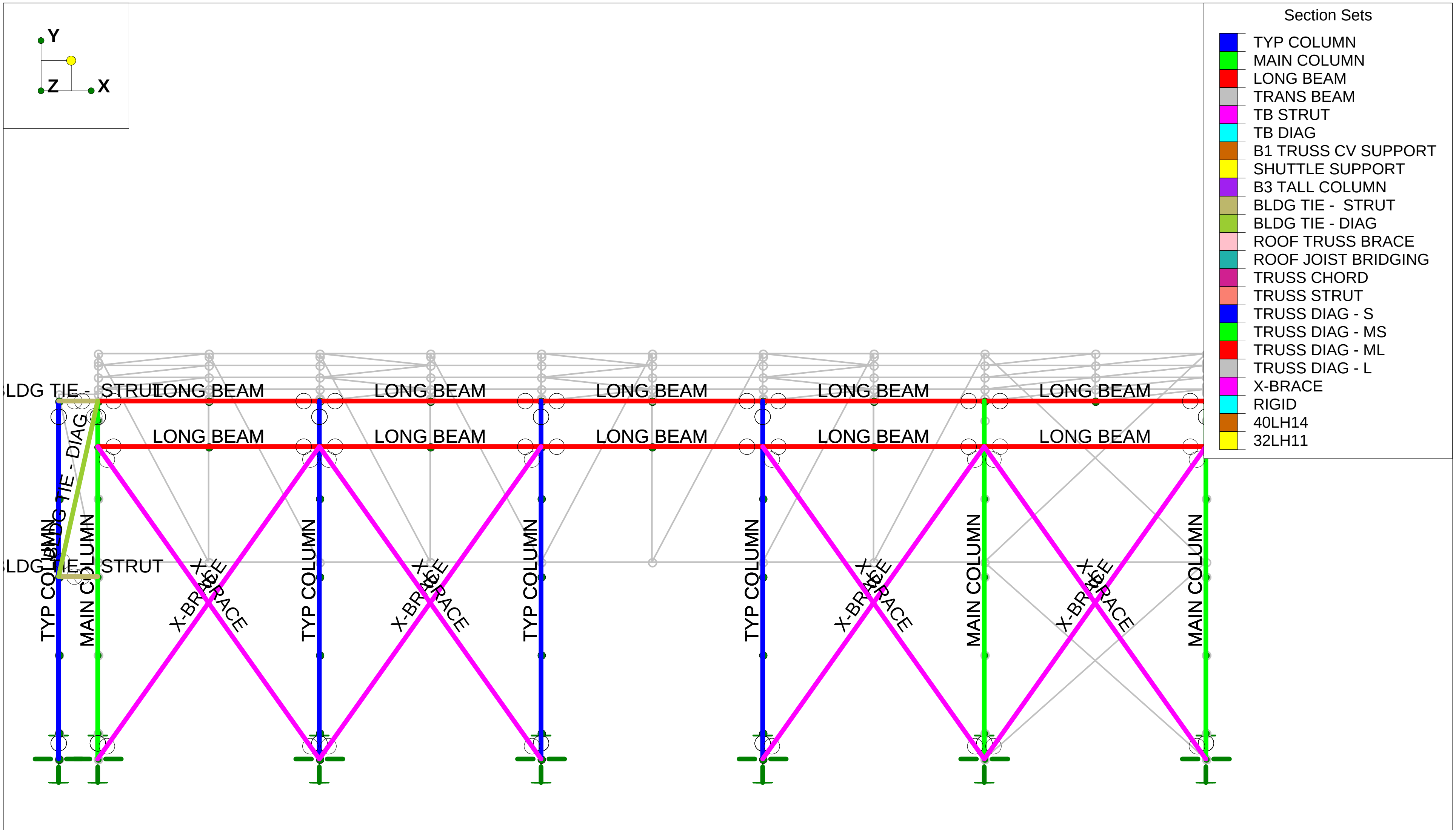


Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

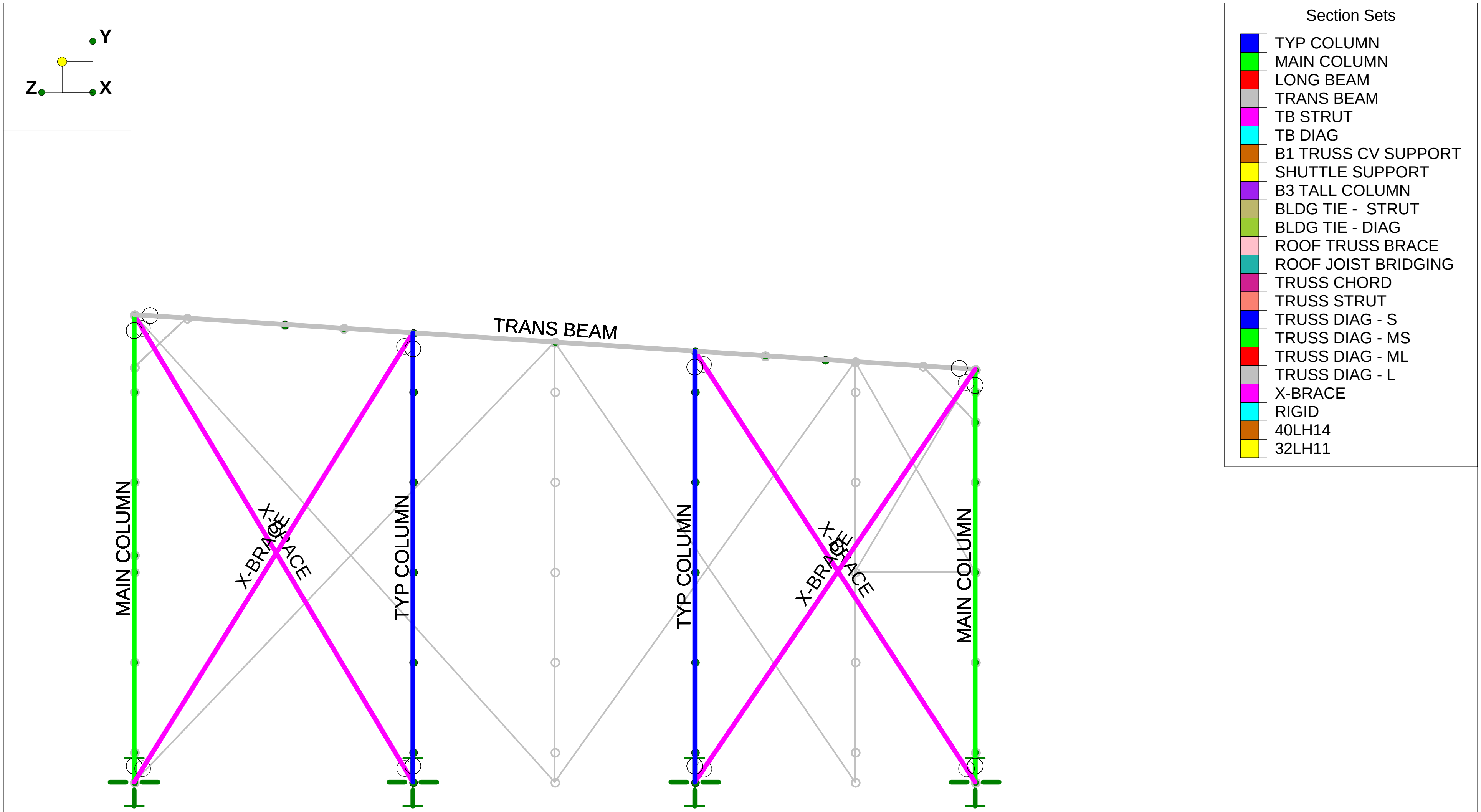
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:32 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B38



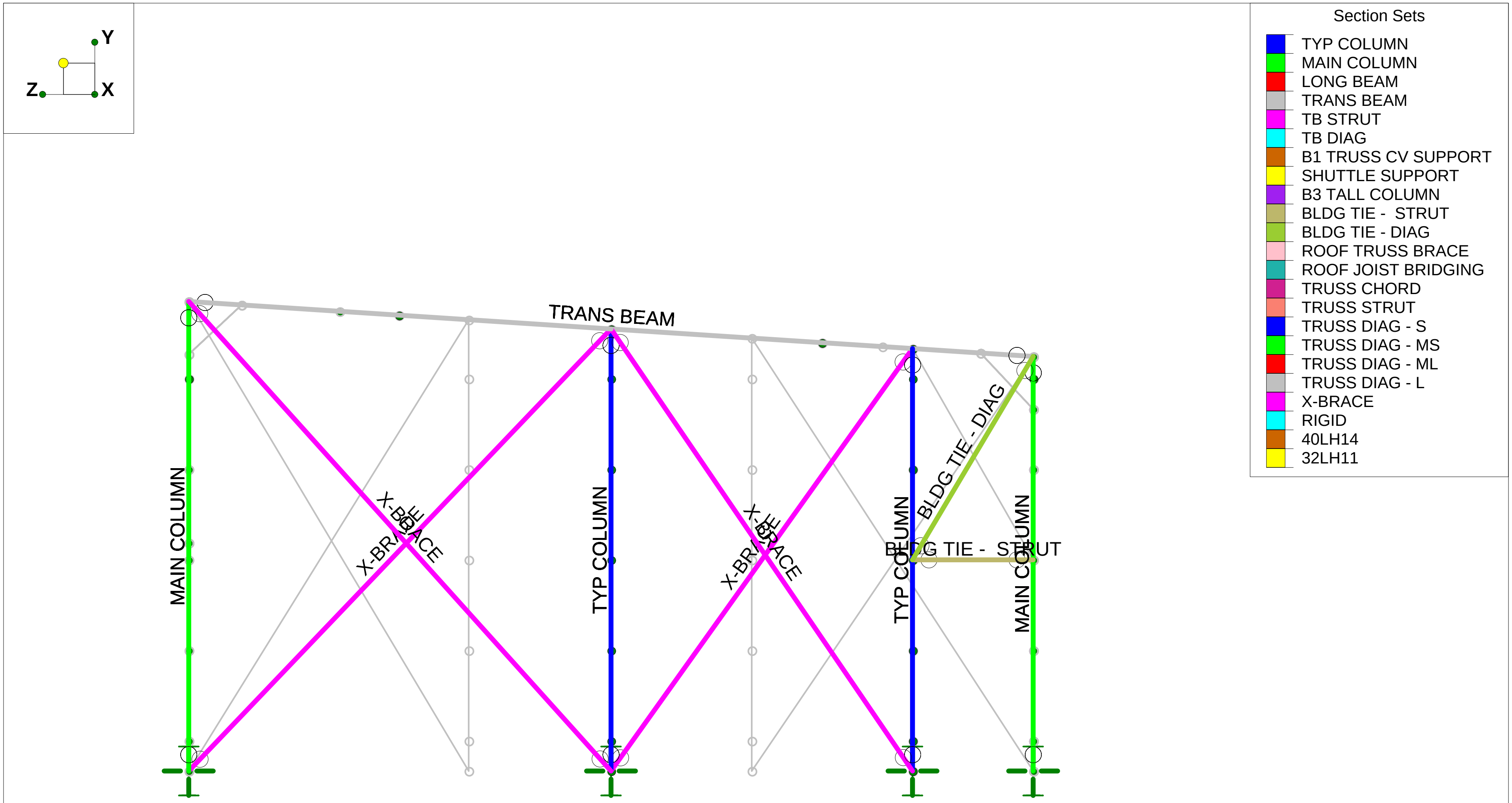
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:34 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B39



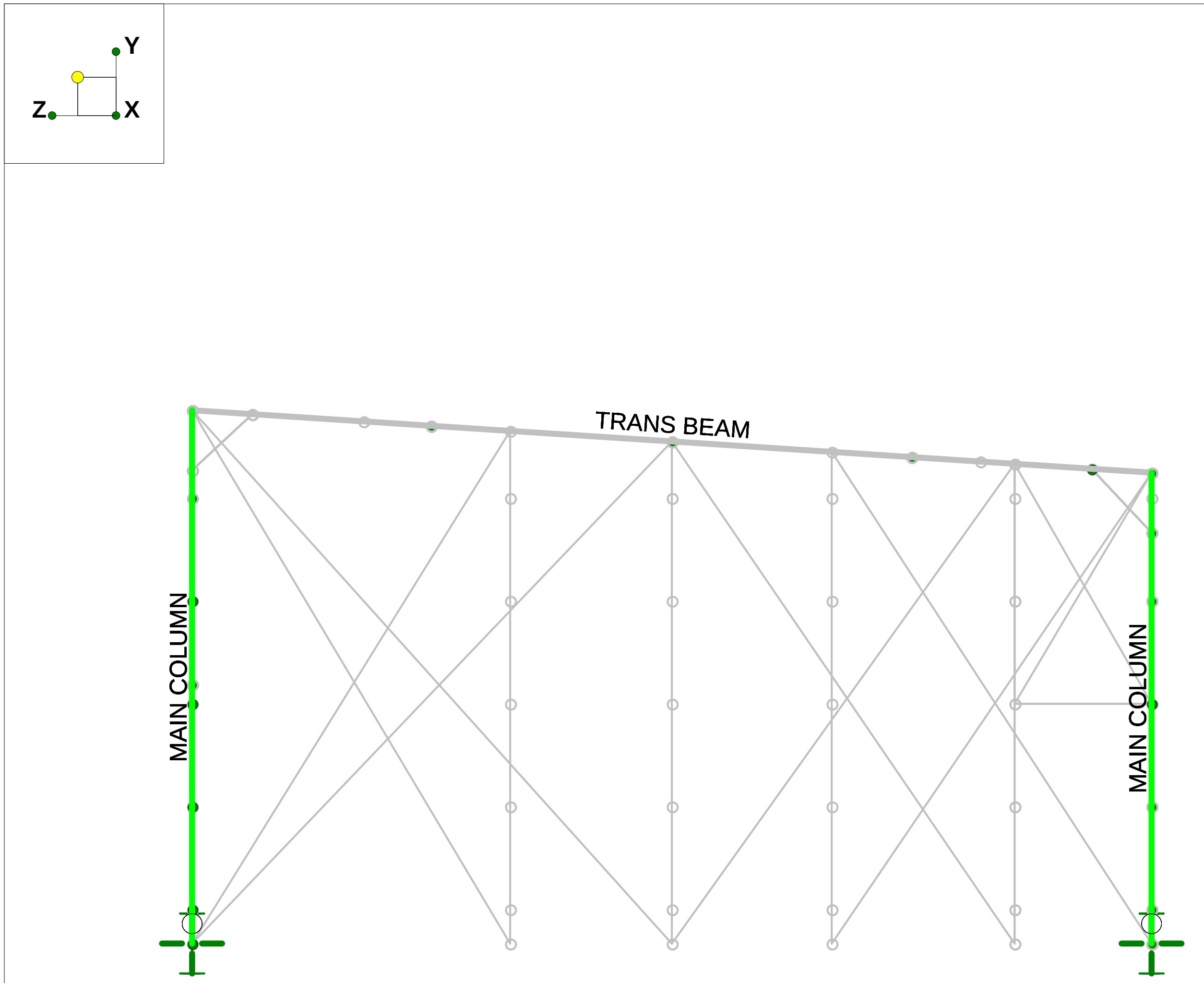
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:34 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B40



SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:36 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B41



Section Sets

- TYP COLUMN
- MAIN COLUMN
- LONG BEAM
- TRANS BEAM
- TB STRUT
- TB DIAG
- B1 TRUSS CV SUPPORT
- SHUTTLE SUPPORT
- B3 TALL COLUMN
- BLDG TIE - STRUT
- BLDG TIE - DIAG
- ROOF TRUSS BRACE
- ROOF JOIST BRIDGING
- TRUSS CHORD
- TRUSS STRUT
- TRUSS DIAG - S
- TRUSS DIAG - MS
- TRUSS DIAG - ML
- TRUSS DIAG - L
- X-BRACE
- RIGID
- 40LH14
- 32LH11

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:35 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B42

Load Combinations

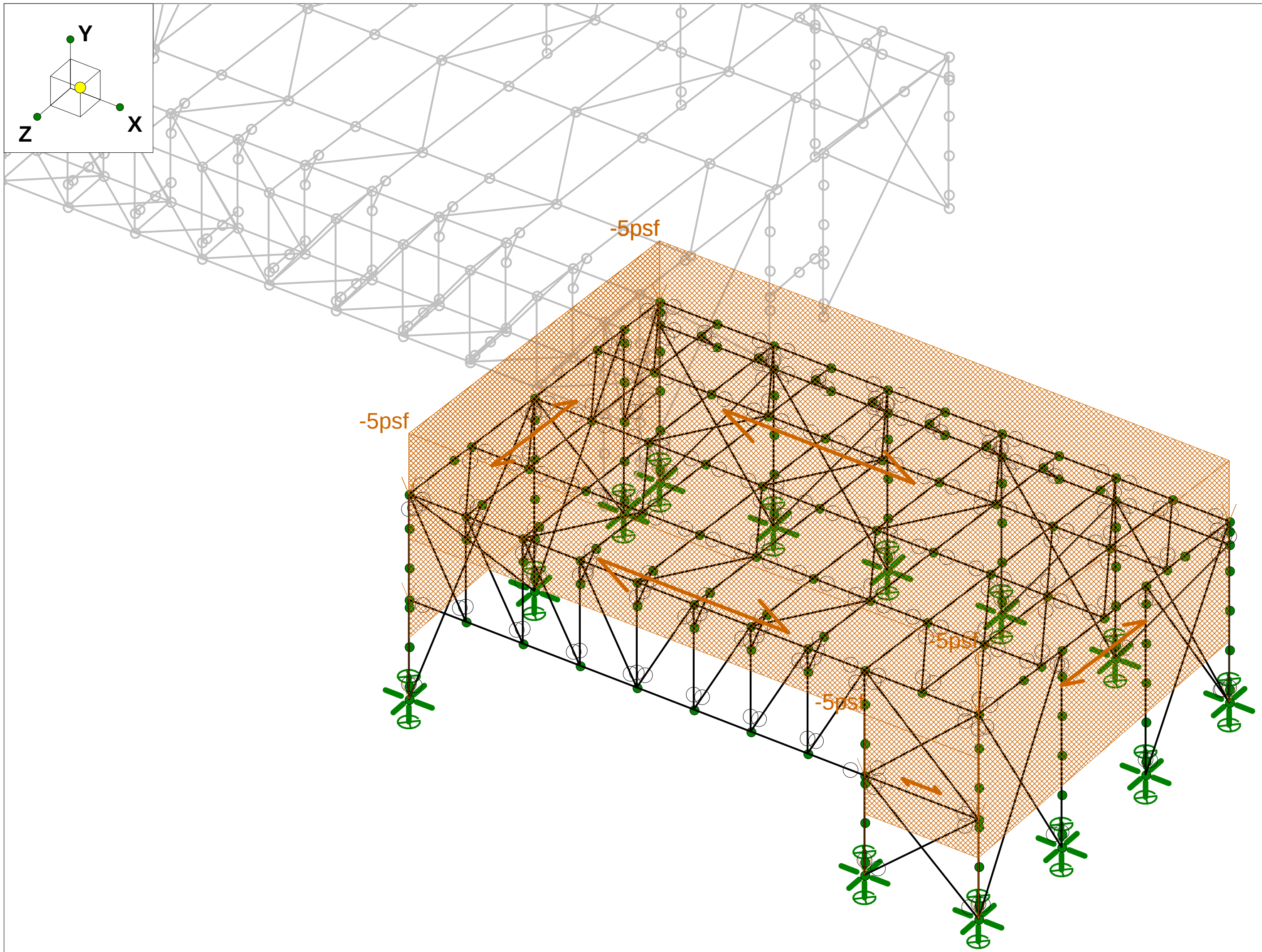
	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	DEAD + MATL		Y		DL	1																		
2	LIVE		Y		LL	1																		
3	SNOW LOAD		Y		3	1																		
4	WIND TRANS1 (WINDWARD + GCpi)		Y		5	1																		
5	WIND TRANS1 (WINDWARD - GCpi)		Y		6	1																		
6	WIND TRANS2 (LEEWARD + GCpi)		Y		7	1																		
7	WIND TRANS2 (LEEWARD - GCpi)		Y		8	1																		
8	WIND LONG + GCpi)		Y		9	1																		
9	WIND LONG - GCpi)		Y		10	1																		
10	SEISMIC - LONG		Y		16	1																		
11	SEISMIC - TRANS		Y		17	1																		
12																								
13	ASD LOAD COMBINATIONS																							
14	D + L		Y		DL	1	LL	1																
15	D + S		Y		DL	1	3	1																
16	D + 0.75L + 0.75S		Y		DL	1	LL	.75	3	.75														
17	D + 0.6W TRANS1 (WINDWARD)		Y		DL	1	5	.6																
18	-GCpi		Y		DL	1	6	.6																
19	D + 0.6W TRANS2 (LEEWARD)		Y		DL	1	7	.6																
20	-GCpi		Y		DL	1	8	.6																
21	D + 0.6W (LONG)		Y		DL	1	9	.6																
22	-GCpi		Y		DL	1	10	.6																
23	D + 0.6W (75% TRANS1 + 75% LONG)		Y		DL	1	5	.45	9	.45														
24	-GCpi		Y		DL	1	6	.45	10	.45														
25	D + 0.6W (75% TRANS2 + 75% LONG)		Y		DL	1	7	.45	9	.45														
26	-GCpi		Y		DL	1	8	.45	10	.45														
27	D + 0.75L + 0.75(0.6W[T1+]) + 0.75S		Y		DL	1	LL	.75	3	.75	5	.45												
28	D + 0.75L + 0.75(0.6W[T1-]) + 0.75S		Y		DL	1	LL	.75	3	.75	6	.45												
29	D + 0.75L + 0.75(0.6W[T2+]) + 0.75S		Y		DL	1	LL	.75	3	.75	7	.45												
30	D + 0.75L + 0.75(0.6W[T2-]) + 0.75S		Y		DL	1	LL	.75	3	.75	8	.45												
31	D + 0.75L + 0.75(0.6W[L+]) + 0.75S		Y		DL	1	LL	.75	3	.75	9	.45												
32	D + 0.75L + 0.75(0.6W[L-]) + 0.75S		Y		DL	1	LL	.75	3	.75	10	.45												
33	D + 0.75L + 0.75(0.6W[0.75T1+L+]) + ...		Y		DL	1	LL	.75	3	.75	5	.338	9	.338										
34	D + 0.75L + 0.75(0.6W[0.75T1-L-]) + 0...		Y		DL	1	LL	.75	3	.75	6	.338	10	.338										
35	D + 0.75L + 0.75(0.6W[0.75T2+L+]) + ...		Y		DL	1	LL	.75	3	.75	7	.338	9	.338										
36	D + 0.75L + 0.75(0.6W[0.75T2-L-]) + 0...		Y		DL	1	LL	.75	3	.75	8	.338	10	.338										
37	D + 0.7Ex (LONG) + Ey		Y		DL	1	16	.7	18	-.7														
38	D + 0.7Ex (LONG) - Ey		Y		DL	1	16	.7	18	.7														
39	D - 0.7Ex (LONG) + Ey		Y		DL	1	16	-.7	18	-.7														
40	D - 0.7Ex (LONG) - Ey		Y		DL	1	16	-.7	18	.7														
41	D + 0.7Ez (TRANS) + Ey		Y		DL	1	17	.7	18	-.7														
42	D + 0.7Ez (TRANS) - Ey		Y		DL	1	17	.7	18	.7														
43	D - 0.7Ez (TRANS) + Ey		Y		DL	1	17	-.7	18	-.7														
44	D - 0.7Ez (TRANS) - Ey		Y		DL	1	17	-.7	18	.7														
45	D + 0.75L + 0.75(0.7Ex+ Ey) + 0.75S		Y		DL	1	LL	.75	16	.525	3	.75	18	-.5...										
46	D + 0.75L + 0.75(0.7Ex- Ey) + 0.75S		Y		DL	1	LL	.75	16	.525	3	.75	18	.525										
47	D + 0.75L - 0.75(0.7Ex+ Ey) + 0.75S		Y		DL	1	LL	.75	16	-.5...	3	.75	18	-.5...										
48	D + 0.75L - 0.75(0.7Ex- Ey) + 0.75S		Y		DL	1	LL	.75	16	-.5...	3	.75	18	.525										
49	D + 0.75L + 0.75(0.7Ez + Ey) + 0.75S		Y		DL	1	LL	.75	17	.525	3	.75	18	-.5...										
50	D + 0.75L + 0.75(0.7Ez -Ey) + 0.75S		Y		DL	1	LL	.75	17	.525	3	.75	18	.525										
51	D + 0.75L - 0.75(0.7Ez + Ey) + 0.75S		Y		DL	1	LL	.75	17	-.5...	3	.75	18	-.5...										
52	D + 0.75L - 0.75(0.7Ez -Ey) + 0.75S		Y		DL	1	LL	.75	17	-.5...	3	.75	18	.525										
53																								
54	LRFD LOAD COMBINATIONS																							
55	1.4D	Y...			DL	1.4																		
56	1.2D + 1.6L + 0.5S	Y...			DL	1.2	LL	1.6	3	.5														

Load Combinations (Continued)

	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
57	1.2D + 1.6S + 0.5W (DOES NOT CON...	Y...																						
58	*1.2D + L+ 0.5S	Y...			DL	1.2	LL	1	3	.5														
59	1.2D + 1.0W (TRANS1) + L + 0.5S	Y...			L...	1	5	1																
60	-GCpi	Y...			L...	1	6	1																
61	TRANS2	Y...			L...	1	7	1																
62	-GCpi	Y...			L...	1	8	1																
63	LONG	Y...			L...	1	9	1																
64	-GCpi	Y...			L...	1	10	1																
65	COMB1	Y...			L...	1	5	.75	9	.75														
66	-GCpi	Y...			L...	1	6	.75	10	.75														
67	COMB2	Y...			L...	1	7	.75	9	.75														
68	-GCpi	Y...			L...	1	8	.75	10	.75														
69	(1.2+Ev)D + 1.0Ex + L + 0.2S	Y...			DL	1.38	E...	1	2	1	3	.2												
70	(1.2+Ev)D - 1.0Ex + L + 0.2S	Y...			DL	1.38	E...	-1	2	1	3	.2												
71	(1.2+Ev)D +1.0Ez + L + 0.2S	Y...			DL	1.38	E...	1	2	1	3	.2												
72	(1.2+Ev)D - 1.0Ez + L + 0.2S	Y...			DL	1.38	E...	-1	2	1	3	.2												
73	(0.9-Ev)D + 1.0Ex	Y...			DL	.72	E...	1																
74	(0.9-Ev)D - 1.0Ex	Y...			DL	.72	E...	-1																
75	(0.9-Ev)D + 1.0Ey	Y...			DL	.72	E...	1																
76	(0.9-Ev)D - 1.0Ey	Y...			DL	.72	E...	-1																

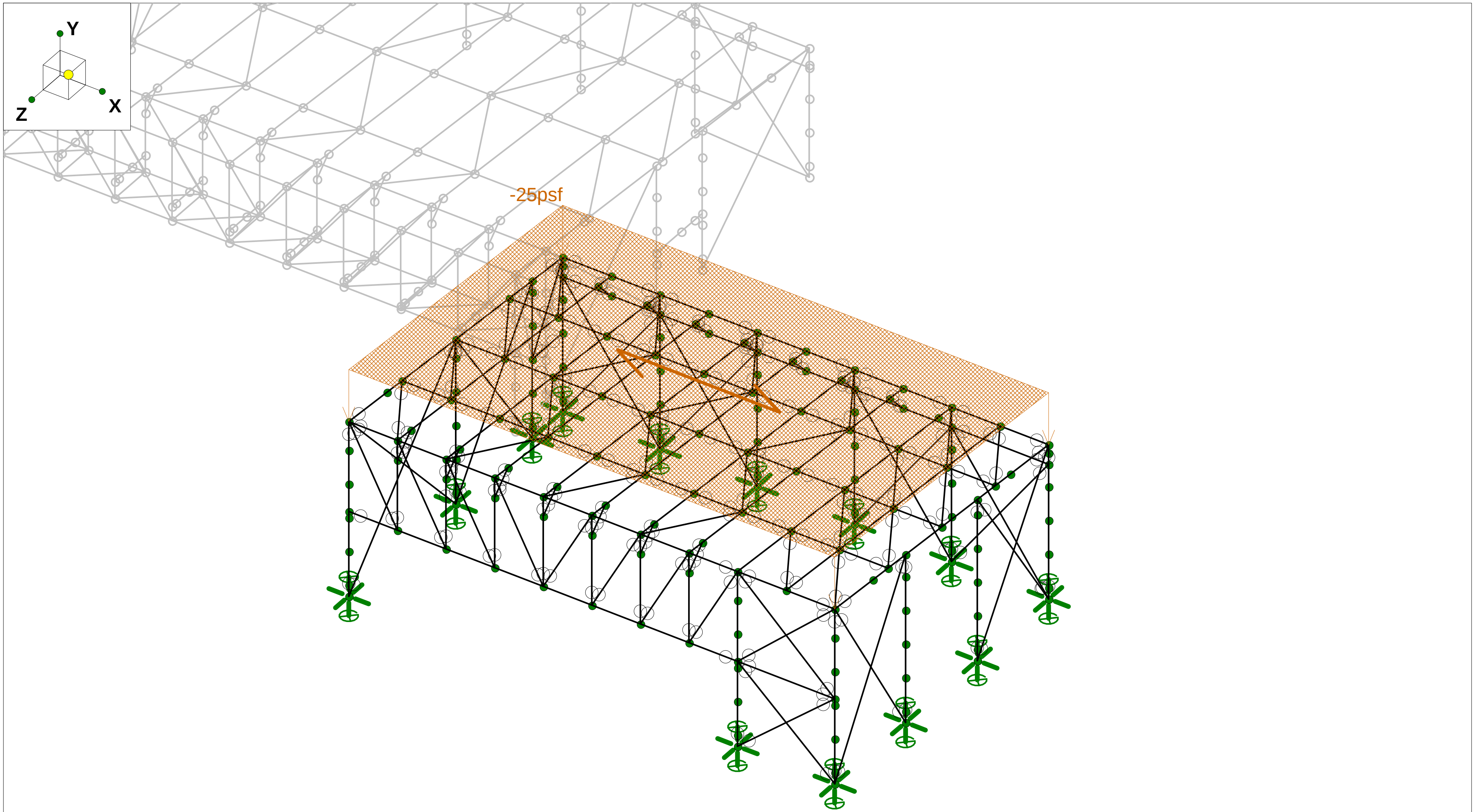
Basic Load Cases

	BLC Description	Category	X Gr...	Y Gr...	Z Gra...	Joint	Point	Distri...	Area(...	Surfa...
1	DEAD LOAD	DL		-1.05					23	
2	LIVE LOAD	LL								
3	ROOF SNOW LOAD	None							4	
5	WIND TRANS 1 +GCpi	None							21	
6	-GCpi	None							21	
7	WIND TRANS 2 +GCpi	None							21	
8	-GCpi	None							21	
9	WIND LONG +GCpi	None							18	
10	-GCpi	None							18	
11		None						707		
16	SEISMIC LONG	ELX	-.27			171			23	
17	SEISMIC TRANSVERSE	ELZ			-.27	171			23	
18	SEISMIC VERTICAL	ELY		-.18		171			23	
19	CV DL	DL				90				
20	CV MATL + IMPACT	DL				62				
21	CV LL	LL				27				
22	CV CHUTE DL	DL				8				



Loads: BLC 1, DEAD LOAD

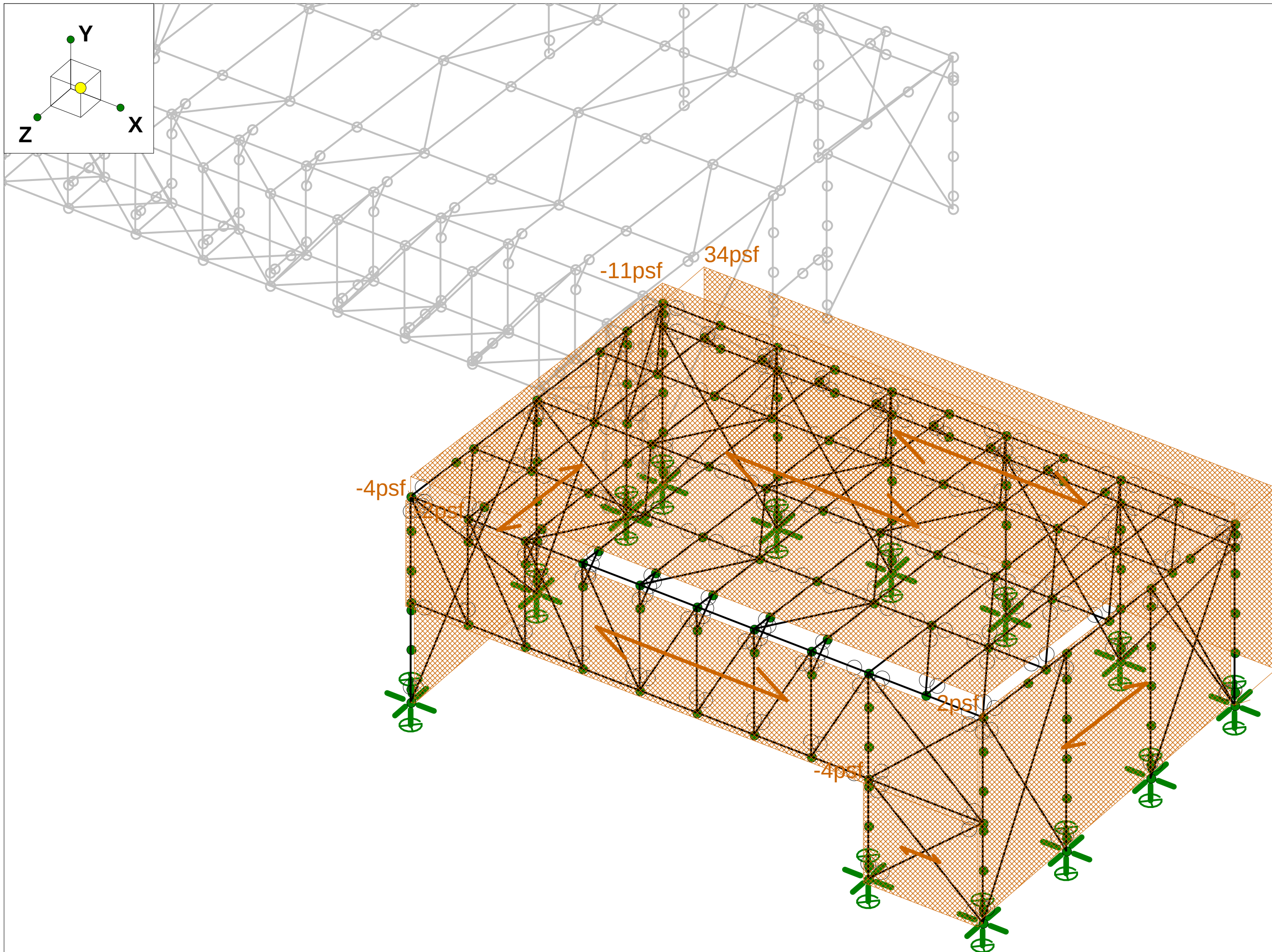
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:52 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Loads: BLC 3, ROOF SNOW LOAD

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:53 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

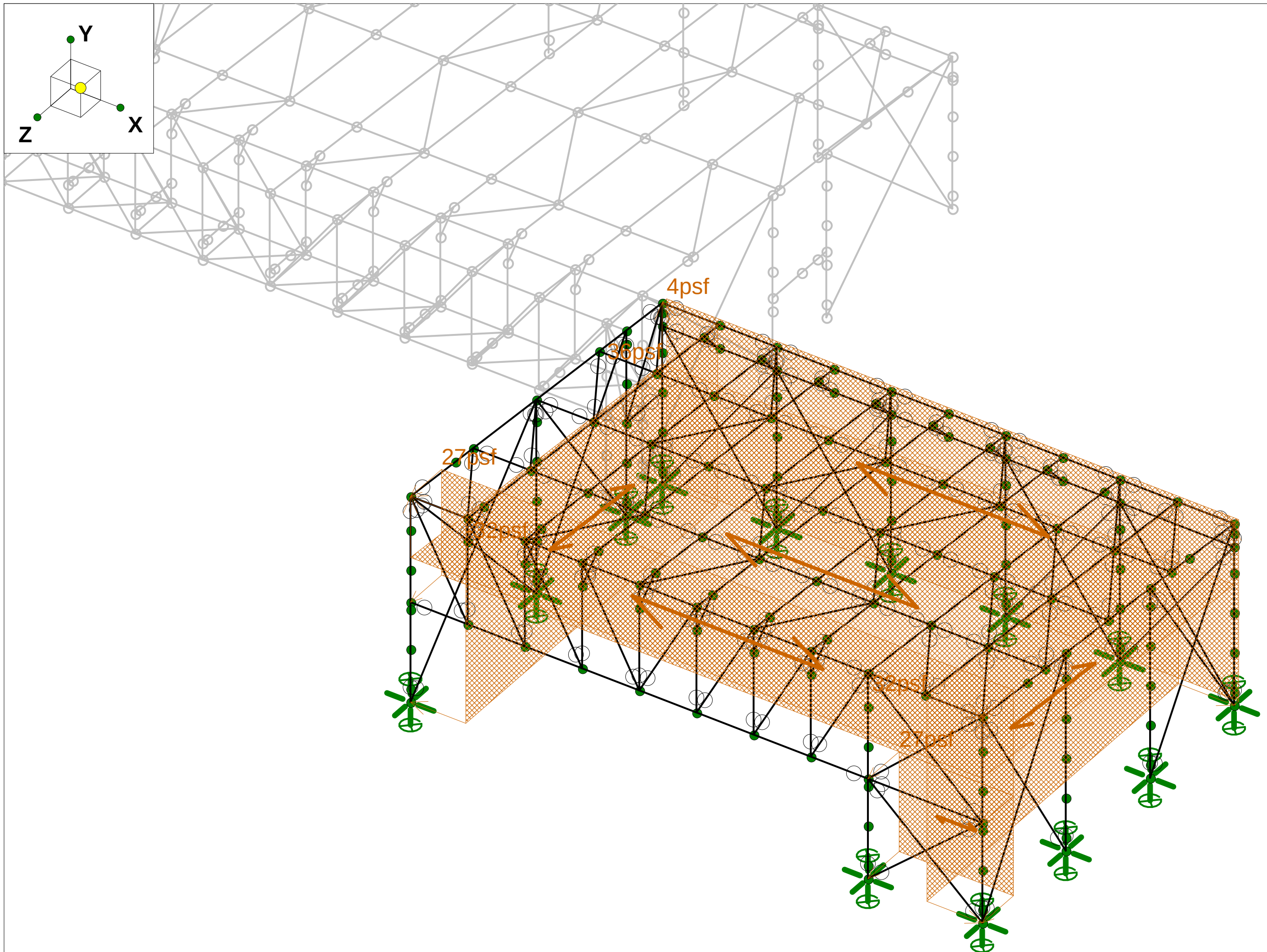
SHEET B47



Loads: BLC 5, WIND TRANS 1 +GCpi

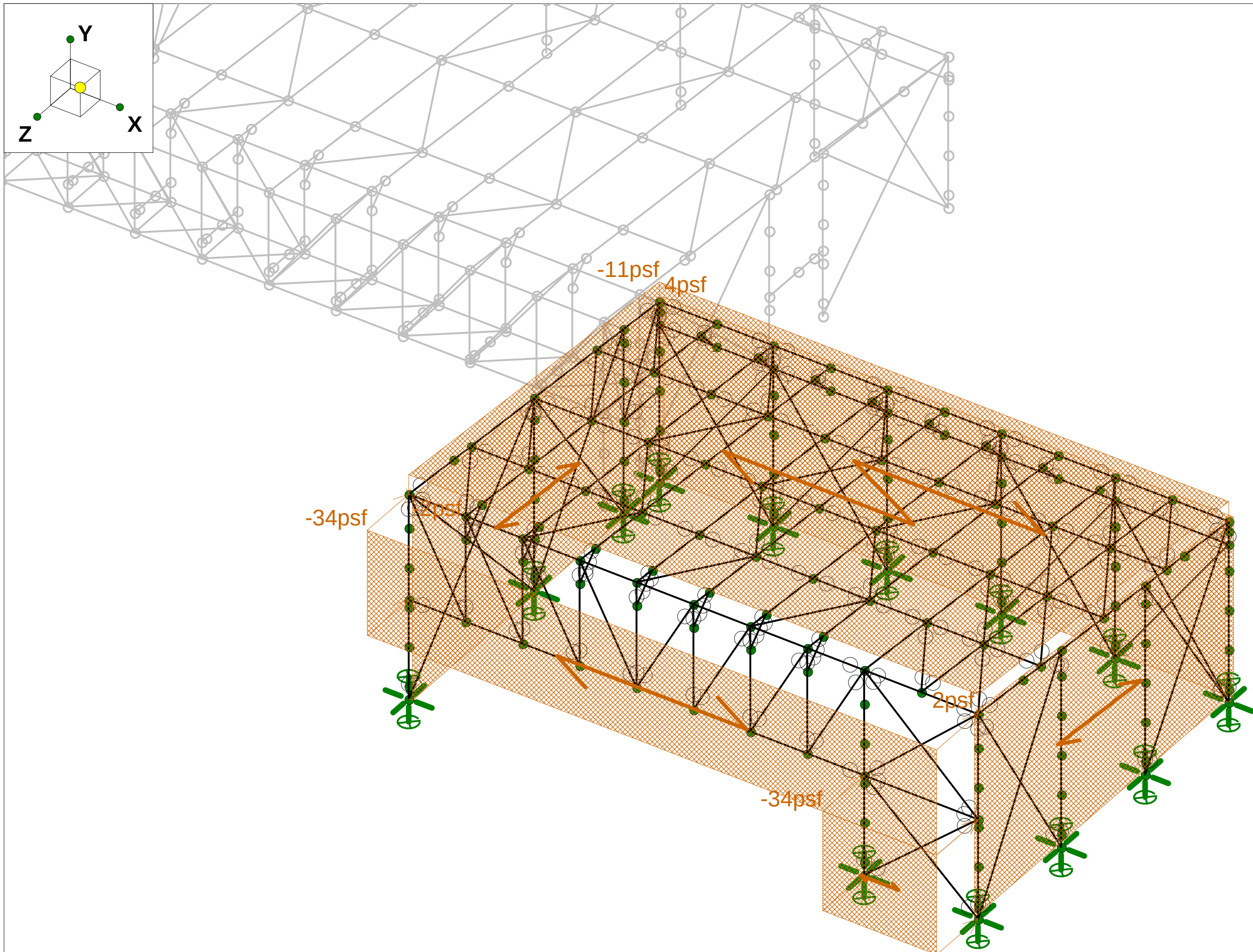
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:53 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B48



Loads: BLC 6, -GCpi

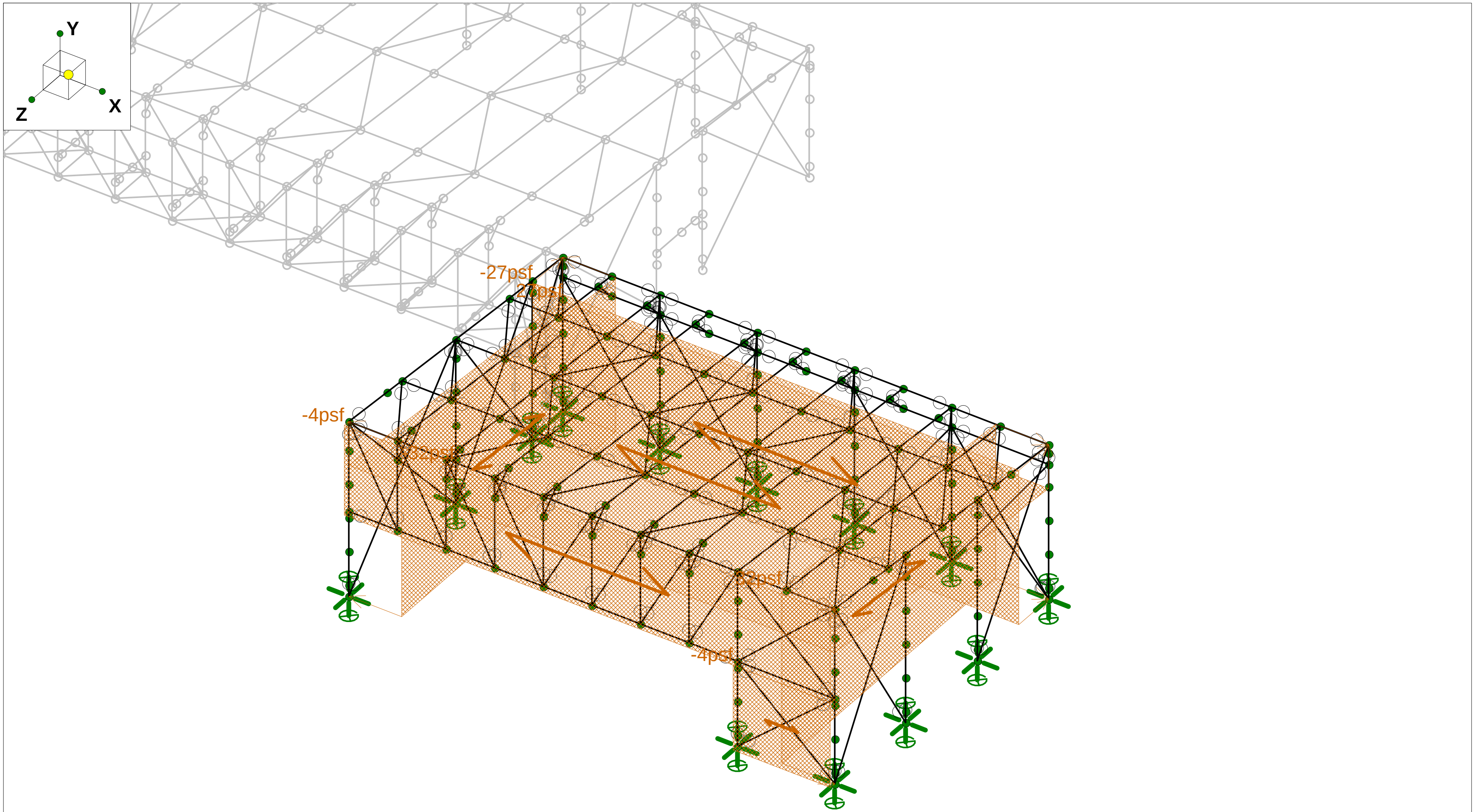
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:53 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Loads: BLC 7, WIND TRANS 2 +GCpi

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:53 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

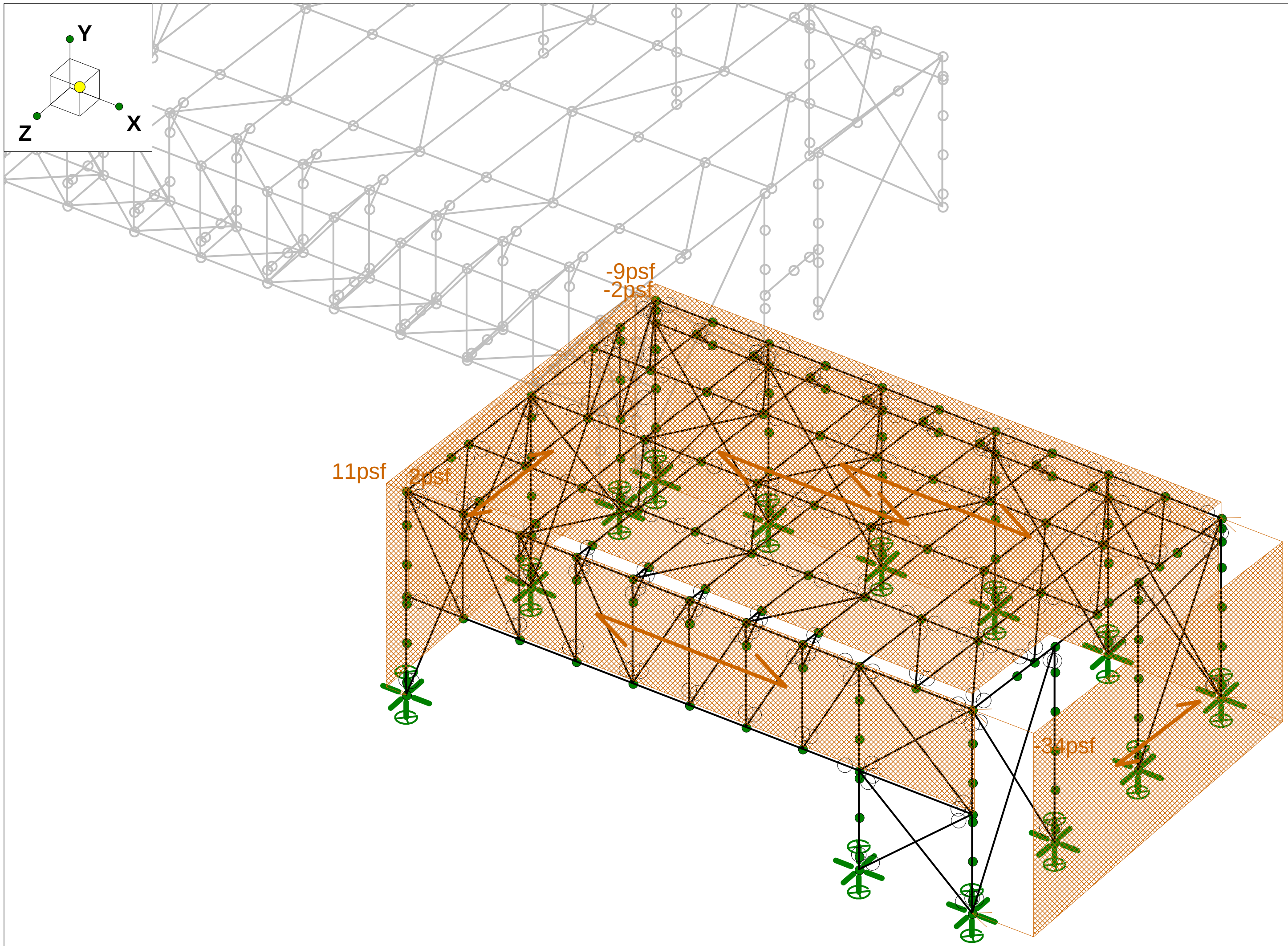
SHEET B50



Loads: BLC 8, -GCpi

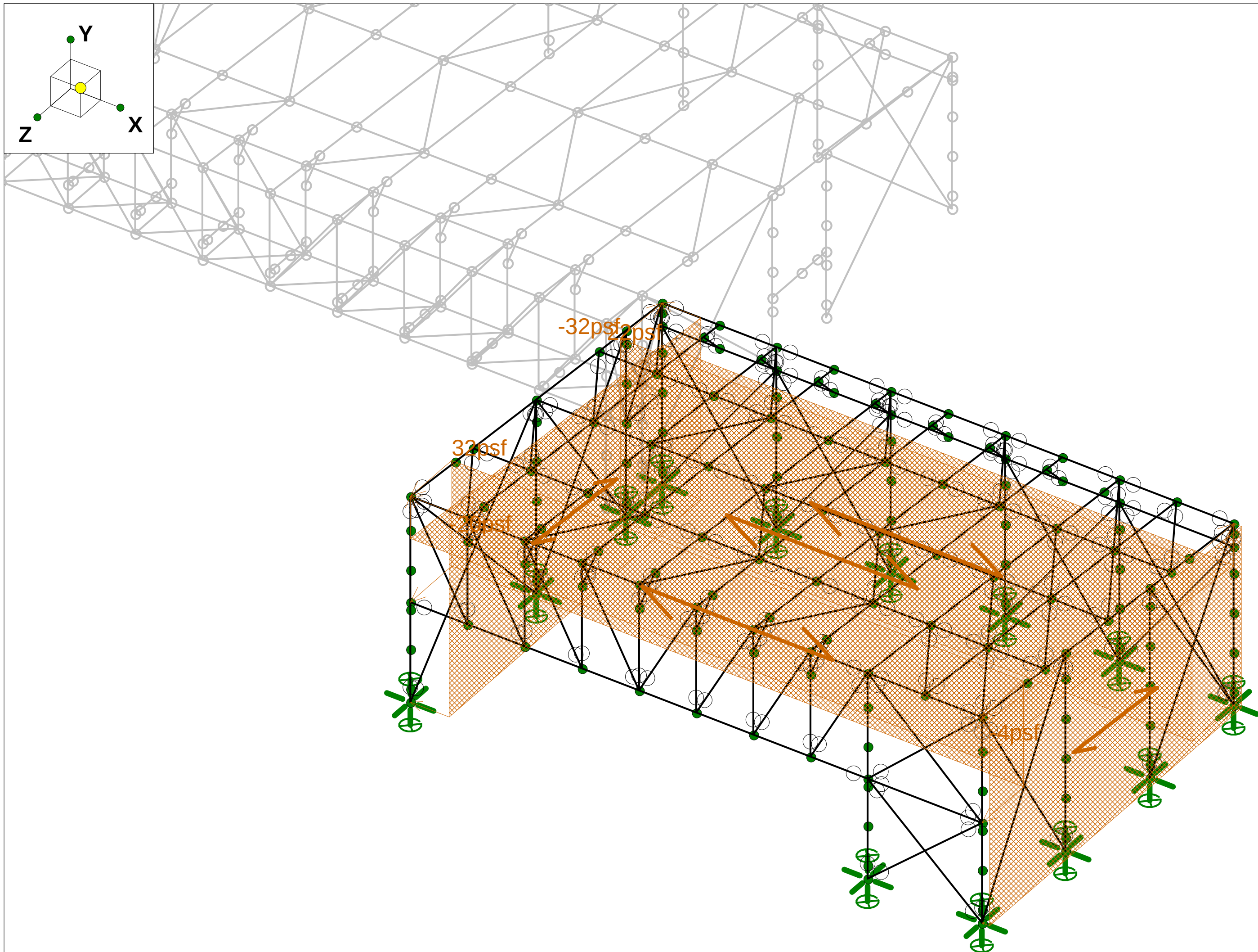
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:54 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B51



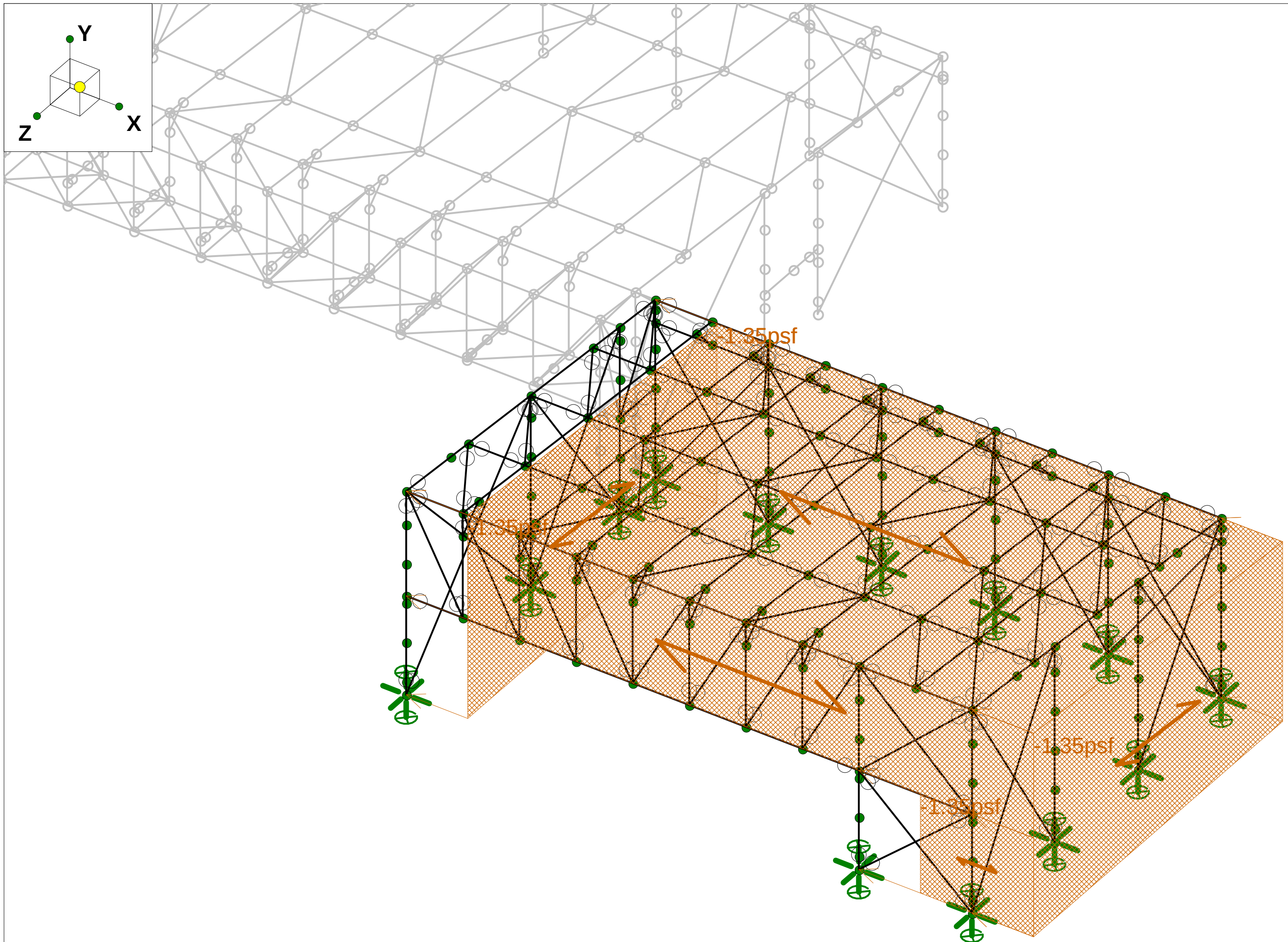
Loads: BLC 9, WIND LONG +GCpi

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:54 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Loads: BLC 10, -GCpi

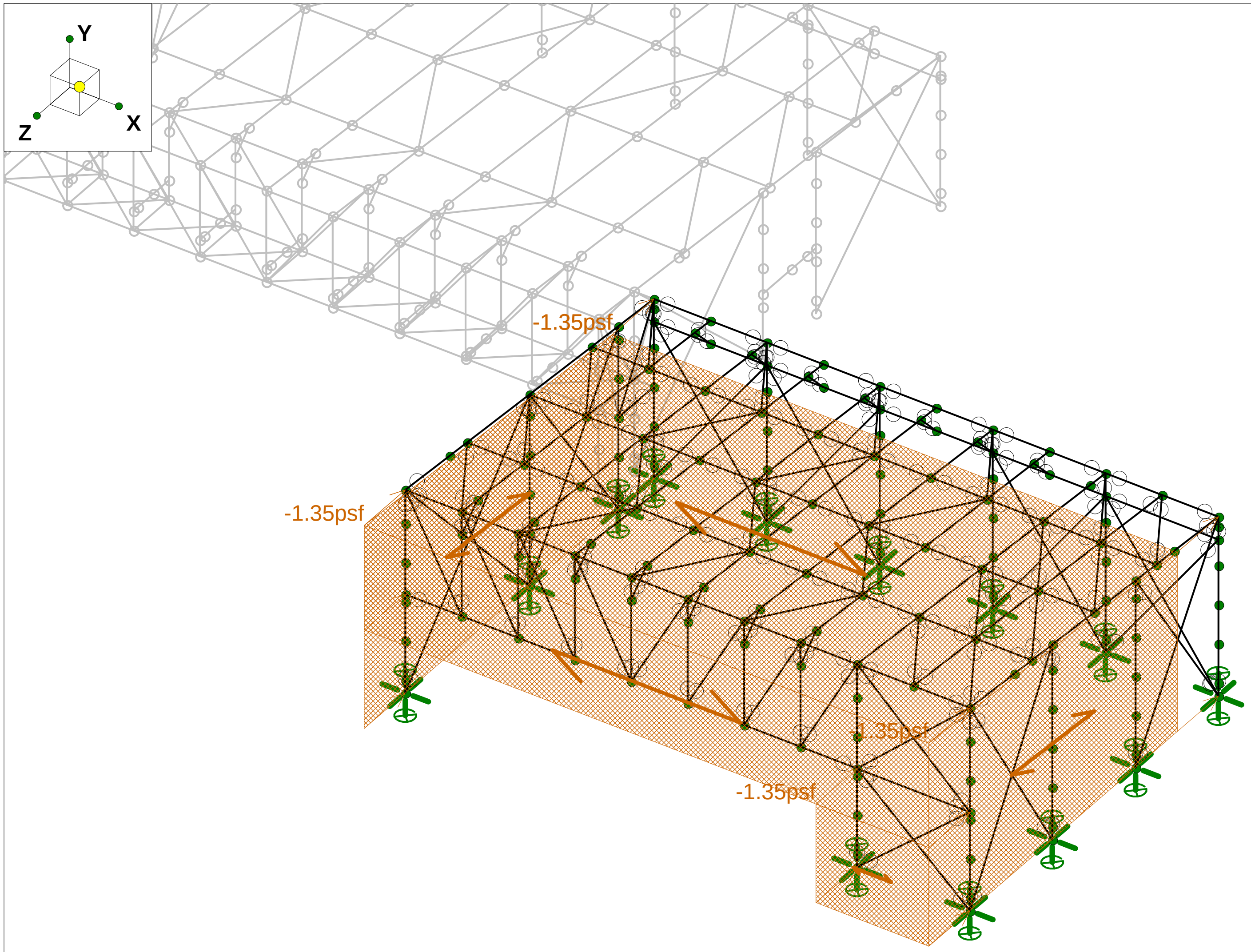
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:54 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Loads: BLC 16, SEISMIC LONG

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:54 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

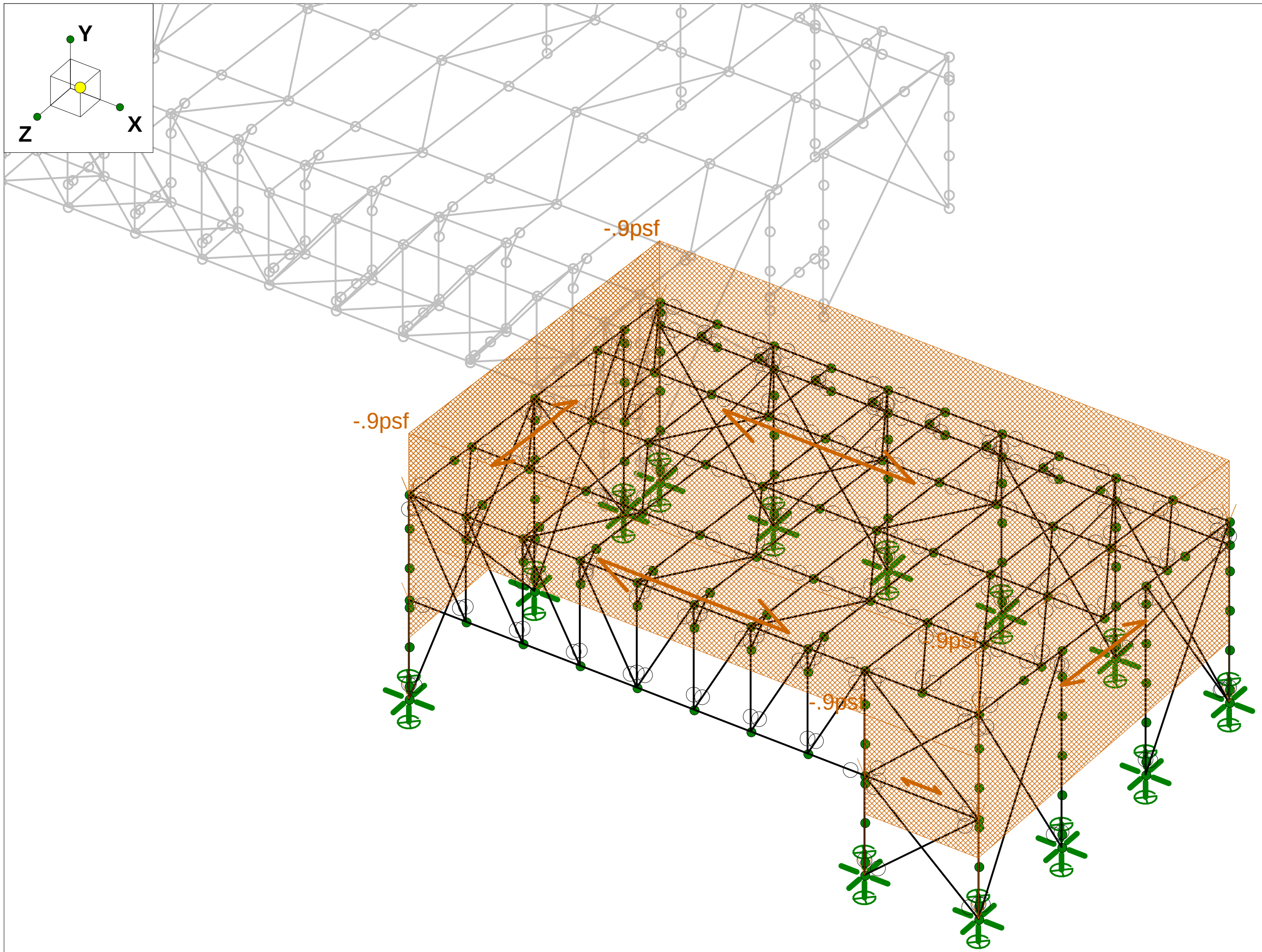
SHEET B54



Loads: BLC 17, SEISMIC TRANSVERSE

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:55 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

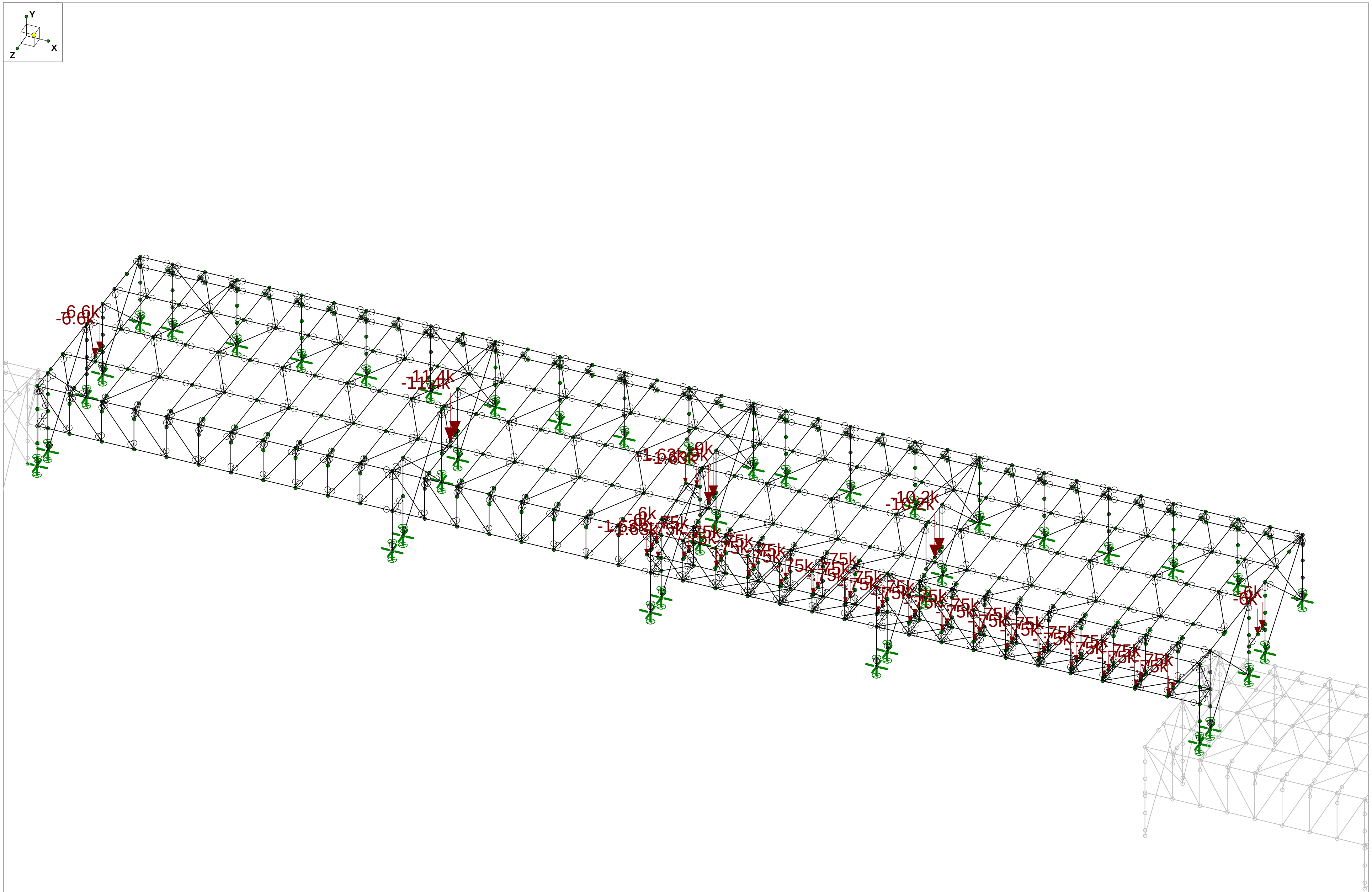
SHEET B55



Loads: BLC 18, SEISMIC VERTICAL

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:55 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

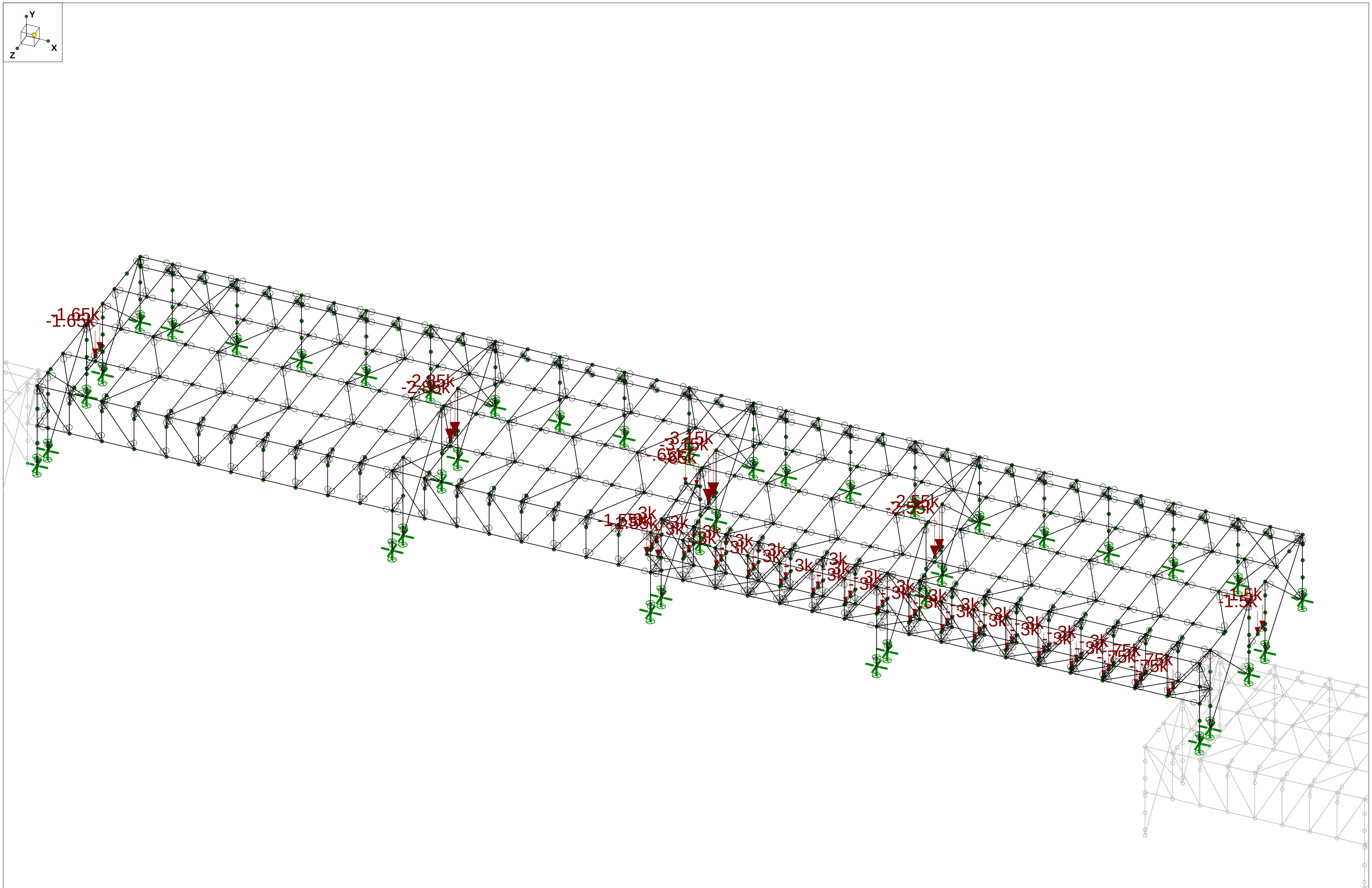
SHEET B56



Loads: BLC 19, CV DL

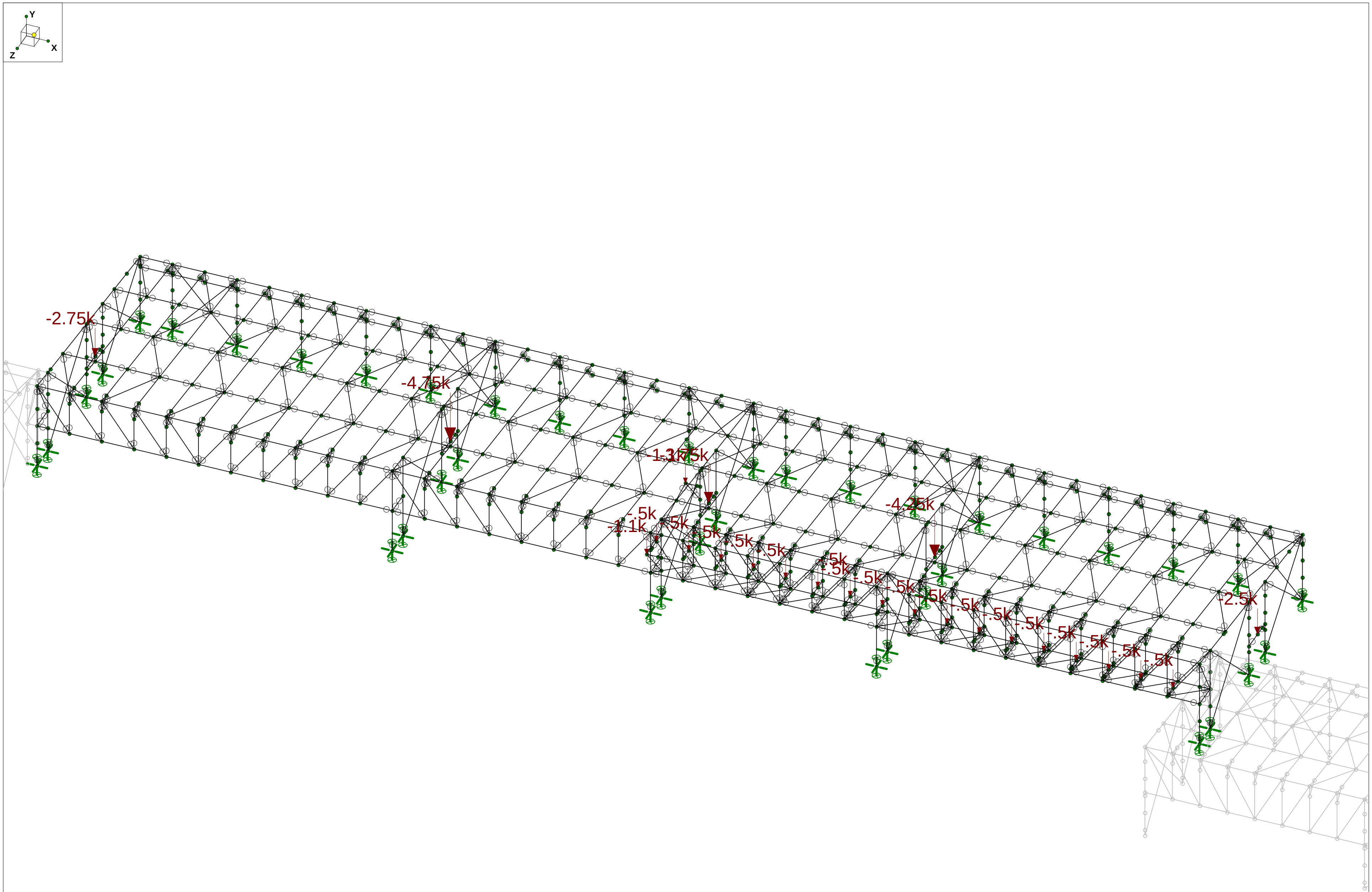
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:58 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B57



Loads: BLC 20, CV MATL + IMPACT

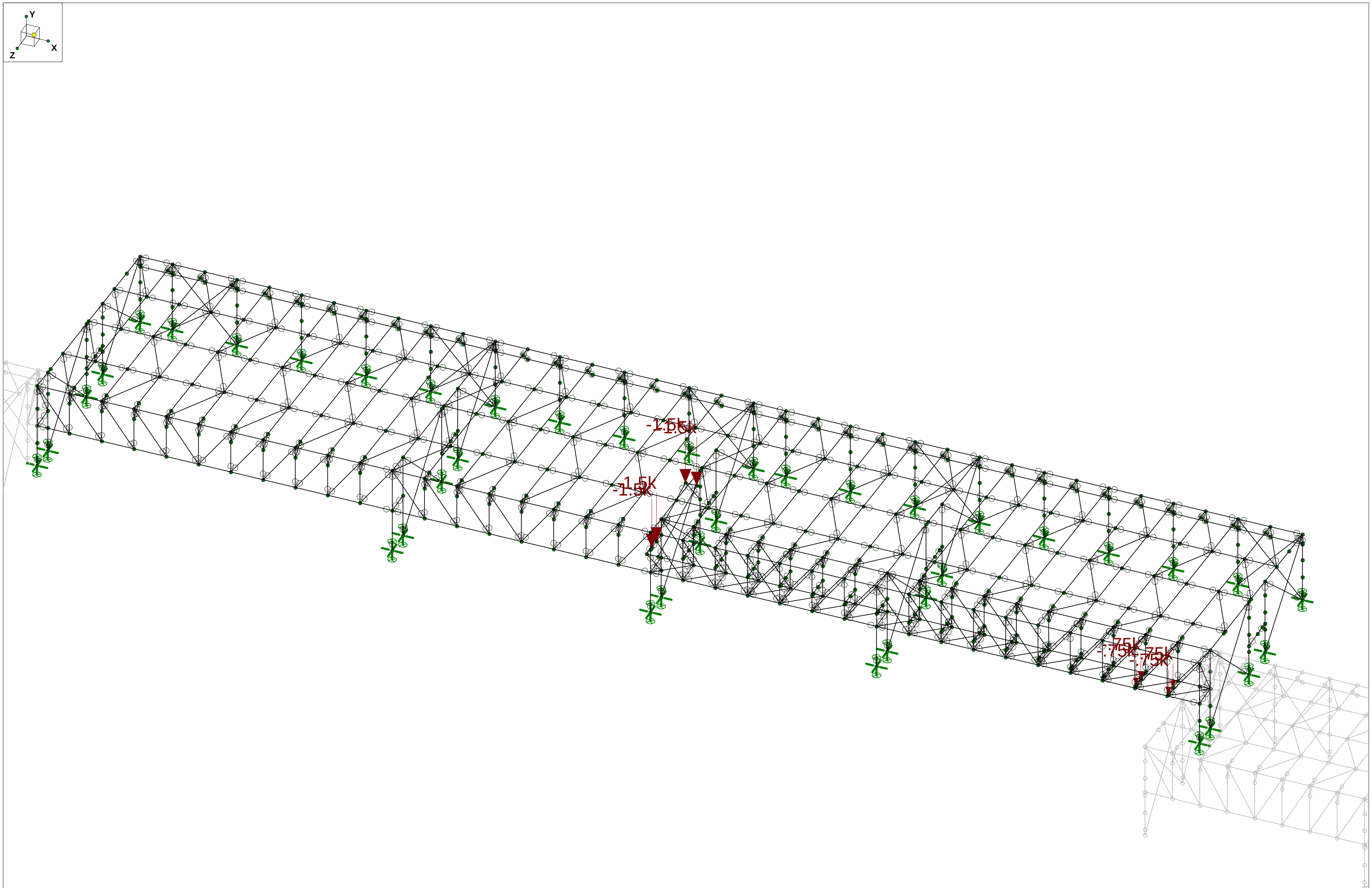
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:58 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Loads: BLC 21, CV LL

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:58 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B59



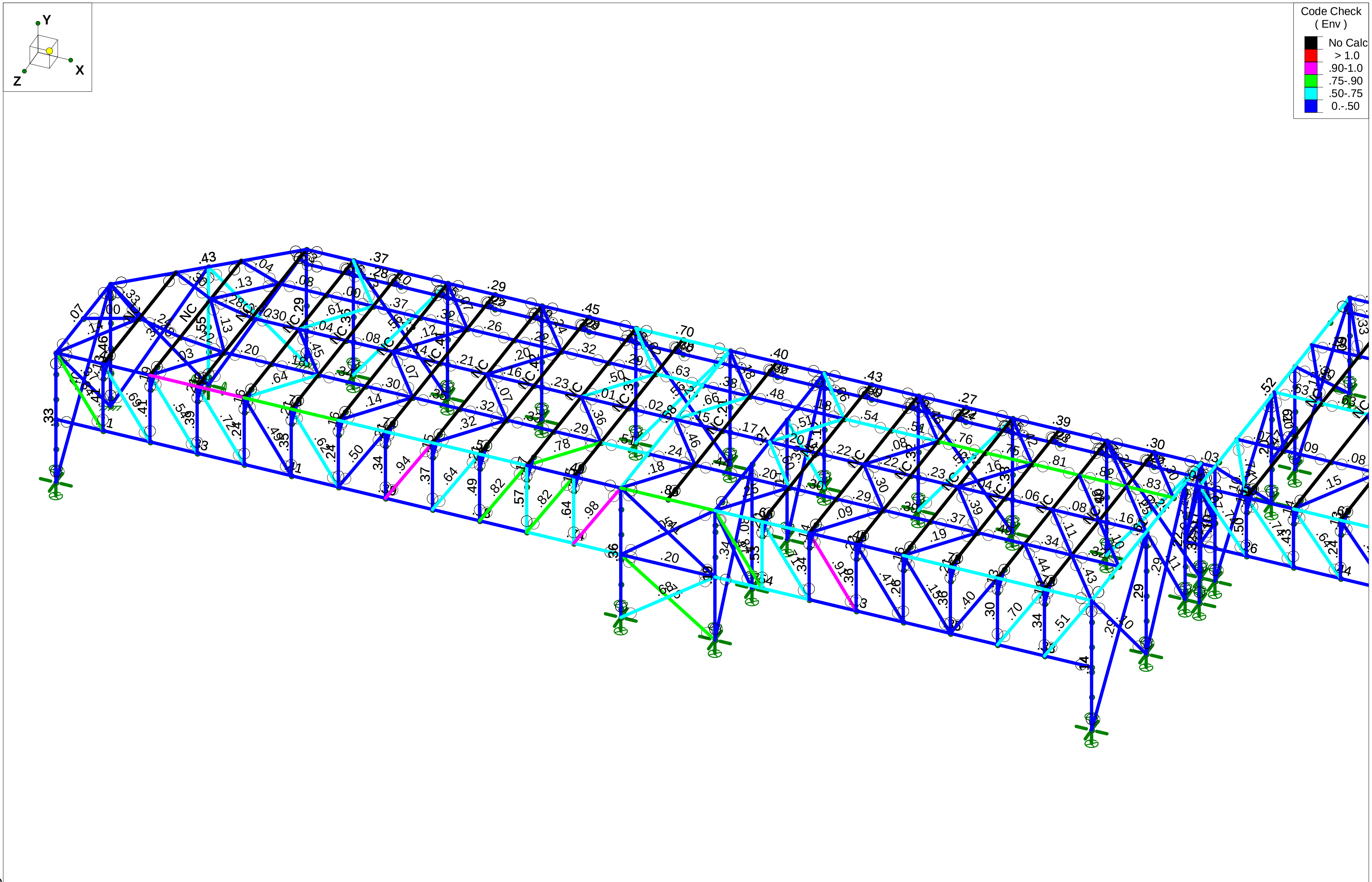
Loads: BLC 22, CV CHUTE DL

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Apr 6, 2019 at 4:59 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B60

Envelope AISC 14th(360-10): ASD Steel Code Checks

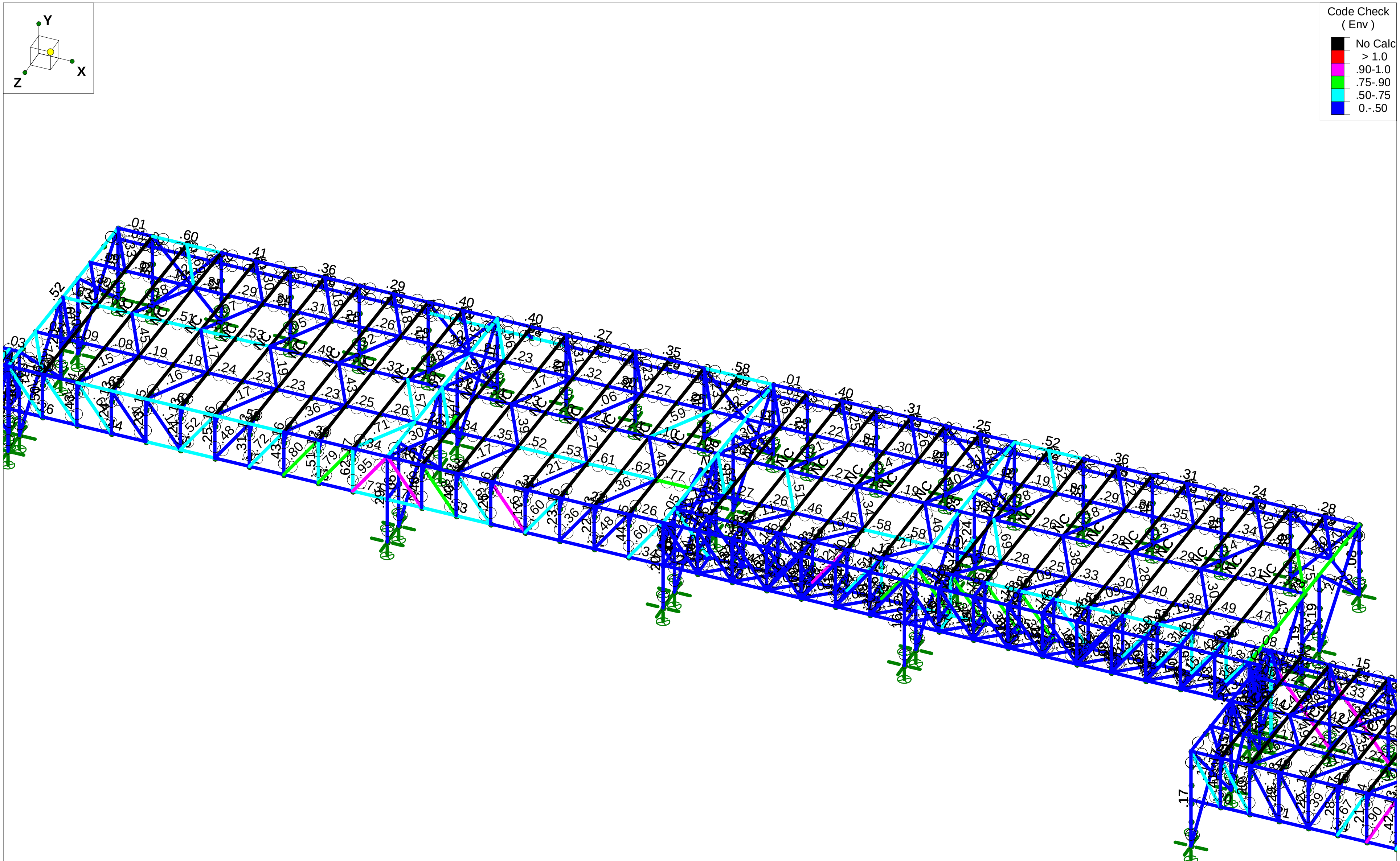
	Memb...	Shape	Code Check	Loc[ft]	LC	Sh...	Loc[ft]	Dir	LC	Pnc/...	Pnt/o...	Mnyy/...	Mnzz/om...	Cb	Eqn
1	M107...	LL4x3x5x6	.977	9.237	33	.019	0	y	27	24.2...	90.108	7.281	7.024	1	H1...
2	M109...	L4x3x4	.962	0	38	.001	0	y	20	3.675	36.431	1.227	1.979	1	H2...
3	M908A	LL4x3.5x...	.952	8.782	33	.017	0	y	27	33.8...	115.545	11.349	8.52	1	H1...
4	M249_1	LL3x2x4x6	.940	10.22	24	.004	0	y	48	7.874	51.737	3.086	3.102	1	H1...
5	M1047	LL3x2x4x6	.922	8.917	24	.007	0	y	20	10.1...	51.737	3.086	3.102	1	H1...
6	M109...	L4x3x4	.917	0	38	.000	0	y	14	3.675	36.431	1.227	1.979	1	H2...
7	M1082	L4x3x4	.917	0	38	.000	0	y	14	3.675	36.431	1.227	1.979	1	H2...
8	M328...	W8x31	.913	10	33	.025	10	z	17	191...	273.353	35.124	75.767	1.58	H1...
9	M847B	LL3x2x4x6	.912	10.22	24	.005	0	y	20	7.874	51.737	3.086	3.102	1	H1...
10	M780A	LL4x3x5x6	.908	8.423	33	.015	0	y	33	27.4	90.108	7.281	7.024	1	H1...
11	M978	LL3x2x4x6	.907	8.065	24	.008	17.205	y	33	8.889	51.737	3.086	3.102	1	H1...
12	M143A	LL3x2x4x6	.905	9.814	24	.004	0	y	48	7.874	51.737	3.086	3.102	1	H1...
13	M1084	L4x3x4	.899	0	38	.000	0	y	36	3.675	36.431	1.227	1.979	1	H2...
14	M203_1	W8x31	.861	20	33	.141	20	y	35	191...	273.353	35.124	75.767	1.572	H1...
15	M1041	LL3x2x4x6	.860	8.917	24	.007	0	y	17	10.1...	51.737	3.086	3.102	1	H1...
16	M781A	LL4x3x4x6	.848	8.423	33	.007	0	y	33	22.4...	72.862	5.723	5.686	1	H1...
17	M256_1	LL4x3x4x6	.843	9.237	33	.017	0	y	27	19.8...	72.862	5.723	5.686	1	H1...
18	M242_1	LL4x3x4x6	.836	9.237	33	.008	0	y	33	19.8...	72.862	5.723	5.686	1	H1...
19	M1043	LL4x3x5x6	.832	8.217	33	.006	16.785	y	16	24.8...	90.108	7.281	7.024	1	H1...
20	M247_1	LL4x3x4x6	.824	9.237	33	.007	18.868	y	20	19.8...	72.862	5.723	5.686	1	H1...
21	M107...	LL4x3x5x6	.823	9.237	33	.004	0	y	34	24.2...	90.108	7.281	7.024	1	H1...
22	M1044	LL4x3.5x...	.819	8.217	33	.026	0	y	33	30.7...	115.545	11.349	8.52	1	H1...
23	M1045	LL4x3x5x6	.810	8.217	33	.011	0	y	20	24.8...	90.108	7.281	7.024	1	H1...
24	M1140	W12x40	.805	6.5	38	.344	12	y	46	234...	350.299	41.916	142.216	1.265	H1...
25	M776B	LL4x3x5x6	.797	8.423	33	.008	0	y	27	27.4	90.108	7.281	7.024	1	H1...
26	M775B	LL4x3.5x...	.789	8.423	33	.009	0	y	33	33.8...	115.545	11.349	8.52	1	H1...
27	M1042	LL4x3x4x6	.783	8.217	33	.005	16.785	y	16	20.3...	72.862	5.723	5.686	1	H1...
28	M396A	W24x104	.781	49.03	21	.044	0	z	21	45.4...	919.162	155.689	145.79	1.239	H1...
29	M329_1	W8x31	.759	0	27	.018	10	z	17	191...	273.353	35.124	71.309	1	H1...
30	M107...	L4x3x4	.758	0	37	.000	0	y	14	5.002	36.431	1.227	2.163	1	H2...



Member Code Checks Displayed (Enveloped)
Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 2:58 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

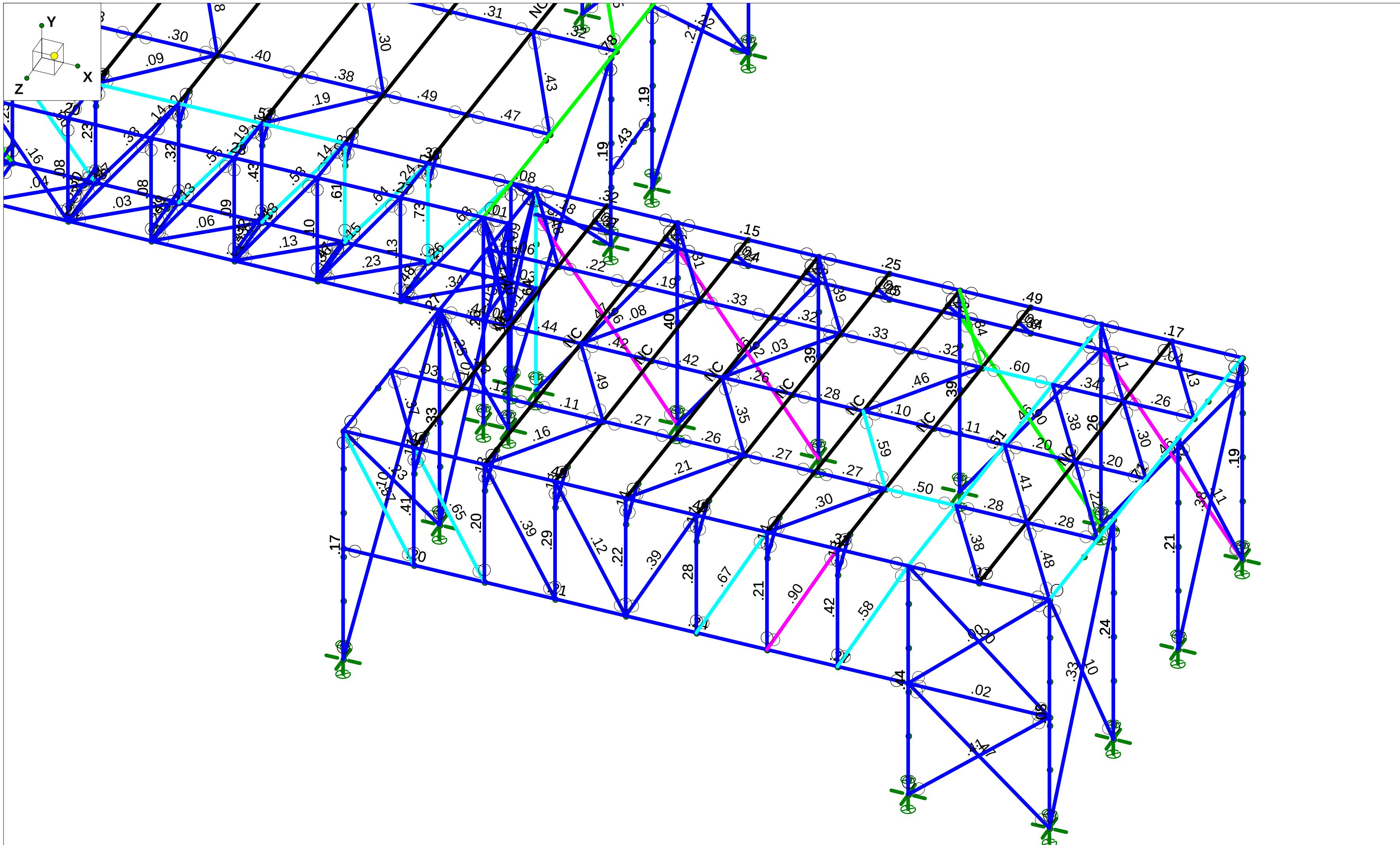
SHEET B62



Member Code Checks Displayed (Enveloped)
Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 2:58 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B63



Code Check
(Env)

|---|---|
Black	No Calc
Red	> 1.0
Magenta	.90-1.0
Green	.75-.90
Cyan	.50-.75
Blue	0.-.50

Member Code Checks Displayed (Enveloped)
Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 2:59 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

IBC DEFL LIMIT CHECK

EXTERIOR MAX DEFLECTION

* DEMAND DETERMINED NEGLECTING ROOF PURLINS & DECKING
MODEL w/ 1/8" STEEL PARTIAL DIAPHRAGM FOR ADD'L STIFFNESS

NEGLECTING ROOF COMPONENTS

BUILDING #1

$\Delta_{z,max} = 6.6''$
@ L = 113.5'

$\Delta = L/206 < L/120$ LIMIT PER IBC OK✓

$\Delta_{x,max} = 11.8''$
@ L = 80'

→ w/ 1/8" DIAPHRAGM @ ROOF $\Delta_{x,max} = 0.5''$

$\Delta = L/1920$ OK✓

BUILDING #2

$\Delta_{x,max} = 10.8''$ → w/ 1/8" DIAPHRAGM @ ROOF $\Delta_{x,max} = 0.5''$
@ L = 56'

Δ_z CONTROLLED BY BUILDING #3

$\Delta = L/1340$ OK✓

BUILDING #3

$\Delta_{z,max} = 7.2''$
@ L = 120'

$\Delta = L/200$ OK✓

IBC DEFL LIMITS

TABLE 1604.3
DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	L	S or W[f]	$D + L$[d, g]
Roof members:[e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240
Exterior walls:			
With plaster or stucco finishes	—	l/360	—
With other brittle finishes	—	l/240	—
With flexible finishes	—	l/120	—
Interior partitions:[b]			
With plaster or stucco finishes	l/360	—	—
With other brittle finishes	l/240	—	—
With flexible finishes	l/120	—	—
Farm buildings	—	—	l/180
Greenhouses	—	—	l/120

For SI: 1 foot = 304.8 mm.

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed l/60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed l/150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed l/90. For roofs, this exception only applies when the metal sheets have no roof covering.

b. Flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

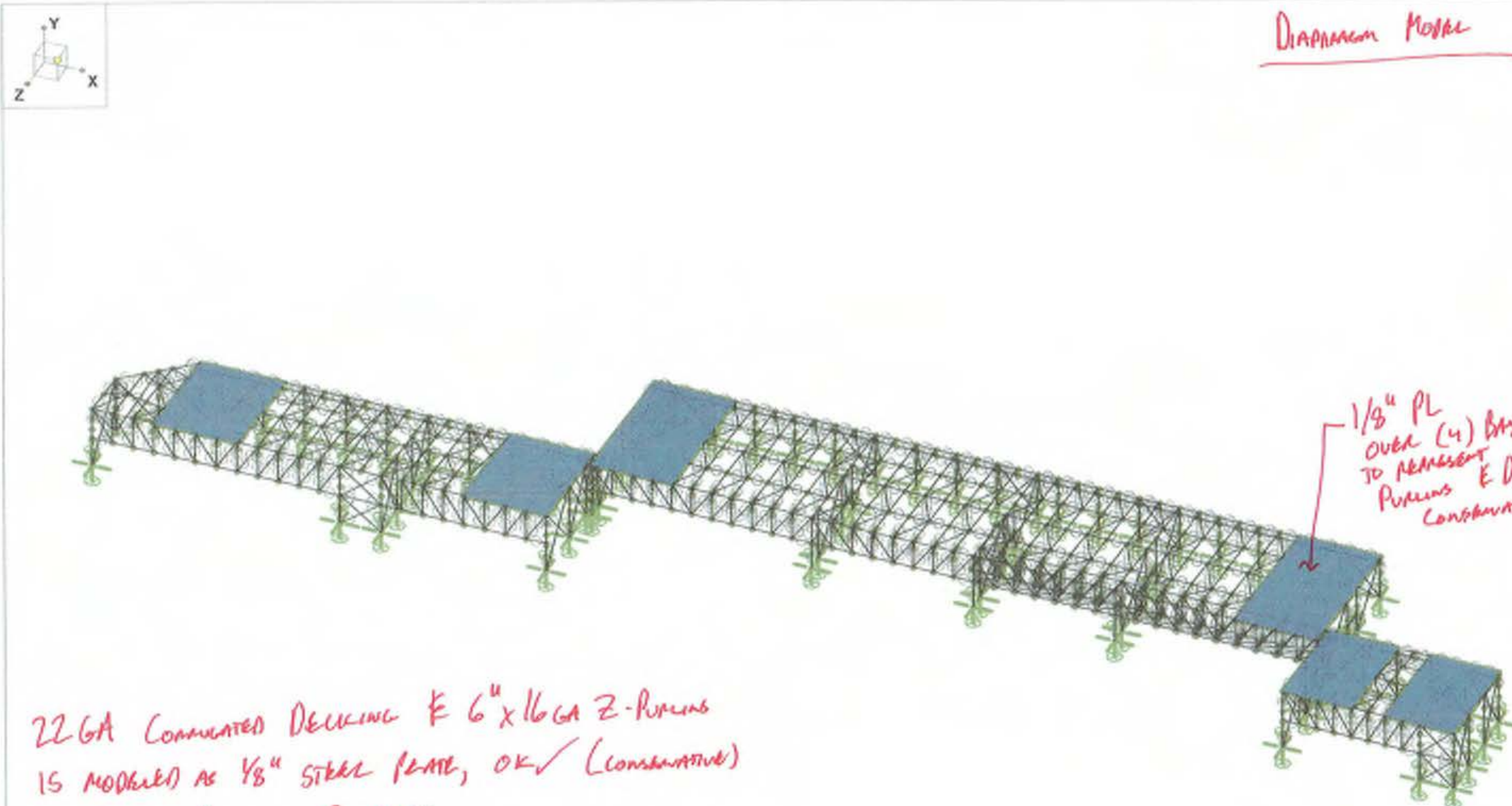
c. See Section 2403 for glass supports.

d. The deflection limit for the $D+L$ load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For wood structural members that are dry at time of installation and used under dry conditions in accordance with the ANSI/AWC NDS, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from $0.5D$. For wood structural members at all other moisture conditions, the creep component of the long-term deflection is permitted to be estimated as the immediate dead load deflection resulting from D. The value of $0.5D$ shall not be used in combination with ANSI/AWC NDS provisions for long-term loading.

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to ensure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1503.4 for roof drainage requirements.

f. The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein. Where members support glass in accordance with Section 2403 using the deflection limit therein, the wind load shall be no less than 0.6 times the "component and cladding" loads for the purpose of determining deflection.

DIAPHRAGM MODEL



22 GA CORRUGATED DECKING & 6" x 16 GA Z-PURLINS
IS MODELED AS 1/8" STEEL PLATE, OK✓ (CONSERVATIVE)
TO PROVIDE DEFLECTION ENVELOPE

Envelope Only Solution

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK -
BS		Nov 2, 2018 at 4:47 PM
18-183B		BLDG 123 Model Rev0 11.1.18 w diaphragm.r3d

MEMBER & CONNECTION DESIGN: AGGREGATE STORAGE BUILDING 1-3

MAIN COLUMN: W24x104

PER RISA, DR = 0.31 MAX, < 1.0 OK ✓

BASEPLATE / ANCHORAGE DESIGN

LRFD REACTIONS

WIND DEMAND FAR EXCEEDS SEISMIC, NO OVERSTRENGTH FACTOR REQUIRED

TENSION = 42 KIPS

V_y = 27 KIPS
V_x = 26 KIPS
) DO NOT OCCUR SIMULTANEOUSLY W/ MAX TENSION

USE 1" BASE PLATE W/ (6) 1"Ø ANCHORS W/ 10" EMBED

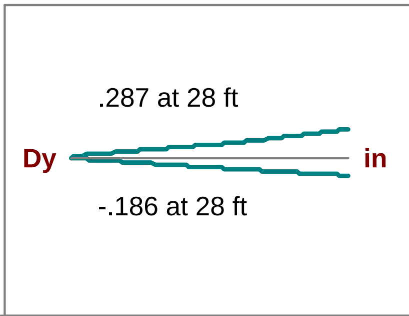
MAIN COLUMN

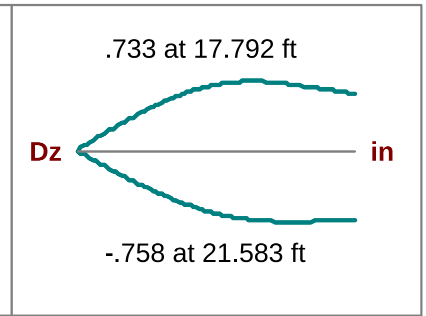
Column:	*M918*
Shape:	**W24x104**
Material:	**A572 Gr.50**
Length:	**28 ft**
I Joint:	**N668**
J Joint:	**N91**

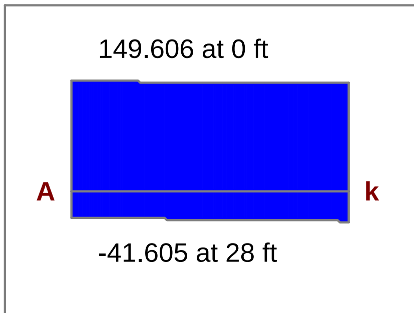
Envelope

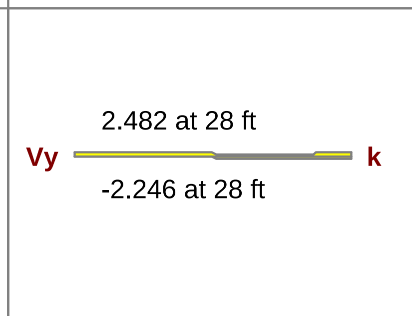
Code Check: **0.314 (LC 27)**

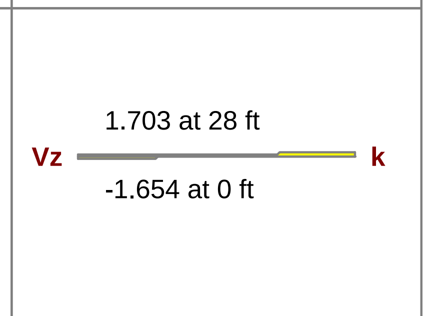
Report Based On 97 Sections

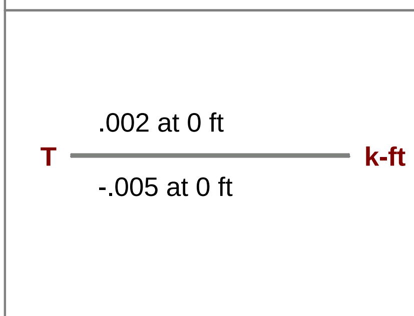


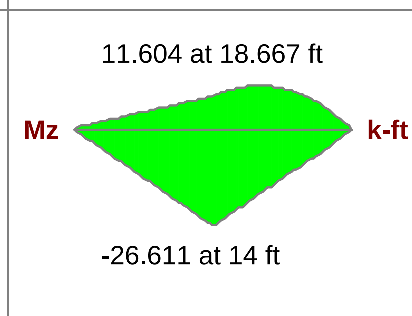


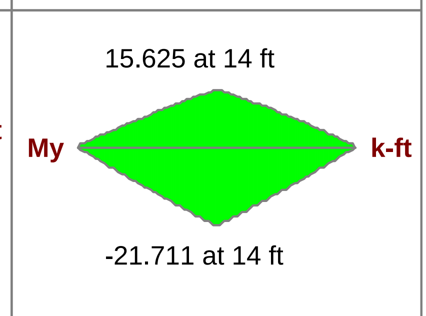


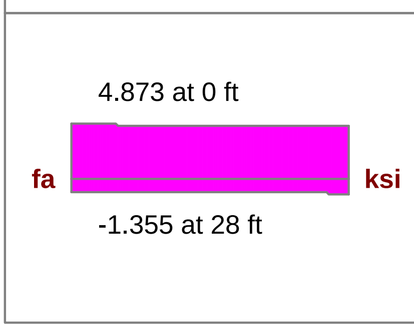


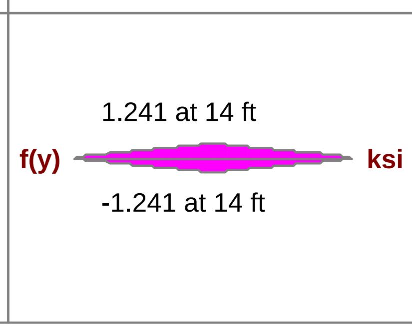


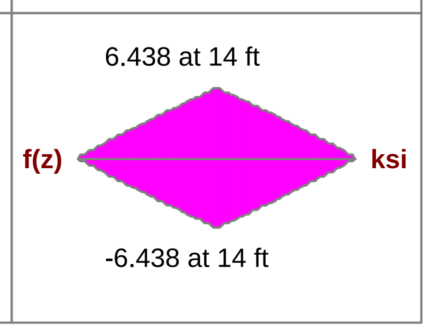












AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.314 (LC 27)**	Max Shear Check	**0.010 (y) (LC 24)**
Location	**14 ft**	Location	**28 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/1170**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.97**

			y-y	z-z
Fy	**50 ksi**	Lb	**14 ft**	**14 ft**
Pnc/om	**703.449 k**	KL/r	**57.84**	**16.719**
Pnt/om	**919.162 k**			
Mny/om	**155.689 k-ft**	L Comp Flange	**14 ft**	
Mnz/om	**667.527 k-ft**	L-torque	**28 ft**	
Vny/om	**241 k**	Tau_b	**1**	
Vnz/om	**344.91 k**			
Cb	**1**			

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	10/1/2018
E-Mail:			

Specifier's comments:

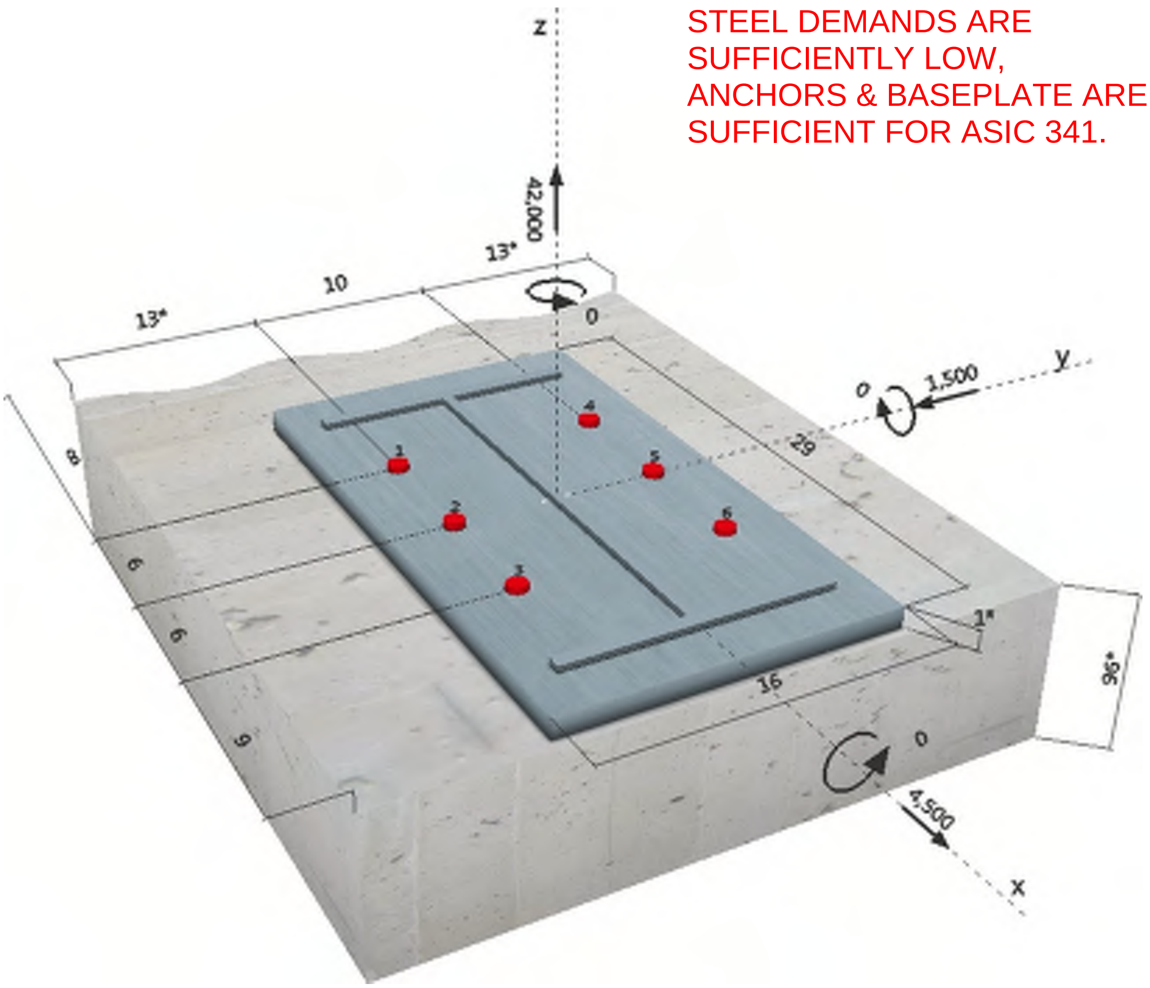
1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 1**
Effective embedment depth:	h_{ef} = 10.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate:	l_x x l_y x t = 29.000 in. x 16.000 in. x 1.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 24.100 in. x 12.800 in. x 0.500 in. x 0.750 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 96.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar with stirrups

R - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)

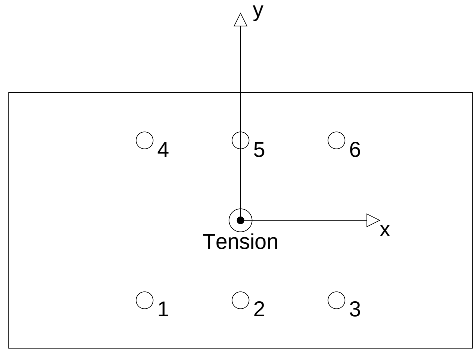
Geometry [in.] & Loading [lb, in.lb]

STEEL DEMANDS ARE SUFFICIENTLY LOW, ANCHORS & BASEPLATE ARE SUFFICIENT FOR ASIC 341.



2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	7,000	791	750	-250
2	7,000	791	750	-250
3	7,000	791	750	-250
4	7,000	791	750	-250
5	7,000	791	750	-250
6	7,000	791	750	-250

max. concrete compressive strain:	- [‰]
max. concrete compressive stress:	- [psi]
resulting tension force in (x/y)=(0.000/0.000):	42,000 [lb]
resulting compression force in (x/y)=(0.000/0.000):	0 [lb]

Anchor forces based on a rigid base plate assumption!

3 Tension load

	Load N_{ua} [lb]	Capacity $\phi\ N_n$ [lb]	Utilization $\beta_N = N_{ua}/\phi\ N_n$	Status
Steel Strength*	7,000	26,361	27	OK
Pullout Strength*	7,000	33,622	21	OK
Concrete Breakout Strength**	42,000	44,554	95	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\ f_{uta}$ ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	7,000

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,000

Calculations

N_p [lb]
48,032

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
48,032	0.700	33,622	7,000

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b$ ACI 318-14 Eq. (17.4.2.1b)
$\phi N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)
$A_{Nc0} = 9 h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
8.667	0.000	0.000	9.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,224.00	676.00	1.000	1.000	0.908	1.000	38,727

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
63,649	0.700	44,554	42,000

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	791	13,708	6	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	4,743	89,109	6	OK
Concrete edge failure in direction x+**	4,743	19,859	24	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
21,089	0.650	13,708	791

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.667	0.000	0.000	9.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,224.00	676.00	1.000	1.000	0.908	1.000	38,727

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
127,298	0.700	89,109	4,743

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-14 Eq. (17.5.2.8)

$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ — ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
9.000	13.000	0.000	1.400	96.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
486.00	364.50	1.000	0.989	1.000	15,369

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
28,369	0.700	19,859	4,743

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.943	0.239	1.000	99	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

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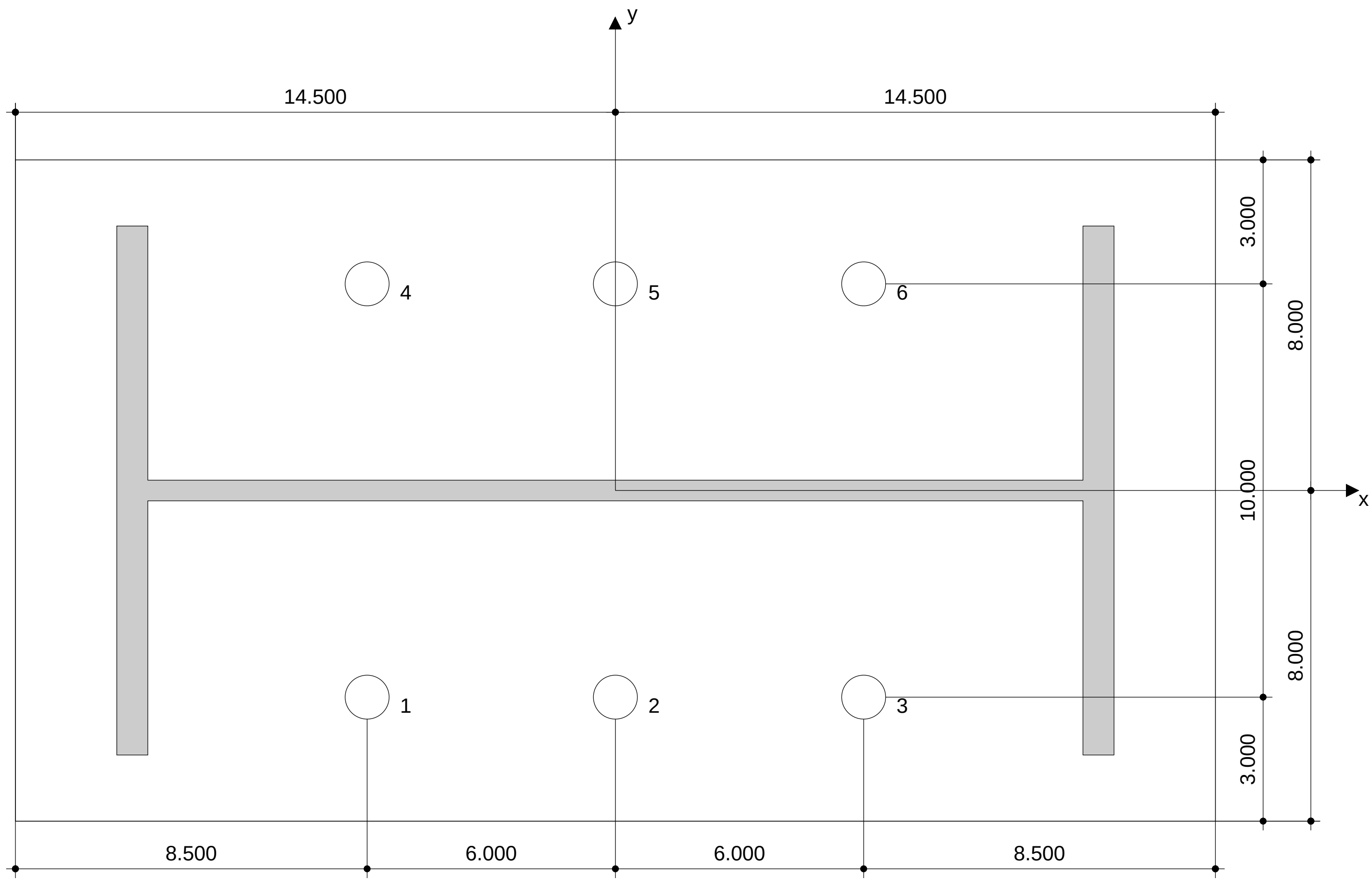
7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 24.100 x 12.800 x 0.500 x 0.750 in.
Hole diameter in the fixture: d_f = 1.063 in.
Plate thickness (input): 1.000 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 1
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 10.000 in.
Minimum thickness of the base material: 11.172 in.

R - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-6.000	-5.000	-	21.000	13.000	23.000
2	0.000	-5.000	-	15.000	13.000	23.000
3	6.000	-5.000	-	9.000	13.000	23.000

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	-6.000	5.000	-	21.000	23.000	13.000
5	0.000	5.000	-	15.000	23.000	13.000
6	6.000	5.000	-	9.000	23.000	13.000

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8 Remarks; Your Cooperation Duties

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Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

Moment Frame Column: W36x194

Per Risa, Max. DR = 0.37 < 1.0 OK ✓

Base Plate / Anchorage Design

LRFD Rxns

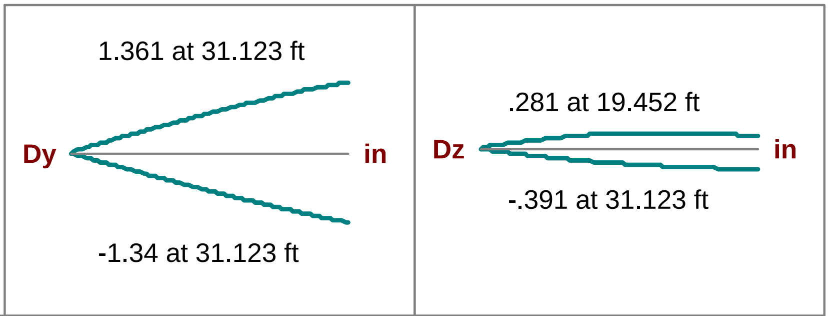
Tension = 15 kips

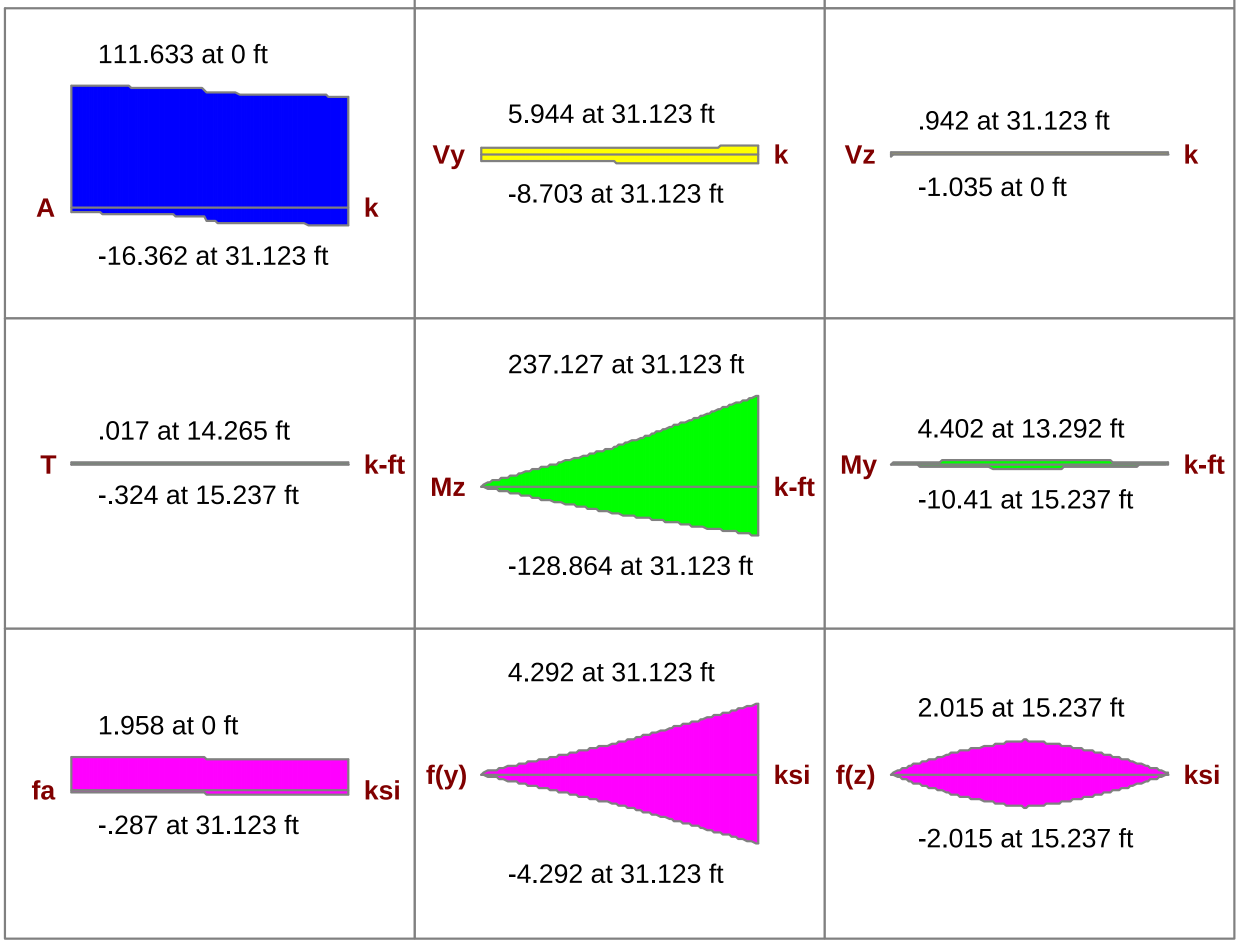
WIND DEMAND FAR EXCEEDS SEISMIC, NO OVERSTRENGTH FACTOR REQUIRED

V_y = 13 kips
V_x = 34 kips ← @ LC37, Controls

Column: *M846A*

Shape: **W36x194**
Material: **A572 Gr.50**
Length: **31.123 ft**
I Joint: **N582A**
J Joint: **N158_1**
Envelope
Code Check: **0.367 (LC 27)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.367 (LC 27)**	Max Shear Check	**0.027 (y) (LC 27)**
Location	**31.123 ft**	Location	**31.123 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/553**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**50 ksi**	Lb	**31.123 ft**	**31.123 ft**
Pnc/om	**404.108 k**	KL/r	**145.606**	**25.633**
Pnt/om	**1706.587 k**			
Mny/om	**243.762 k-ft**	L Comp Flange	**31.123 ft**	
Mnz/om	**1710.808 k-ft**	L-torque	**31.123 ft**	
Vny/om	**558.45 k**	Tau_b	**1**	
Vnz/om	**547.76 k**			
Cb	**1.774**			

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E-Mail:			

Specifier's comments:

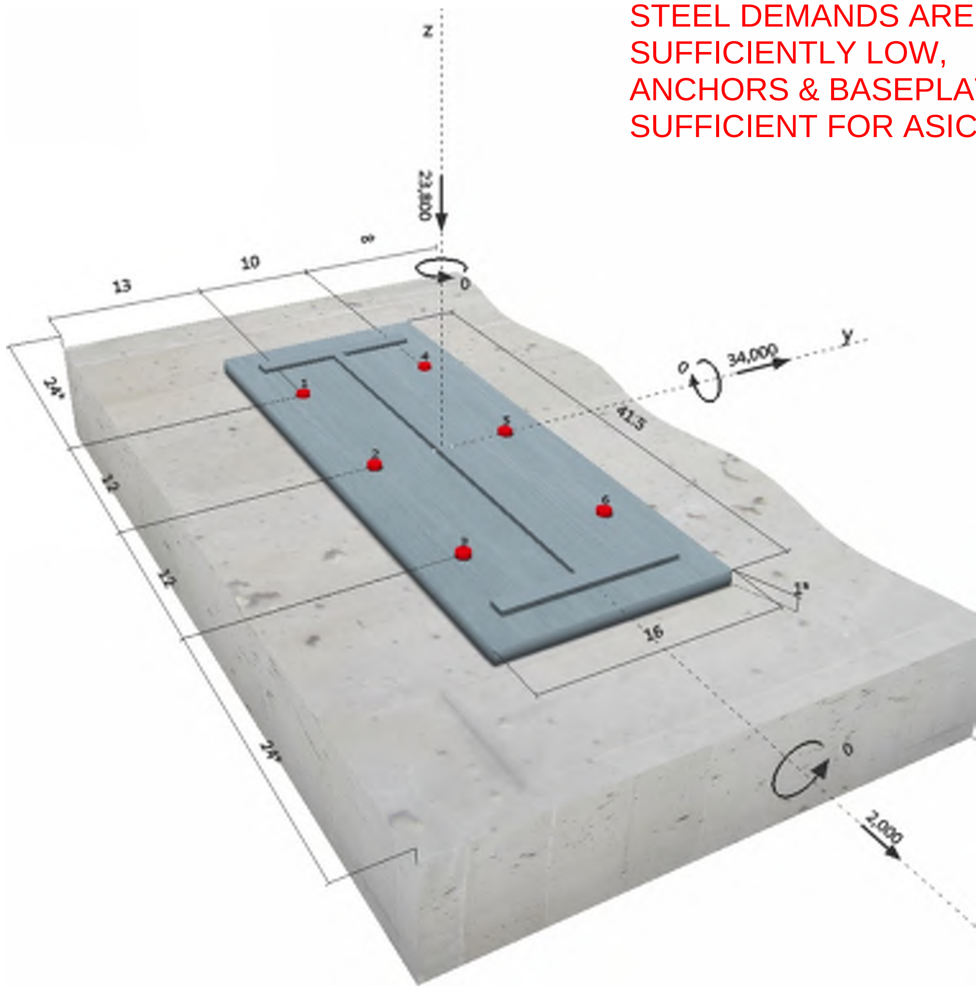
1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 1**
Effective embedment depth:	h_{ef} = 12.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate:	l_x x l_y x t = 41.500 in. x 16.000 in. x 1.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 36.500 in. x 12.100 in. x 0.765 in. x 1.260 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 108.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar

R - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)

Geometry [in.] & Loading [lb, in.lb]

STEEL DEMANDS ARE SUFFICIENTLY LOW, ANCHORS & BASEPLATE ARE SUFFICIENT FOR ASIC 341.



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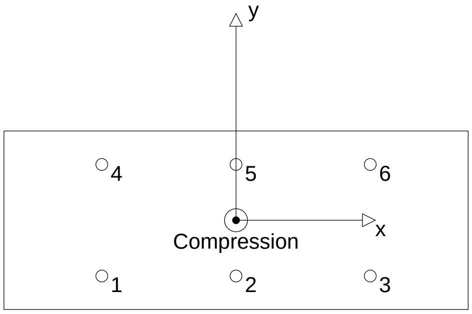
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	5,676	333	5,667
2	0	5,676	333	5,667
3	0	5,676	333	5,667
4	0	5,676	333	5,667
5	0	5,676	333	5,667
6	0	5,676	333	5,667



max. concrete compressive strain: 0.01 [‰]
max. concrete compressive stress: 36 [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 23,800 [lb]

Anchor forces based on a rigid base plate assumption!

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	5,676	13,708	42	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	34,059	155,044	22	OK
Concrete edge failure in direction x+**	34,059	37,237	92	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
21,089	0.650	13,708	5,676

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)

$\phi\, V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)

$N_b = 16\, \lambda_a \sqrt{f_c'}\, h_{ef}^{5/3}$ ACI 318-14 Eq. (17.4.2.2b)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	12.000	0.000	0.000	13.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	16	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,460.00	1,296.00	1.000	1.000	0.917	1.000	63,648

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕ V_{cpg} [lb]	V_{ua} [lb]
221,491	0.700	155,044	34,059

4.3 Concrete edge failure in direction x+

V_{cbg}	$= \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$	ACI 318-14 Eq. (17.5.2.1b)
$\phi\ V_{cbg} \geq V_{ua}$		ACI 318-14 Table 17.3.1.1
A_{Vc}	see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)	
A_{Vc0}	$= 4.5\ c_{a1}^2$	ACI 318-14 Eq. (17.5.2.1c)
$\psi_{ec,V}$	$= \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.5)
$\psi_{ed,V}$	$= 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.6b)
$\psi_{h,V}$	$= \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-14 Eq. (17.5.2.8)
V_b	$= 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$	ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
24.000	13.000	0.000	1.200	108.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
2,124.00	2,592.00	1.000	0.808	1.000	66,925

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
53,196	0.700	37,237	34,059

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

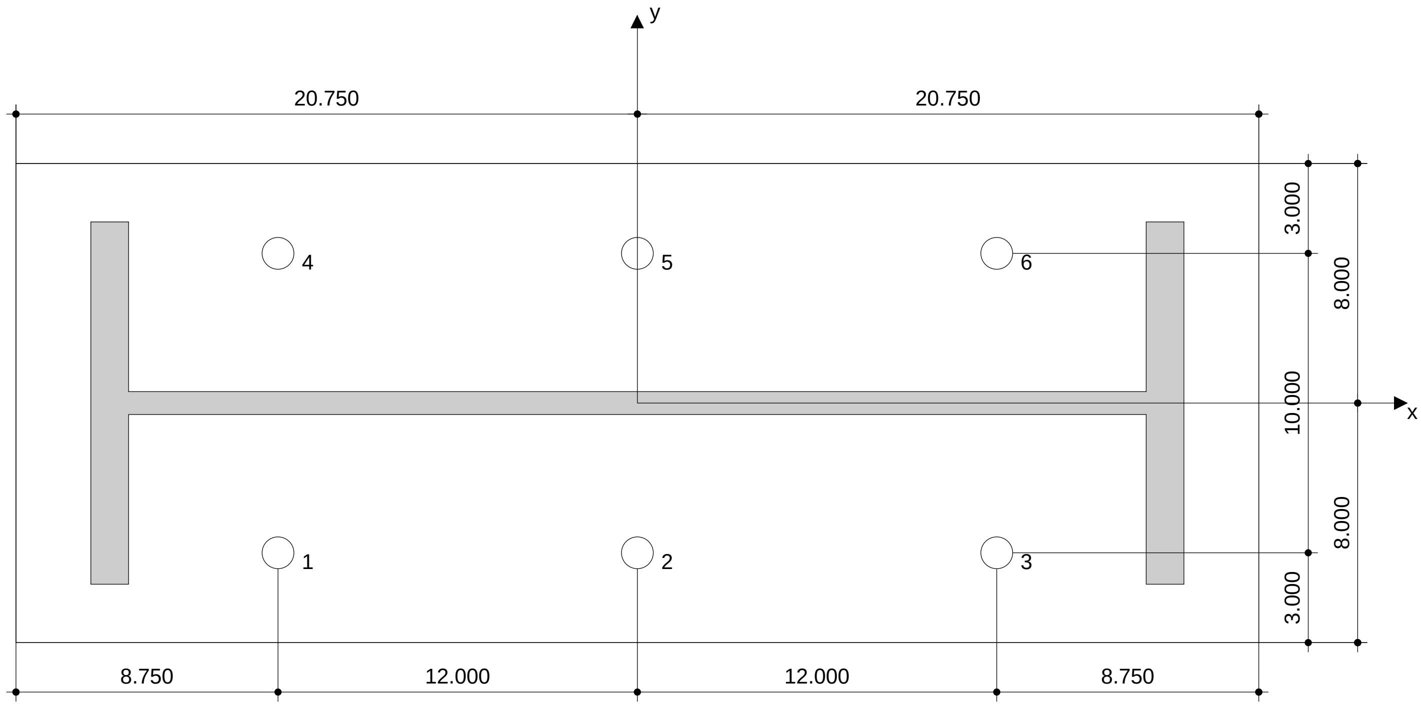
Fastening meets the design criteria!

6 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 36.500 x 12.100 x 0.765 x 1.260 in.
Hole diameter in the fixture: d_f = 1.063 in.
Plate thickness (input): 1.000 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 1
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 12.000 in.
Minimum thickness of the base material: 13.172 in.

R - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-12.000	-5.000	24.000	48.000	13.000	-
2	0.000	-5.000	36.000	36.000	13.000	-
3	12.000	-5.000	48.000	24.000	13.000	-

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
4	-12.000	5.000	24.000	48.000	23.000	-
5	0.000	5.000	36.000	36.000	23.000	-
6	12.000	5.000	48.000	24.000	23.000	-

7 Remarks; Your Cooperation Duties

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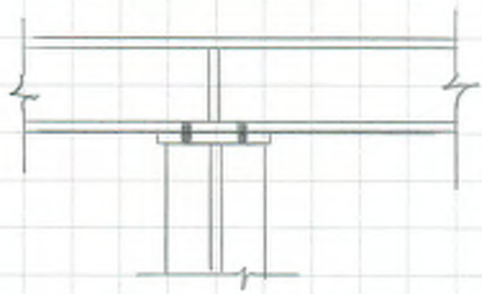
Typical Column: W12x65

Per Risa, D_R = 0.48 MAX. < 1.0 OK ✓

Connection Design



Max. Rxns

Tension = 29 kips @ LC24

V_x = 25 kips @ LC19
V_y = 8 kips @ LC24

Seismic Rxns (2.0 Overstrength Factor)

* Controls Over Other Rxns

Tension = 16 kips @ LC41 x2 = 32 kips

V_x = 22 kips @ LC 50 x2 = 44 kips
V_y = 3 kips @ LC42 x2 = 6 kips

These Do Not Occur Simultaneously

Try 1/2" PL w/(4) 3/4"Ø Bolts

T_n/Ω = 19.9^k x4 = 79.6 kips D_R = 0.40 < 1.0 OK ✓

V_n/Ω = 11.9^k x4 = 47.6 kips D_R = 0.92 < 1.0 OK ✓

$\boxed{\text{Use 1/2" PL w/(4) 3/4"Ø Bolts}}$

Base Plate / Anchorage Design

LRFD Rxns

Tension = 27 kips
V_y = 29 kips
V_x = 20 kips

WIND DEMAND FAR EXCEEDS SEISMIC, NO OVERSTRENGTH REQUIRED

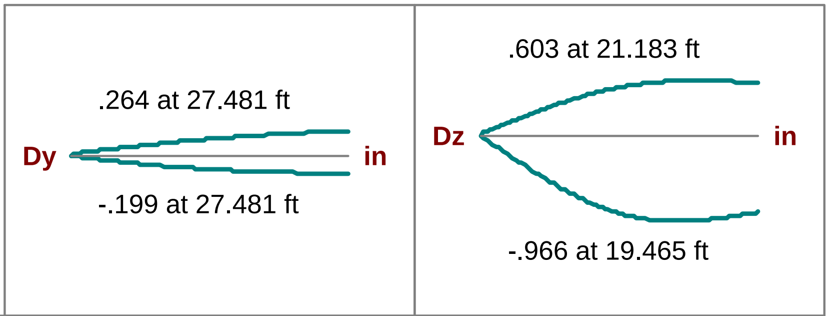
Column: ***M976B***

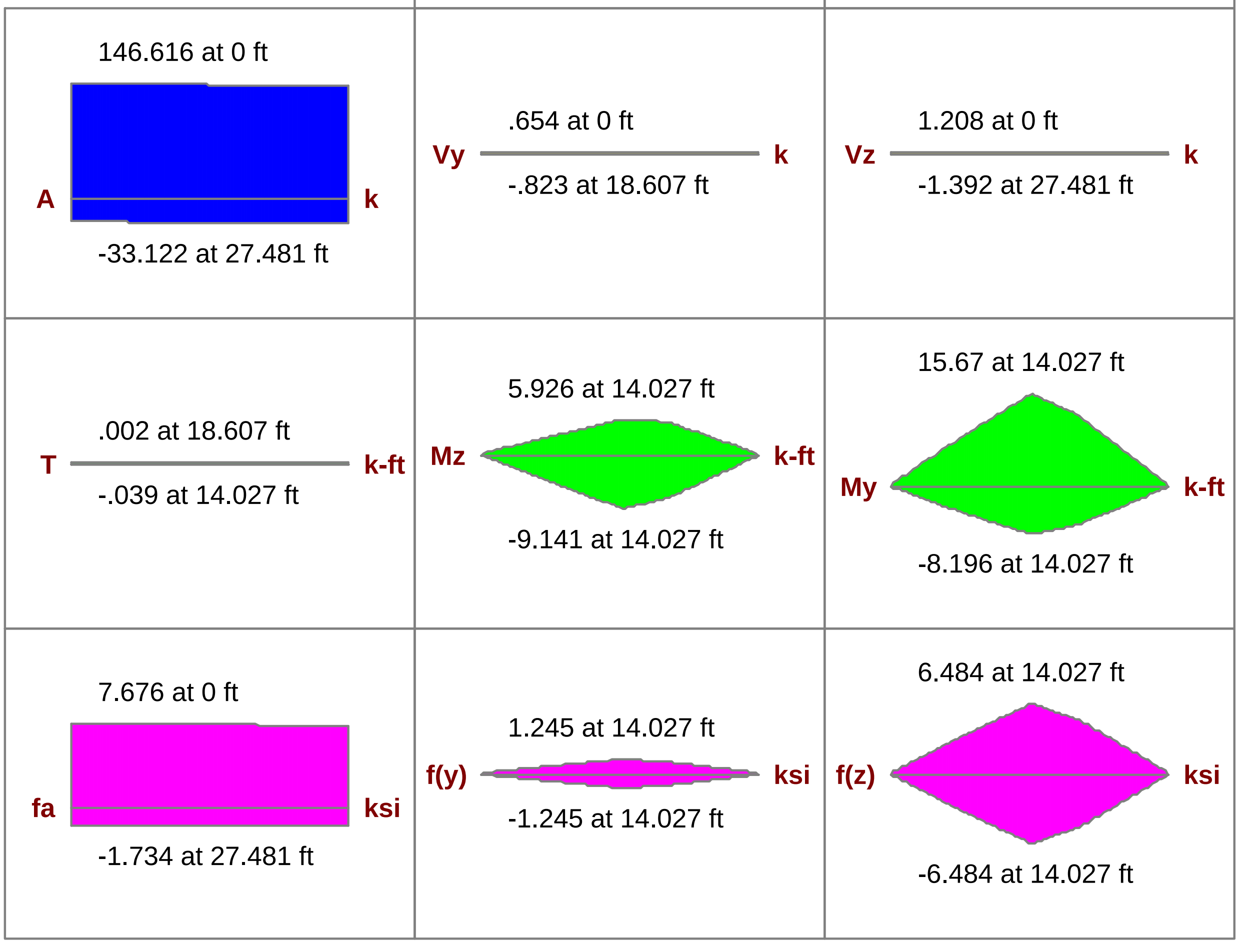
Shape:	**W12x65**
Material:	**A572 Gr.50**
Length:	**27.481 ft**
I Joint:	**N785A**
J Joint:	**N786A**

Envelope

Code Check: **0.479 (LC 33)**

Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.479 (LC 33)**	Max Shear Check	**0.013 (z) (LC 38)**
Location	**14.027 ft**	Location	**27.481 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/3123**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**14 ft**	**14 ft**
Pnc/om	**393.304 k**	KL/r	**55.661**	**31.803**
Pnt/om	**571.856 k**			
Mny/om	**106.985 k-ft**	L Comp Flange	**14 ft**	
Mnz/om	**229.525 k-ft**	L-torque	**27.481 ft**	
Vny/om	**94.38 k**	Tau_b	**1**	
Vnz/om	**260.838 k**			
Cb	**1**			

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Specifier's comments:

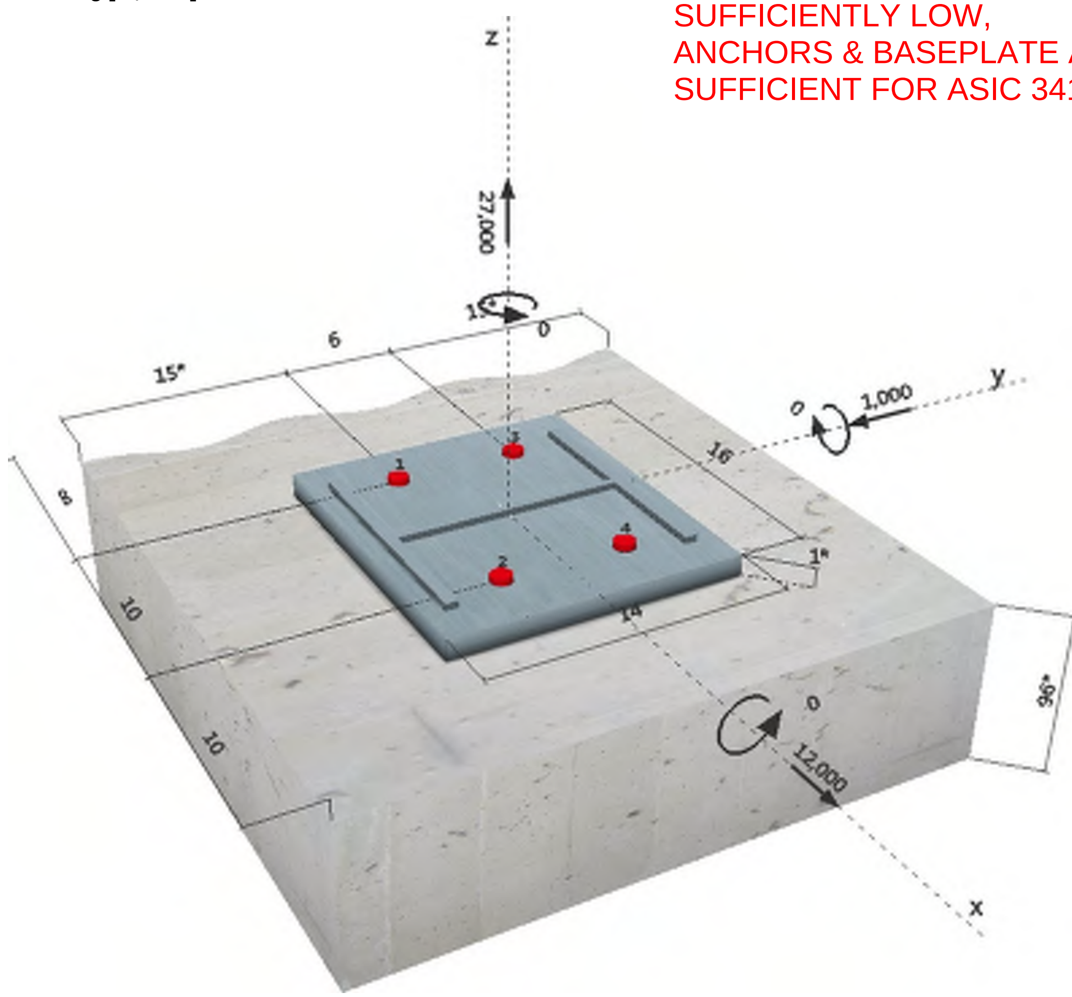
1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 1**
Effective embedment depth:	h_{ef} = 12.000 in.
Material:	ASTM F 1554
Proof:	Design method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.000 in.
Anchor plate:	l_x x l_y x t = 16.000 in. x 14.000 in. x 1.000 in.; (Recommended plate thickness: not calculated
Profile:	W shape (AISC); (L x W x T x FT) = 12.100 in. x 12.000 in. x 0.390 in. x 0.605 in.
Base material:	cracked concrete, 4000, f_c' = 4,000 psi; h = 96.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: >= No. 4 bar with stirrups

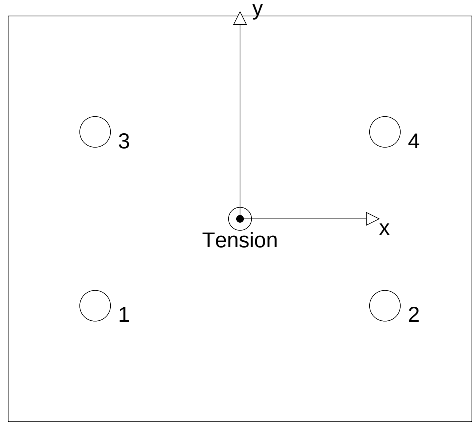
[R] - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)

Geometry [in.] & Loading [lb, in.lb]

STEEL DEMANDS ARE SUFFICIENTLY LOW, ANCHORS & BASEPLATE ARE SUFFICIENT FOR ASIC 341.



2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	6,750	3,010	3,000	-250
2	6,750	3,010	3,000	-250
3	6,750	3,010	3,000	-250
4	6,750	3,010	3,000	-250

max. concrete compressive strain:	- [‰]
max. concrete compressive stress:	- [psi]
resulting tension force in (x/y)=(0.000/0.000):	27,000 [lb]
resulting compression force in (x/y)=(0.000/0.000):	0 [lb]

Anchor forces based on a rigid base plate assumption!

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	6,750	26,361	26	OK
Pullout Strength*	6,750	33,622	21	OK
Concrete Breakout Strength**	27,000	42,336	64	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$	ACI 318-14 Eq. (17.4.1.2)
$\phi\ N_{sa} \geq N_{ua}$	ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,148

Results

N_{sa} [lb]	$\phi_{\ steel}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,148	0.750	26,361	6,750

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-14 Eq. (17.4.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-14 Eq. (17.4.3.4)
$\phi N_{pN} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	1.50	1.000	4,000

Calculations

N_p [lb]
48,032

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
48,032	0.700	33,622	6,750

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$ ACI 318-14 Eq. (17.4.2.1b)
$\phi N_{cbg} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)
$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e'_N}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\, \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
10.000	0.000	0.000	10.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,260.00	900.00	1.000	1.000	0.900	1.000	48,000

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
60,480	0.700	42,336	27,000

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	3,010	13,708	22	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	12,042	84,672	15	OK
Concrete edge failure in direction x+**	12,042	21,168	57	OK

* anchor having the highest loading **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-14 Eq. (17.5.1.2b)
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

V_{sa} [lb]
21,089

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
21,089	0.650	13,708	3,010

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)
$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)
$A_{Nc0} = 9\, h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	10.000	0.000	0.000	10.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	-	24	1.000	4,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,260.00	900.00	1.000	1.000	0.900	1.000	48,000

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
120,960	0.700	84,672	12,042

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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$	ACI 318-14 Eq. (17.5.2.1b)
$\phi\ V_{cbg} \geq V_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)	
$A_{Vc0} = 4.5\ c_{a1}^2$	ACI 318-14 Eq. (17.5.2.1c)
$\psi_{ec,V} = \left(\frac{1}{1 + \frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.5)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-14 Eq. (17.5.2.6b)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-14 Eq. (17.5.2.8)
$V_b = 9\ \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$	ACI 318-14 Eq. (17.5.2.2b)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
10.000	15.000	0.000	1.400	96.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
8.000	1.000	1.000	4,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
540.00	450.00	1.000	1.000	1.000	18,000

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
30,240	0.700	21,168	12,042

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.638	0.569	5/3	87	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

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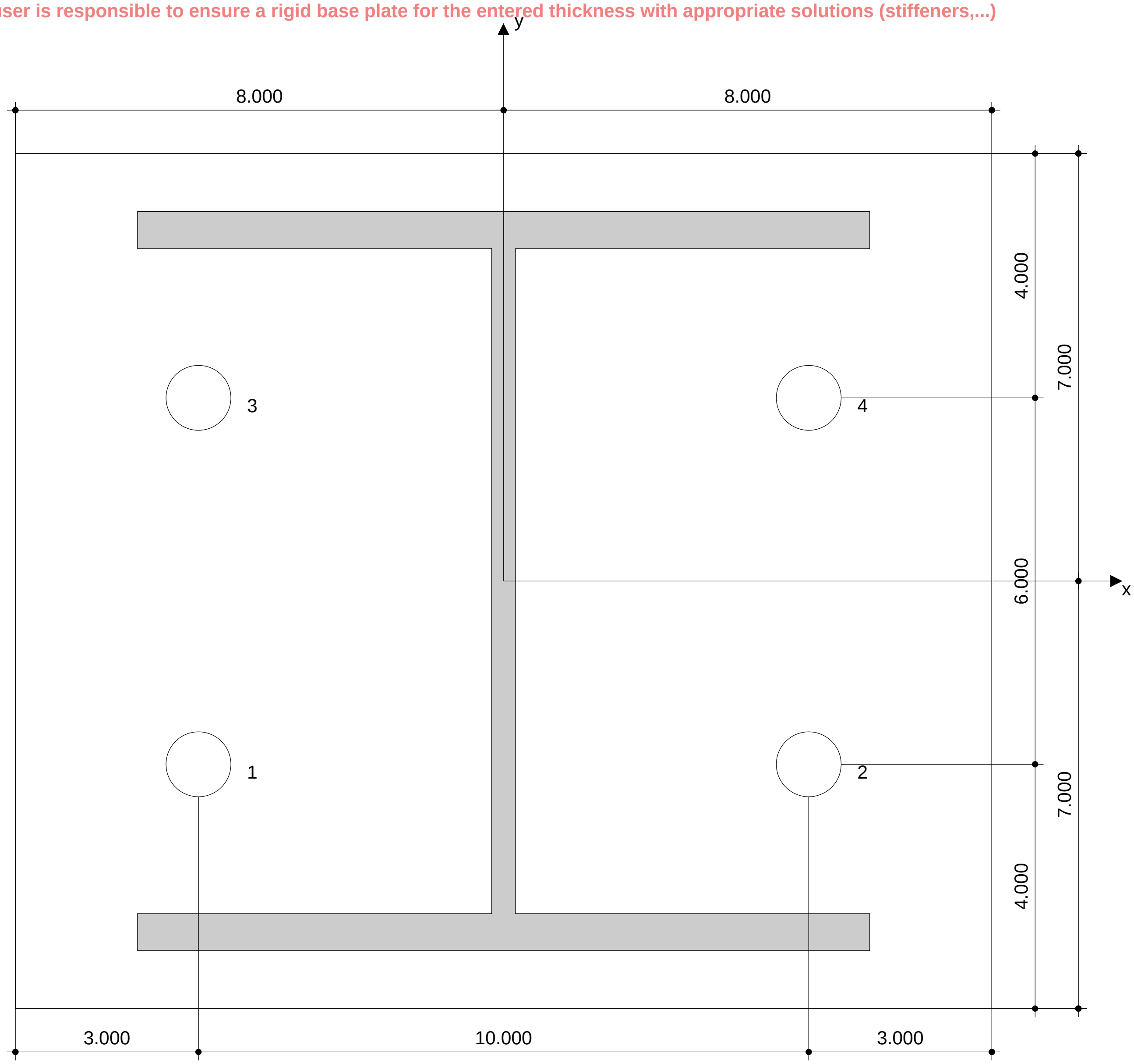
7 Installation data

Anchor plate, steel: -
Profile: W shape (AISC); 12.100 x 12.000 x 0.390 x 0.605 in.
Hole diameter in the fixture: d_f = 1.063 in.
Plate thickness (input): 1.000 in.
Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 1
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 12.000 in.
Minimum thickness of the base material: 13.172 in.

R - user is responsible to ensure a rigid base plate for the entered thickness with appropriate solutions (stiffeners,...)



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.000	-3.000	-	20.000	15.000	21.000
2	5.000	-3.000	-	10.000	15.000	21.000
3	-5.000	3.000	-	20.000	21.000	15.000
4	5.000	3.000	-	10.000	21.000	15.000

8 Remarks; Your Cooperation Duties

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Smith Monroe Gray
ENGINEERS, INC.

CLIENT

PROJECT

BY DATE REV.

JOB NO. SHEET OF

Typ. Transverse Beam - W24x104

Max DR = 0.78 < 1.0 OK✓

Max Rxns

Compression = 13^k
Tension = 11^k
$Shear_x$ = 16^k
$Shear_y$ = 17^k

Typical (7) 3/4"Ø Bolt Cxn OK By Inspection

* ADD CJP Flange Weld @ Moment Cxn OK✓

USE CJP MOMENT FRAME CONNECTION PER AISC 341 E1.6

6. Connections

Beam-to-column connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections in accordance with this section.

6b. FR Moment Connections

FR moment connections that are part of the *seismic force resisting system* (SFRS) shall satisfy at least one of the following requirements:

(c) FR moment connections between wide flange beams and the flange of wide flange columns shall either satisfy the requirements of Section E2.6 or E3.6, or shall satisfy the following requirements:

(1) All welds at the beam-to-column connection shall satisfy the requirements of Chapter 3 of ANSI/AISC 358.

(2) Beam flanges shall be connected to column flanges using complete-joint-penetration (CJP) groove welds.

(3) The shape of weld access holes shall be in accordance with subclause 6.10.1.2 of AWS D1.8/D1.8M. Weld access hole quality requirements shall be in accordance with subclause 6.10.2 of AWS D1.8/D1.8M.

(4) Continuity plates shall satisfy the requirements of Section E3.6f.

Exception: The welded joints of the continuity plates to the column flanges are permitted to be complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds with reinforcement, or two-sided fillet welds. The *required strength* of these joints shall not be less than the *available strength* of the contact area of the plate with the column flange.

(5) The beam web shall be connected to the column flange using either a CJP groove weld extending between weld access holes, or using a bolted single plate shear connection designed for required shear strength per Equation E1-1.

(2) Continuity Plate Thickness

Where continuity plates are required, the thickness of the plates shall be determined as follows:

(a) For one-sided connections, continuity plate thickness shall be at least one-half of the thickness of the beam flange.

(b) For two-sided connections, the continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column.

Continuity plates shall also conform to the requirements of Section J10 of the *Specification*.

(3) Continuity Plate Welding

Continuity plates shall be welded to column flanges using CJP groove welds.

Continuity plates shall be welded to column webs using groove welds or fillet welds. The required strength of the sum of the welded joints of the continuity plates to the column web shall be the smallest of the following:

(a) The sum of the *design strengths* in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges

(b) The design strength in shear of the contact area of the plate with the column web

(c) The design strength in shear of the column panel zone

(d) The sum of the *expected yield strengths* of the beam flanges transmitting force to the continuity plates

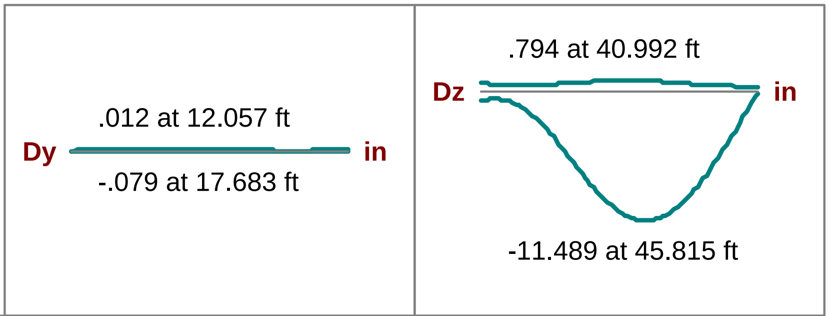
Beam: ***M396A***

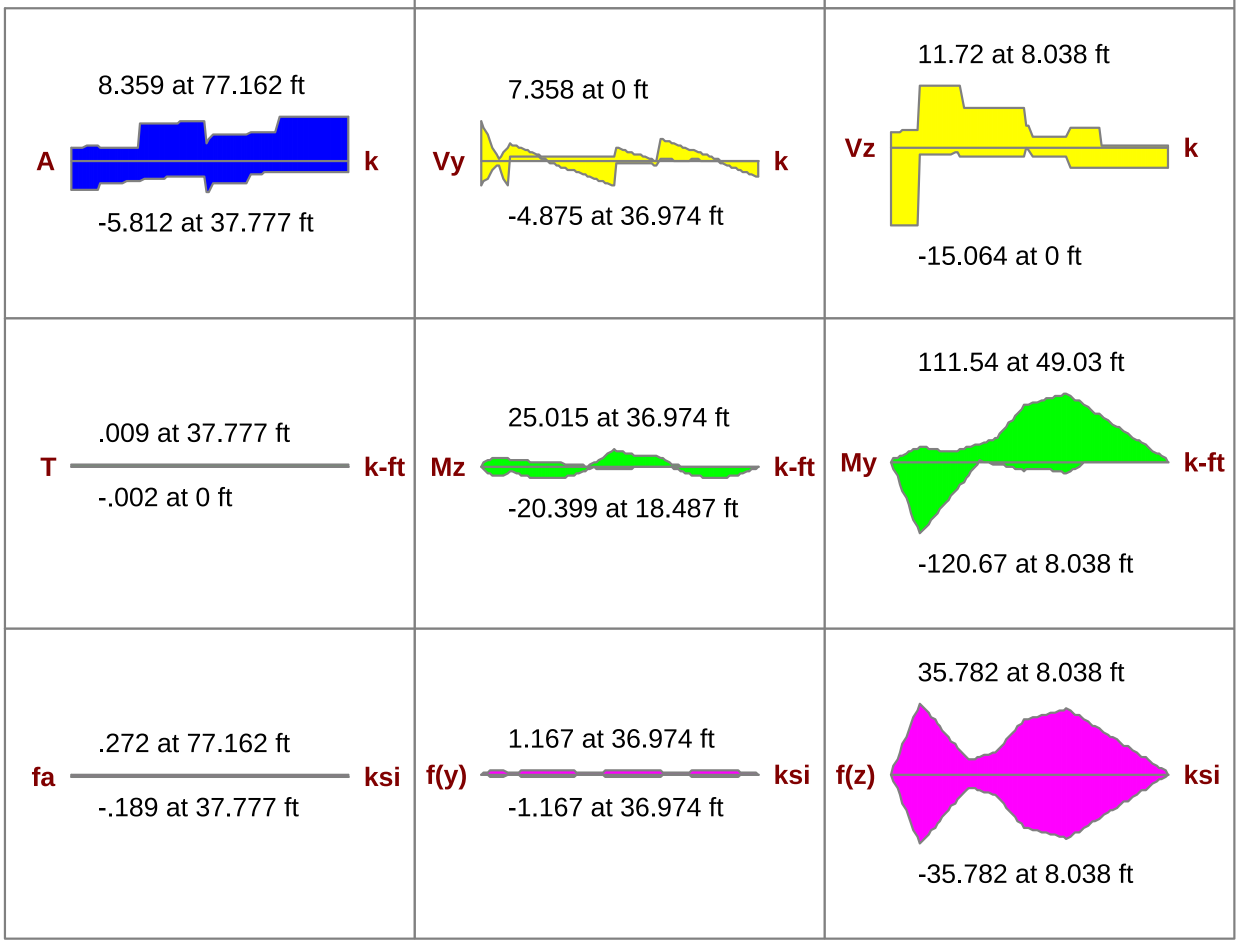
Shape:	**W24x104**
Material:	**A572 Gr.50**
Length:	**77.162 ft**
I Joint:	**N12**
J Joint:	**N17**

Envelope

Code Check: **0.781 (LC 21)**

Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.781 (LC 21)**	Max Shear Check	**0.044 (z) (LC 21)**	Max Defl Ratio	**L/9114**
Location	**49.03 ft**	Location	**0 ft**	Location	**20.094 ft**
Equation	**H1-1b**			Span	**2**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.945**

			y-y	z-z
Fy	**50 ksi**	Lb	**77.162 ft**	**77.162 ft**
Pnc/om	**45.406 k**	KL/r	**318.79**	**92.145**
Pnt/om	**919.162 k**			
Mny/om	**155.689 k-ft**	L Comp Flange	**77.162 ft**	
Mnz/om	**145.788 k-ft**	L-torque	**77.162 ft**	
Vny/om	**241 k**	Tau_b	**1**	
Vnz/om	**344.91 k**			
Cb	**1.238**			

Long Beam: W12x65

Per RISA, DR = 0.59 < 1.0 OK✓

* Seismic Controls w/ Overstrength

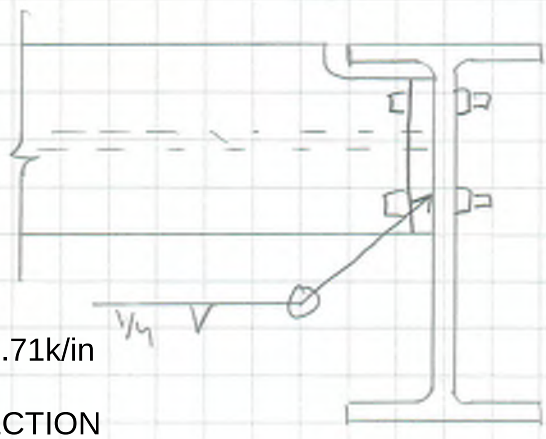
Tension = 24 kips x 2.0 = 48 kips
Compression = 27 kips x 2.0 = 54 kips

Shear = 8.0 kips x 2.0 = 16 kips

Connection Design

Use 1/2" End PL w/ (4) 3/4"⌀ Bolts

T_n/Ω = 19.9k x 4 = 79.6k OK✓



MIN. 1/4" WELD LENGTH REQ'D = (48k + 16k)/3.71k/in
= 17.3"
(2) WEBS = ~20" WELD LENGTH, OK BY INSPECTION

Truss Bottom Support

* Seismic Demand Controls

Axial = 6k x 2 = 12k
Shear = 3k x 2 = 6k

$\boxed{\text{Dbl 3/8" Angle w/ (2) 3/4" ⌀ Bolts OK By Insp.}}$

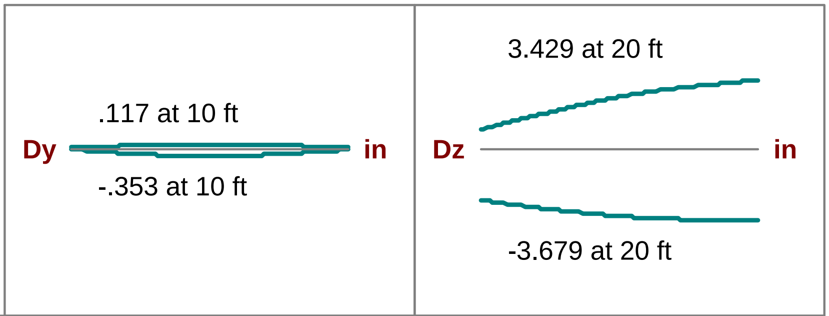
Beam: ***M912C***

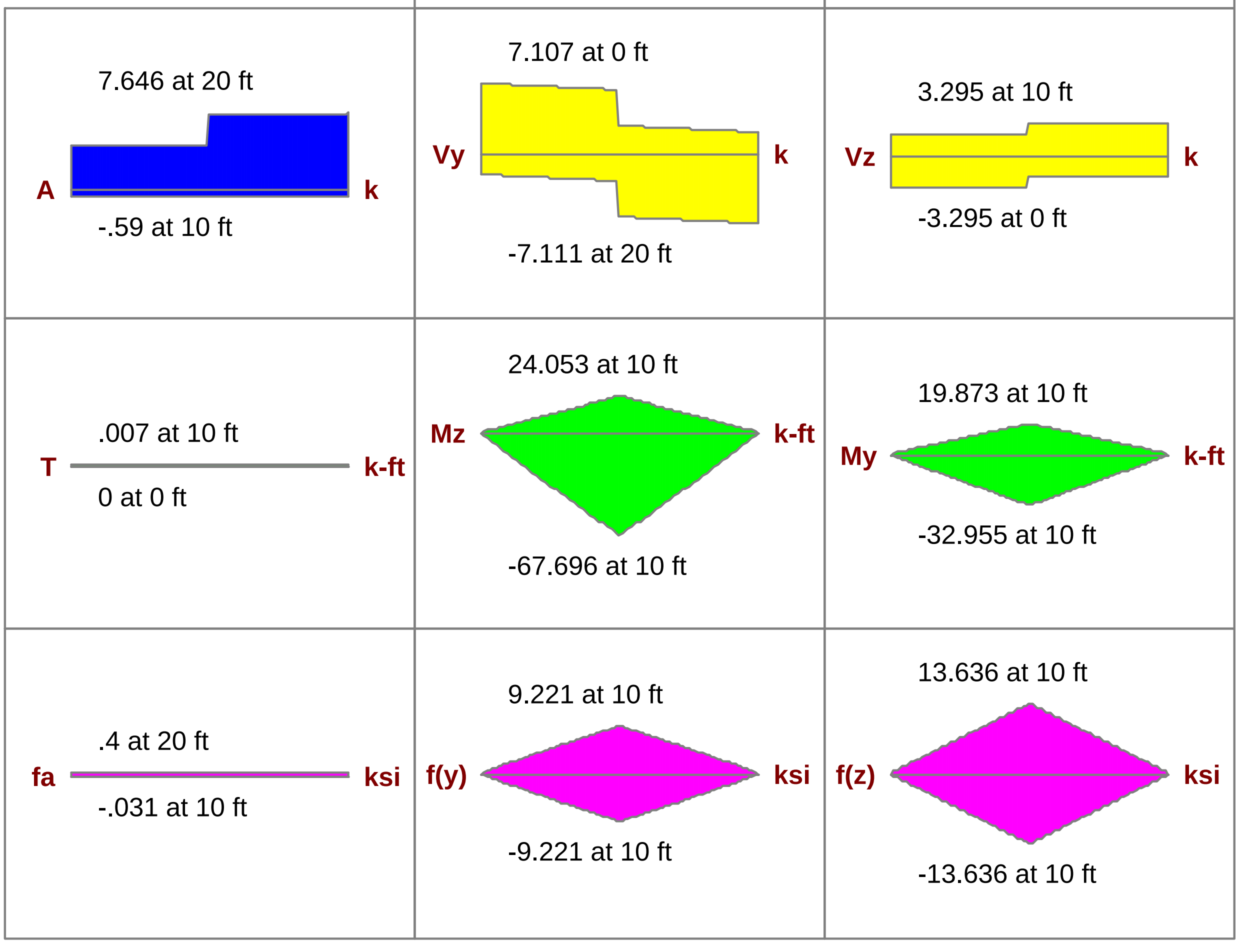
Shape:	**W12x65**
Material:	**A572 Gr.50**
Length:	**20 ft**
I Joint:	**N88**
J Joint:	**N97**

Envelope

Code Check: **0.597 (LC 27)**

Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.597 (LC 27)**	Max Shear Check	**0.075 (y) (LC 33)**	Max Defl Ratio	**L/711**
Location	**10 ft**	Location	**20 ft**	Location	**10 ft**
Equation	**H1-1b**			Span	**1**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**20 ft**	**20 ft**
Pnc/om	**360.173 k**	KL/r	**79.516**	**45.432**
Pnt/om	**571.856 k**			
Mny/om	**106.985 k-ft**	L Comp Flange	**20 ft**	
Mnz/om	**237.022 k-ft**	L-torque	**20 ft**	
Vny/om	**94.38 k**	Tau_b	**1**	
Vnz/om	**260.838 k**			
Cb	**1.305**			

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X-BRACING: L4x3x1/4

PER RISA, MAX. DR = 0.96 < 1.00 OK ✓

CONNECTION DESIGN

MAX. TENSION = 36 KIPS

MAX. TENSION (SEISMIC) = 36 KIPS x 2.0 (OVERSTRENGTH) = 72 KIPS

BRACE YIELD, $T_n = \frac{R_y \cdot F_y \cdot A_g}{\Omega} = \frac{1.5 \cdot 36\text{ ksi} \cdot 1.69\text{ in}^2}{1.5} = 60.8$ KIPS ← CONTROLS

TRY (4) 7/8"Ø BOLTS

$V_n/\Omega = 16.2\,^{k}/\text{BOLT} \times 4 = 64.8^{\text{KIPS}}$

$R_n/\Omega = \frac{1.5\,(1.5'' - 15/16''/2)\cdot 0.25'' \cdot 58^{\text{ksi}}}{2.0} + \frac{3 \cdot 1.5 \cdot (3'' - 15/16'') \cdot 0.25'' \cdot 58^{\text{ksi}}}{2.0}$

$= 78.5^{k}$

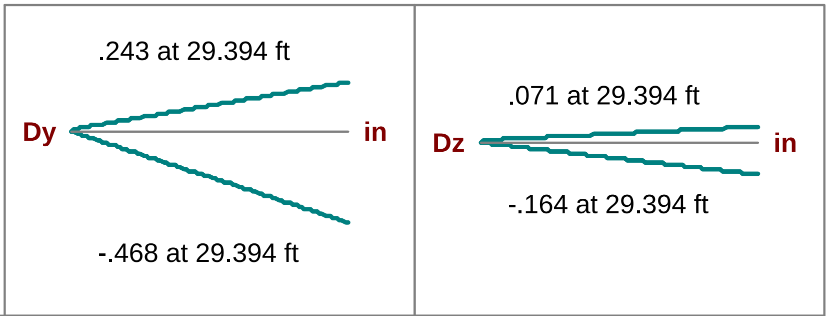
$\boxed{\text{USE (4) 7/8"Ø BOLTS}}$

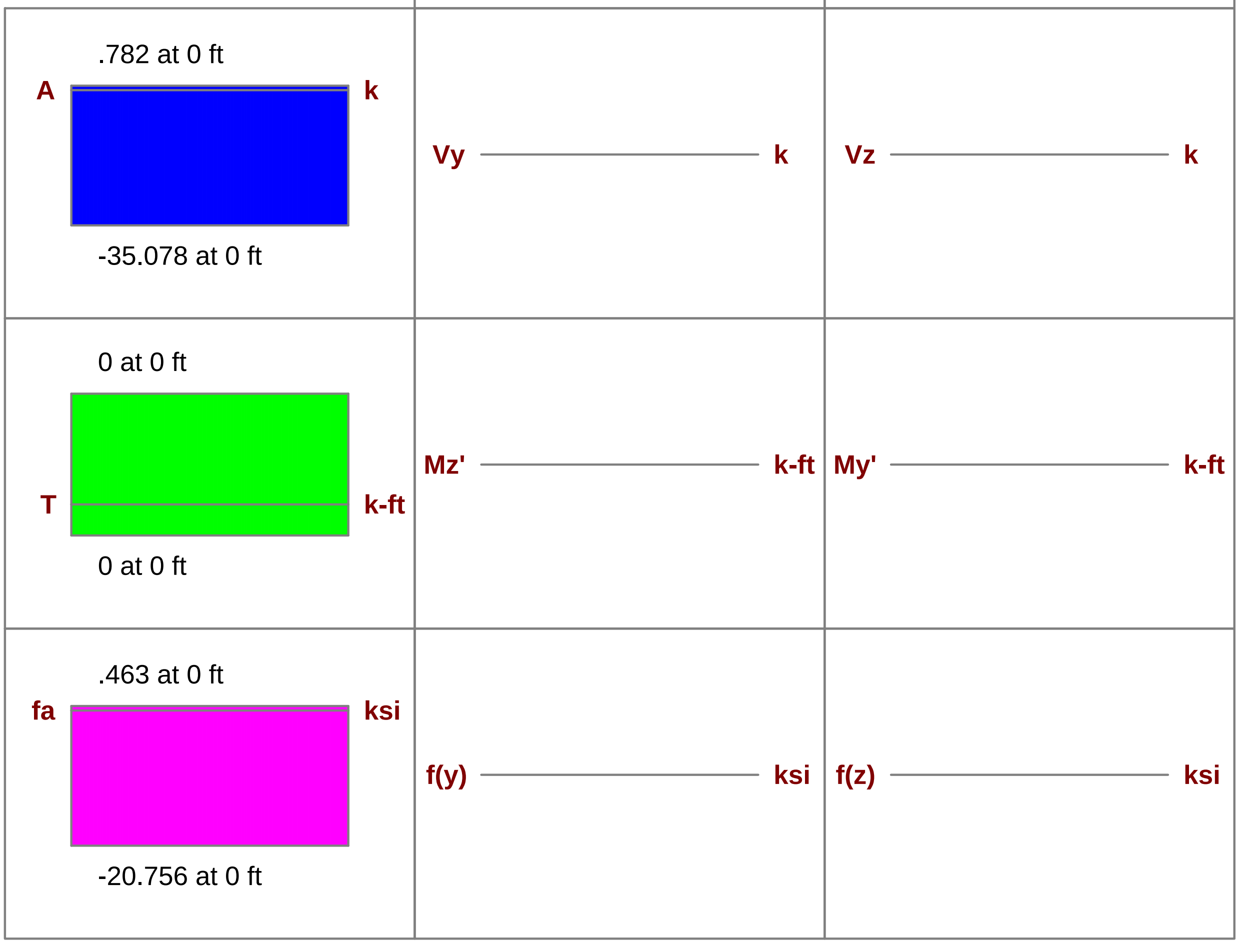
SHEET B99

TYP. X-BRACE

VBrace: *M1094A*

Shape: **L4x3x4**
Material: **A36 WTLS**
Length: **29.394 ft**
I Joint: **N66_1**
J Joint: **N608A**
Envelope
Code Check: **0.963 (LC 38)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check
Direct Analysis Method

Max Bending Check	**0.963 (LC 38)**	Max Shear Check	**0.001 (y) (LC 20)**
Location	**0 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/10000**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=.912**
Bending Web	**Non-Compact**	Compression Web	**Slender**	**Qa=1**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**14 ft**	**14 ft**
Pnc/om	**3.675 k**	KL/r	**262.911**	**118.62**
Pnt/om	**36.431 k**			
Mny'/om	**1.227 k-ft**	L Comp Flange	**14 ft**	
Mnz'/om	**1.979 k-ft**	L-torque	**29.394 ft**	
Vny/om	**12.934 k**	Tau_b	**1**	
Vnz/om	**9.701 k**			
Cb	**1**			

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BUILDING TIE STRUT: W12x40

PER RISA, MAX. DR = 0.07 < 1.0 OK ✓

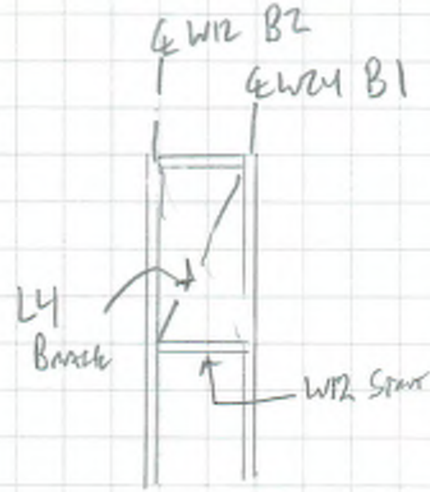
CONNECTION DESIGN

MAX RXN

TENSION = 51 KIPS
COMPRESSION = 22 KIPS

SEISMIC RXN (w/ 2.0 OVERSTRENGTH, (CONTROLS)

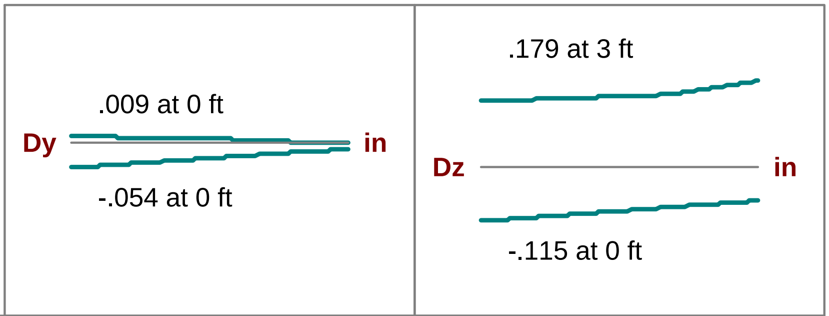
TENSION = 51 KIPS x 2.0 = 102 KIPS
COMPRESSION = 16 KIPS x 2.0 = 32 KIPS

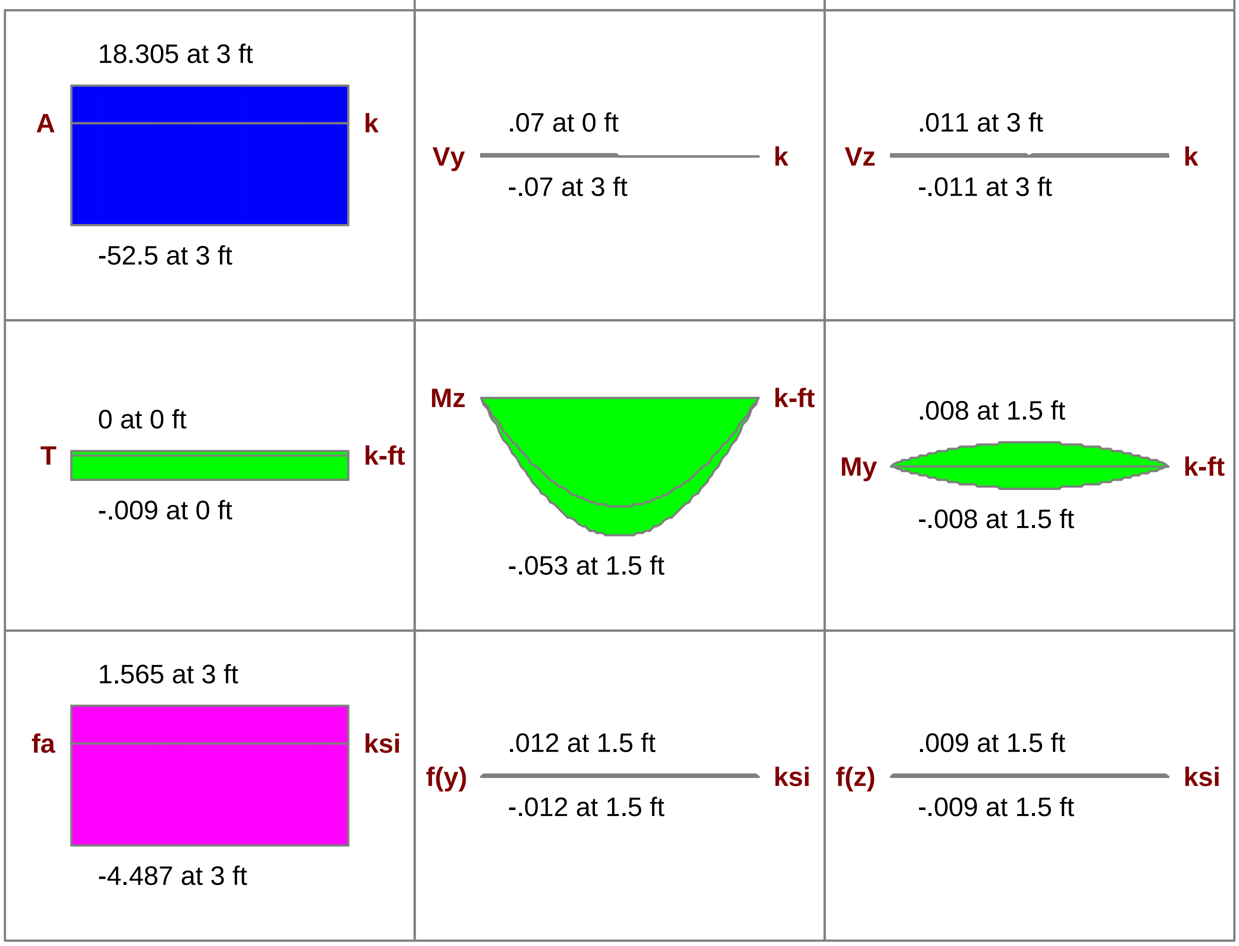


* USE FULLY WELDED CXN

Beam: *M806*

Shape: **W12x40**
Material: **A572 Gr.50**
Length: **3 ft**
I Joint: **N545A**
J Joint: **N16_1**
Envelope
Code Check: **0.075 (LC 38)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.075 (LC 38)**	Max Shear Check	**0.004 (y) (LC 27)**	Max Defl Ratio	**L/10000**
Location	**1.531 ft**	Location	**3 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**0**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=1**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=.996**

			y-y	z-z
Fy	**50 ksi**	Lb	**3 ft**	**3 ft**
Pnc/om	**340.216 k**	KL/r	**18.543**	**7.028**
Pnt/om	**350.299 k**			
Mny/om	**41.916 k-ft**	L Comp Flange	**3 ft**	
Mnz/om	**142.216 k-ft**	L-torque	**3 ft**	
Vny/om	**70.21 k**	Tau_b	**1**	
Vnz/om	**148.209 k**			
Cb	**1.136**			

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BUILDING TIE BRACE: L4x3x1/4

PER RISA, MAX. DR = 0.39 < 1.0 OK ✓

CONNECTION DESIGN

MAX TENSION = 15 KIPS

MAX TENSION (SEISMIC) = 13 KIPS x 2.0 = 26 KIPS

$\boxed{\text{USE (2) 7/8" Ø BOLTS}}$

$$R_n = \frac{1.5 \cdot (1.5'' - 15/16''/2) \cdot 0.25'' \cdot 58^{ksi} + 1.5 \cdot (3'' - 15/16'') \cdot 0.25'' \cdot 58^{ksi}}{2.0}$$

$= 33.6^{KIPS}$

TRY 3/16" x 10" WELD
ASD CAPACITY = 2.78 k/in
Tallow = 10" x 2.78 k/in = 27.8 k/in

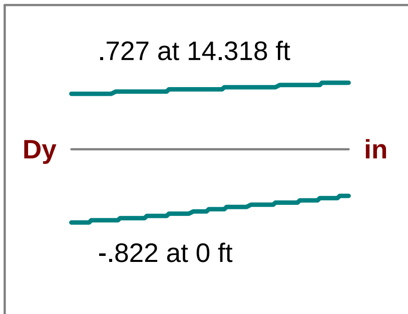
VBrace: ***M1132C***

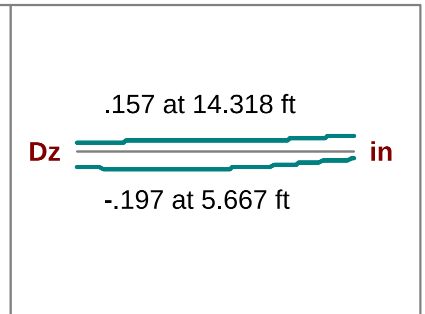
Shape:	**L4x3x4**
Material:	**A36 Gr.36**
Length:	**14.318 ft**
I Joint:	**N11**
J Joint:	**N878C**

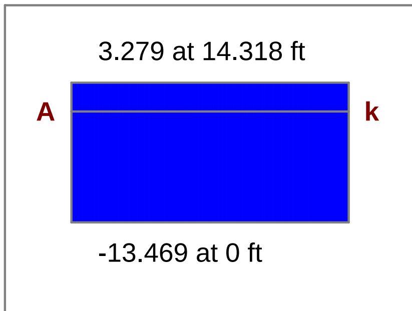
Envelope

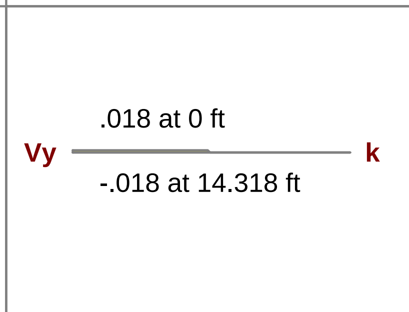
Code Check: **0.370 (LC 35)**

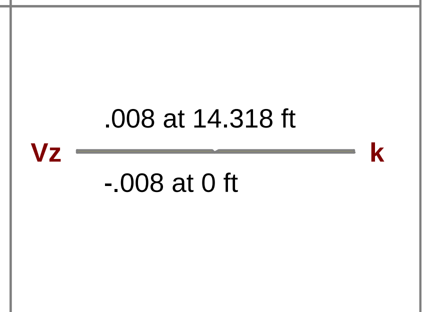
Report Based On 97 Sections

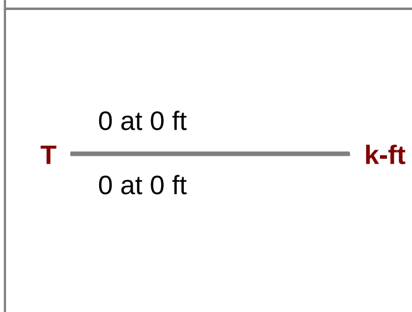


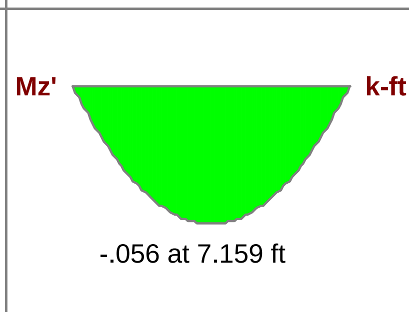


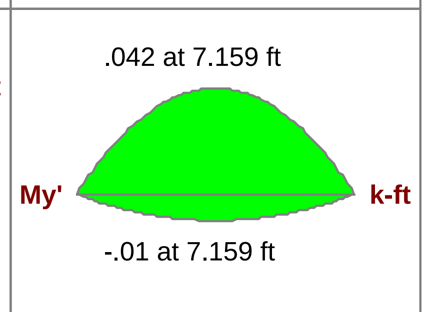


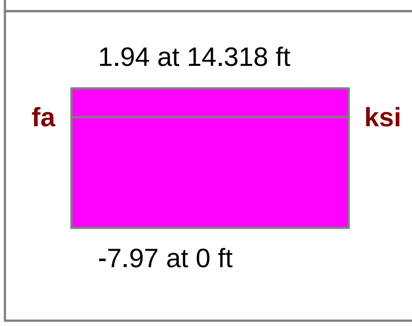


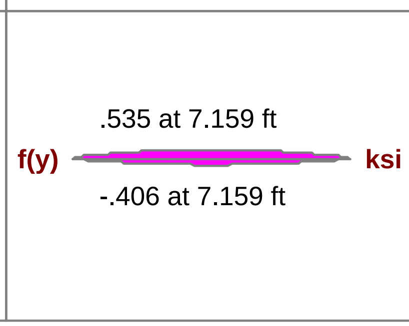


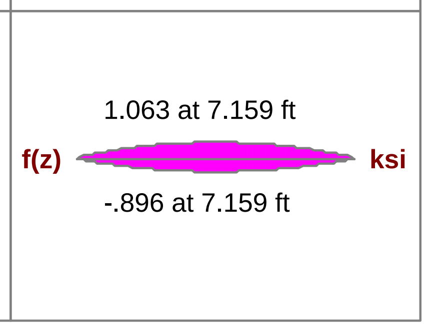












AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.370 (LC 35)**	Max Shear Check	**0.002 (y) (LC 21)**
Location	**0 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/10000**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=.912**
Bending Web	**Non-Compact**	Compression Web	**Slender**	**Qa=1**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**14.318 ft**	**14.318 ft**
Pnc/om	**3.514 k**	KL/r	**268.879**	**121.313**
Pnt/om	**36.431 k**			
Mny'/om	**1.227 k-ft**	L Comp Flange	**14.318 ft**	
Mnz'/om	**2.091 k-ft**	L-torque	**14.318 ft**	
Vny/om	**12.934 k**	Tau_b	**1**	
Vnz/om	**9.701 k**			
Cb	**1.136**			

TRUSS CHORD: W8x31

PER RISA, MAX. D_R = 0.93 < 1.0 OK✓

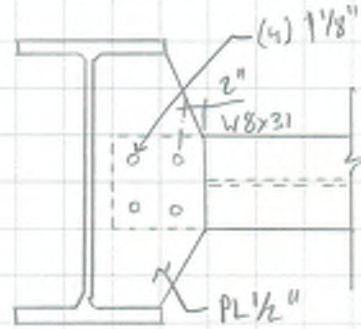
CONNECTION DESIGN

MAX. RXNS

TENSION = 112 KIPS
COMPRESSION = 105 KIPS

SEISMIC RXNS (2.0 OVERSTRENGTH FACTOR)

TENSION = 94 KIPS
COMPRESSION = 95 KIPS x 2.0 = 190 KIPS



TRY FLANGE STIFFENER (1/2") W/(8) 1 1/8"Ø BOLTS

1 1/8"Ø BOLT, V_n/Ω = 26.8 k/BOLT

W8x31, t_f = 0.435"

R_n/Ω = 54.8 k/in x 0.435" = 23.8 k/BOLT ← CONTROLS
48.9 k/in x 0.5" = 24.4 k/BOLT

(8) 1 1/8"Ø BOLTS → 23.8 k/BOLT x 8 = 190.4 KIPS OK✓

STIFFENER WELD
$l_{weld} \approx 4\cdot(20" + 2.5") = 120"$

WELD DEMAND → R_u = 190k/120" = 1.6 k/in → 5/16" FILLET OK By INSP.

$\boxed{\text{USE 1/2" STIFFENERS w/(8) 1 1/8"Ø BOLTS}}$

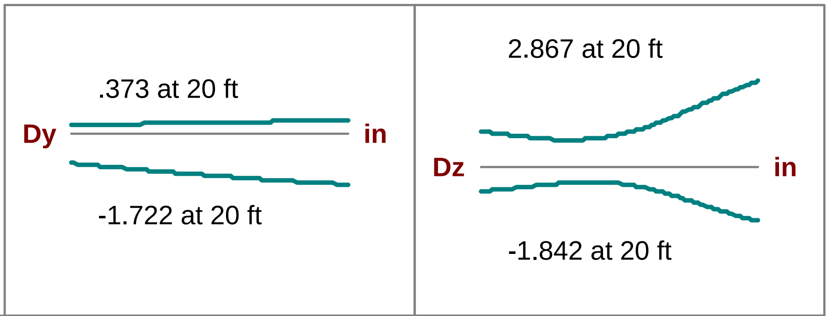
Beam: ***M328A_1***

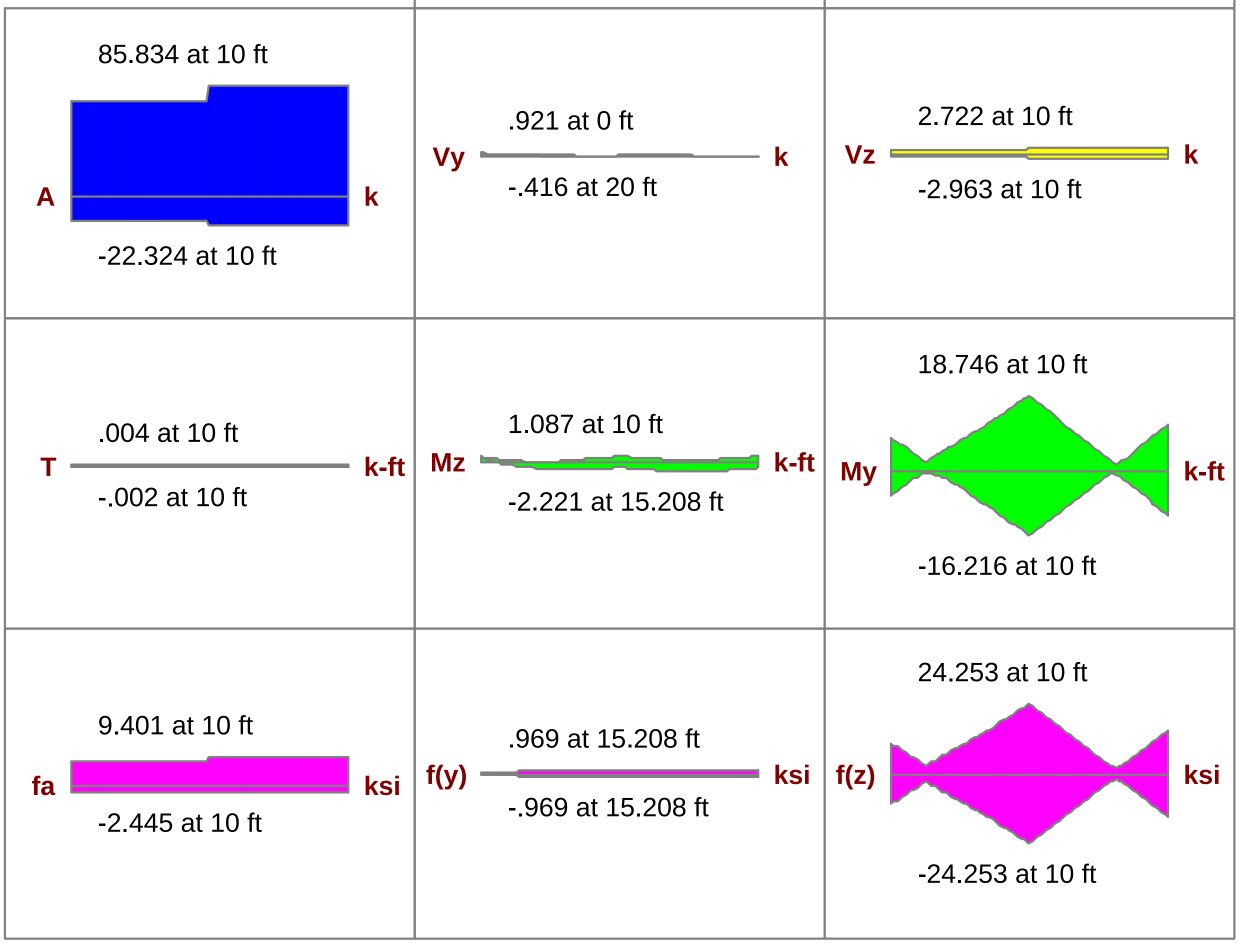
Shape:	**W8x31**
Material:	**A572 Gr.50**
Length:	**20 ft**
I Joint:	**N15_1**
J Joint:	**N55_1**

Envelope

Code Check: **0.925 (LC 33)**

Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.925 (LC 33)**	Max Shear Check	**0.025 (z) (LC 17)**	Max Defl Ratio	**L/6854**
Location	**10 ft**	Location	**10 ft**	Location	**15 ft**
Equation	**H1-1a**			Span	**2**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**10 ft**	**10 ft**
Pnc/om	**191.199 k**	KL/r	**59.529**	**34.572**
Pnt/om	**273.353 k**			
Mny/om	**35.124 k-ft**	L Comp Flange	**10 ft**	
Mnz/om	**71.309 k-ft**	L-torque	**20 ft**	
Vny/om	**45.6 k**	Tau_b	**1**	
Vnz/om	**125.03 k**			
Cb	**1**			

TRUSS DIAGONAL (SMALL) : LL3x2x1/4

PER RISA, MAX. DR = 0.86 < 1.0 OK✓

CONNECTION DESIGN

MAX. TENSION = 33 KIPS

MAX. TENSION (SEISMIC) = 30 KIPS x 2.0 (OVERSTRENGTH) = 60 KIPS ← CONTROLS

BRACE YIELD, $T_n = \frac{R_y \cdot F_y \cdot A_g}{\alpha} = \frac{1.5 \cdot 36\text{ ksi} \cdot 1.20\text{ in}^2 \cdot 2}{1.5} = 86.4^k$

TRY (3) 3/4"Ø BOLTS

$V_n/\Omega = 23.9^k$/BOLT → CONTROLS, x3 = 71.7 KIPS

$R_r/\Omega = 52.2^k/\text{in} \times 0.5" = 26.1^k$/BOLT

$R_n/\Omega = 52.2^k/\text{in}$ OK✓

$\boxed{\text{USE 1/2" PL w/(3) 3/4"Ø BOLTS}}$

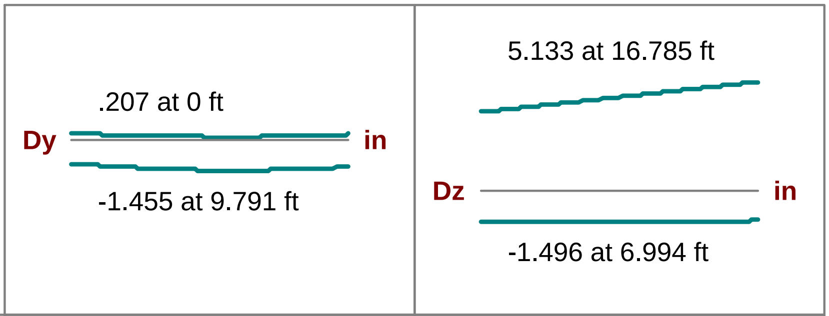
3/16" WELD

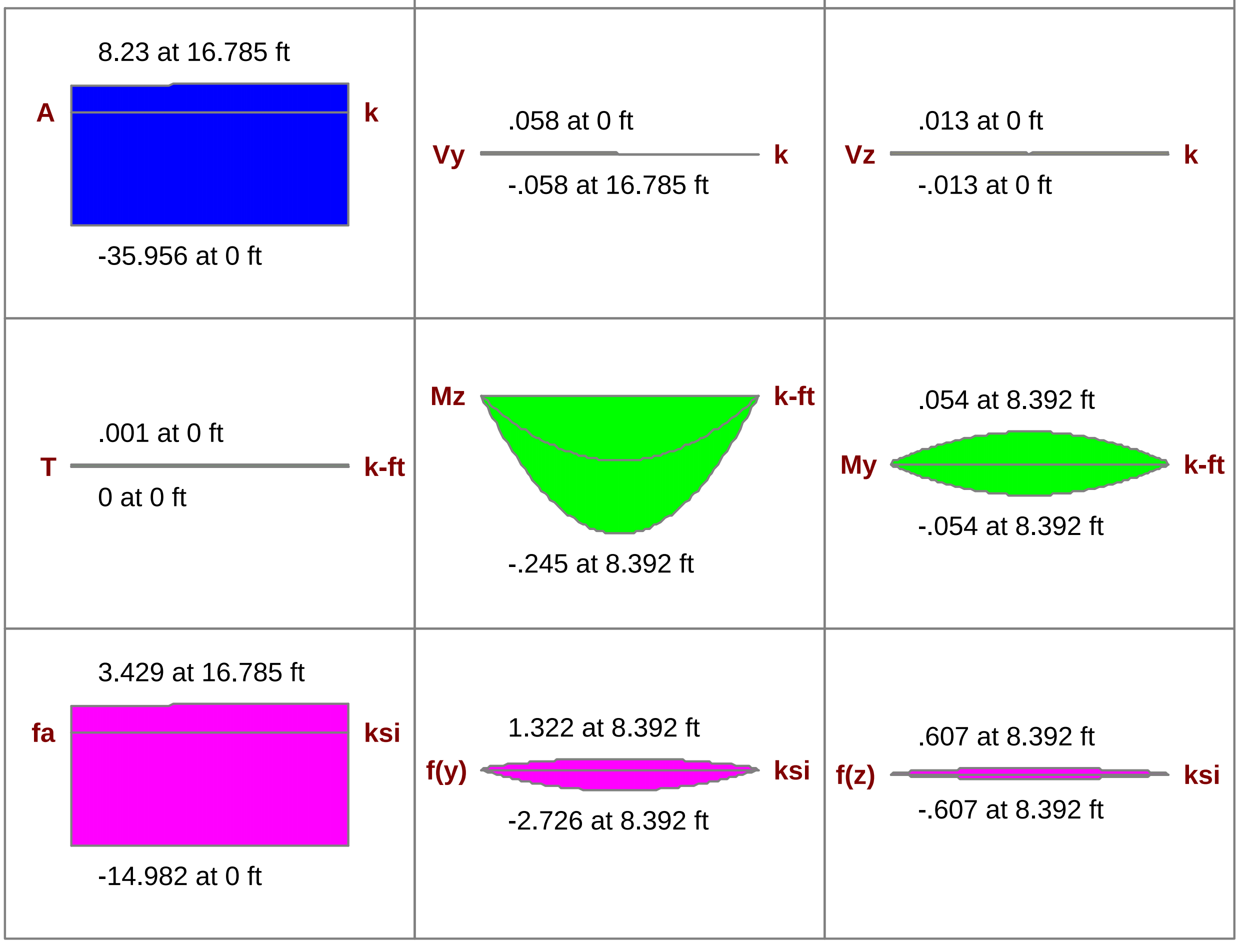
$R_n/\Omega = 2.78^k/\text{in}$ $L_r = 60^k / 2.78^k/\text{in} = 21.6"$ → 11"/ANGLE

$\boxed{\text{USE 3/16" FILLET w/ 11" MIN LENGTH PER ANGLE}}$

VBrace: ***M1041***

Shape: **LL3x2x4x6**
Material: **A36 Gr.36**
Length: **16.785 ft**
I Joint: **N804A**
J Joint: **N794A**
Envelope
Code Check: **0.860 (LC 24)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check
Direct Analysis Method

Max Bending Check	**0.860 (LC 24)**	Max Shear Check	**0.007 (y) (LC 44)**
Location	**8.917 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/819**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**36 ksi**	Lb	**15 ft**	**15 ft**
Pnc/om	**10.113 k**	KL/r	**174.626**	**188.864**
Pnt/om	**51.737 k**			
Mny/om	**3.086 k-ft**	L Comp Flange	**15 ft**	
Mnz/om	**3.102 k-ft**	L-torque	**16.785 ft**	
Vny/om	**19.401 k**	Tau_b	**1**	
Vnz/om	**12.934 k**			
Cb	**1**			

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TRUSS DIAGONAL (MEDIUM-SMALL): LL4x3x1/4

PER RISA, MAX. DR = 0.85 < 1.0 OK✓

CONNECTION DESIGN

MAX. TENSION = 58 KIPS

MAX. TENSION (SEISMIC) = 50 KIPS x 2.0 (OVERSTRENGTH) = 100 KIPS

TRY 3/16" FILLET

$R_n/\Omega = 2.78\ ^k/in \rightarrow L_r = 100^k/2.78 = 36"$

USE 18" FILLET WELD PER ANGLE

TRY 1/2" PL GUSSET - BLOCK SHEAR

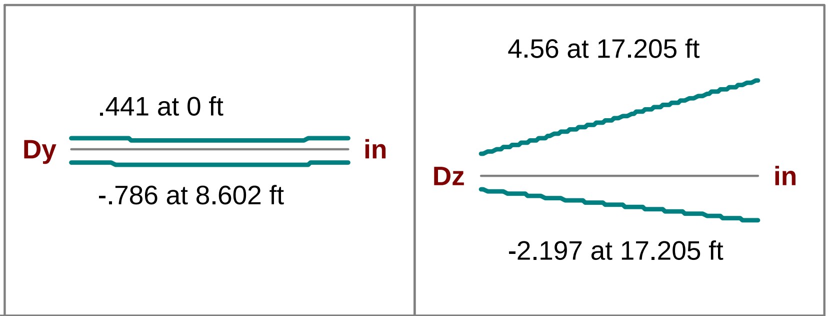
$$\frac{R_n}{\Omega} = \frac{36\text{ ksi} \cdot 0.6 \cdot (7" \cdot 0.75" \cdot 2) + 58\text{ ksi} \cdot 0.75" \cdot 4"}{2.0}$$

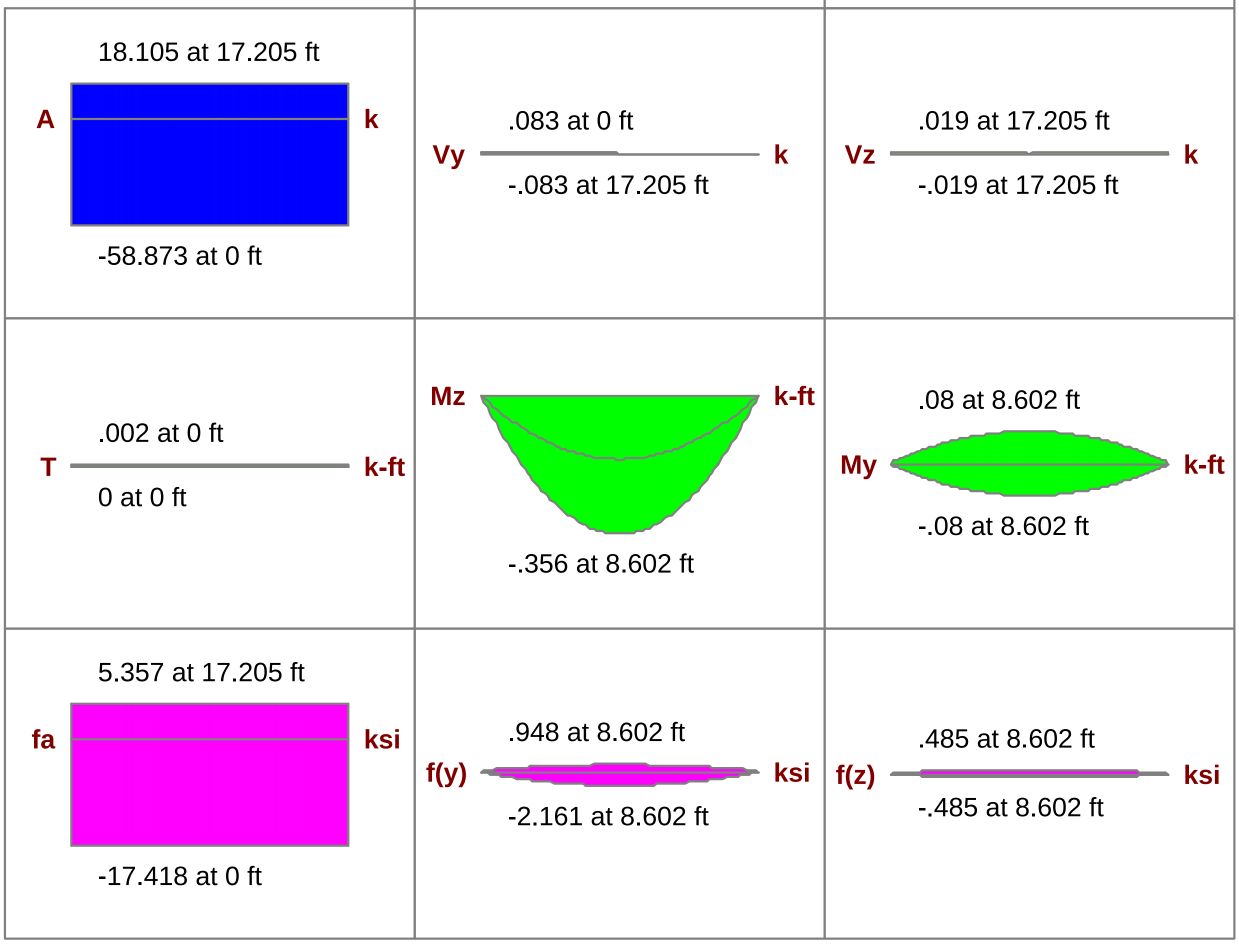
= 200 KIPS OK✓

USE 1/2" GUSSET

VBrace: ***M781A***

Shape: **LL4x3x4x6**
Material: **A36 Gr.36**
Length: **17.205 ft**
I Joint: **N43**
J Joint: **N564**
Envelope
Code Check: **0.847 (LC 33)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.847 (LC 33)**	Max Shear Check	**0.007 (y) (LC 33)**
Location	**8.423 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/1387**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=.912**
Bending Web	**Non-Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**36 ksi**	Lb	**16 ft**	**16 ft**
Pnc/om	**22.425 k**	KL/r	**136.168**	**150.514**
Pnt/om	**72.862 k**			
Mny/om	**5.723 k-ft**	L Comp Flange	**16 ft**	
Mnz/om	**5.686 k-ft**	L-torque	**17.205 ft**	
Vny/om	**25.868 k**	Tau_b	**1**	
Vnz/om	**19.401 k**			
Cb	**1**			

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JOB NO. SHEET OF

TRUSS DIAGONAL (MEDIUM-LARGE): LL4x3x5/16

PER RISA, MAX. DR = 0.98 < 1.0 OK ✓

CONNECTION DESIGN

MAX. TENSION = 77 KIPS

MAX. TENSION (SEISMIC) = 67 KIPS x 2.0 (OVERSTRENGTH) = 134 KIPS

TRY 1/4" FILLET WELD

$R_n/\Omega = 3.71\ ^k/_{IN} \rightarrow L_r = 134^k / 3.71\ ^k/_{IN} = 37''$

$\boxed{\text{USE } 19'' \times 1/4'' \text{ FILLET WELD PER ANGLE}}$

3/4" PL BLOCK SHEAR

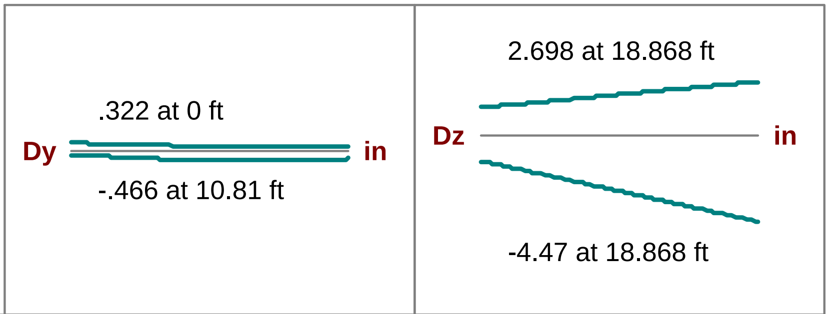
$0.6 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$

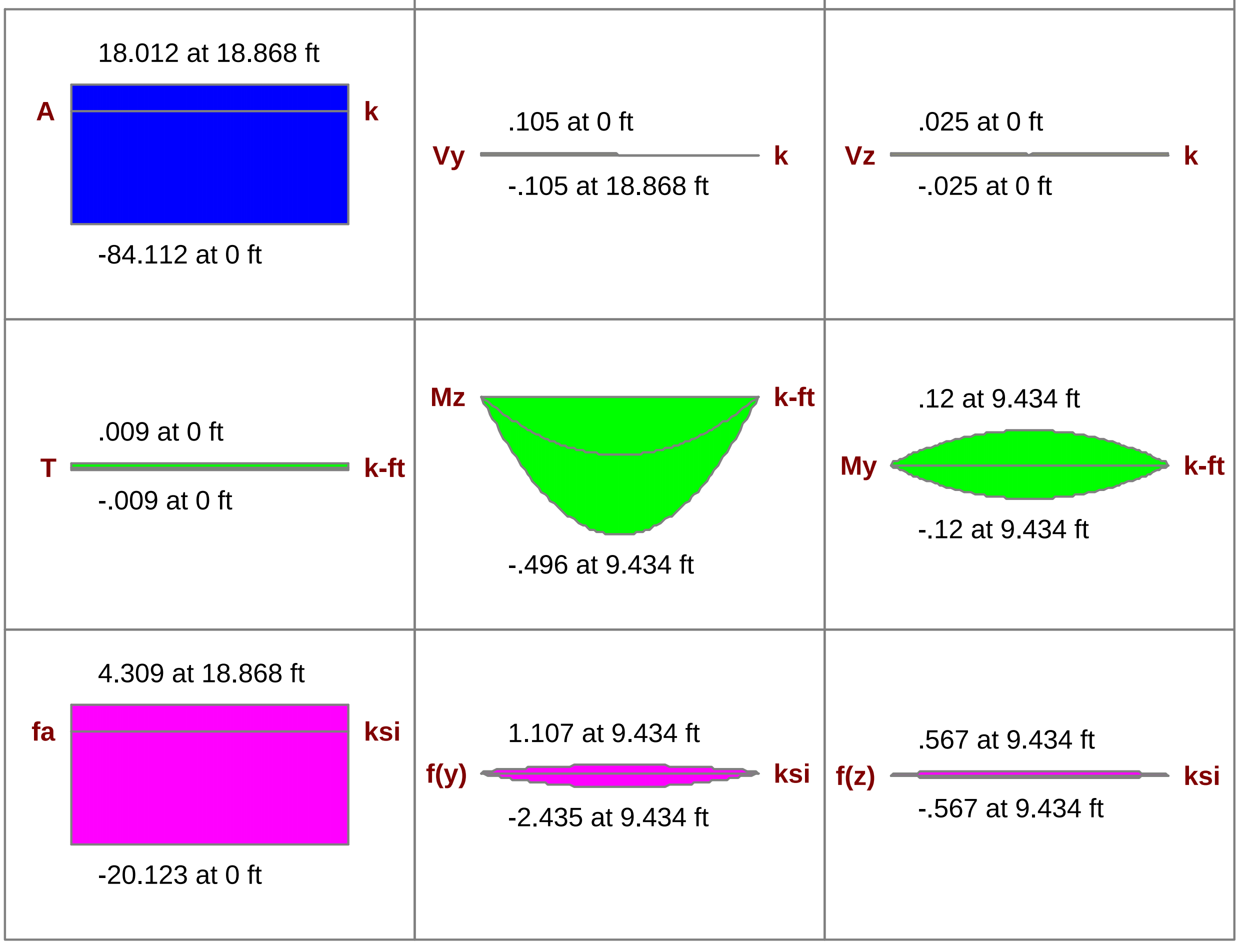
$$\frac{R_n}{\Omega} = \frac{0.6 \cdot 36\,\text{ksi} \cdot (0.75'' \cdot 7.5'' \cdot 2) + 58 \cdot \text{ksi} \cdot (4'' \cdot 0.75'')}{2.0}$$

$= 208.5$ KIPS OK ✓

VBrace: ***M1078A***

Shape: **LL4x3x5x6**
Material: **A36 Gr.36**
Length: **18.868 ft**
I Joint: **N158_1**
J Joint: **N1142**
Envelope
Code Check: **0.977 (LC 33)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.977 (LC 33)**	Max Shear Check	**0.017 (y) (LC 27)**
Location	**9.237 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/1110**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=.997**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y	z-z
Fy	**36 ksi**	Lb	**17 ft**	**17 ft**
Pnc/om	**24.271 k**	KL/r	**142.638**	**160.892**
Pnt/om	**90.108 k**			
Mny/om	**7.281 k-ft**	L Comp Flange	**17 ft**	
Mnz/om	**7.024 k-ft**	L-torque	**18.868 ft**	
Vny/om	**32.335 k**	Tau_b	**1**	
Vnz/om	**24.251 k**			
Cb	**1**			

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JOB NO. SHEET OF

TRUSS DIAGONAL (LARGE): LL4x3½x3/8

PER RISA, MAX. DR = 0.95 < 1.0 OK ✓

CONNECTION DESIGN

MAX. TENSION = 107 KIPS

MAX. TENSION (SEISMIC) = 93 KIPS x 2.0 (OVERSTRENGTH) = 186 KIPS

TRY 5/16" FILLET

$R_n/\Omega = 4.64$ K/IN → $L_r = 186^k / 4.64^{k/in} = 41"$

$\boxed{\text{USE 5/16" x 22" WELD @ EA. ANGLE}}$

TRY 3/4" PL, BLOCK SHEAR

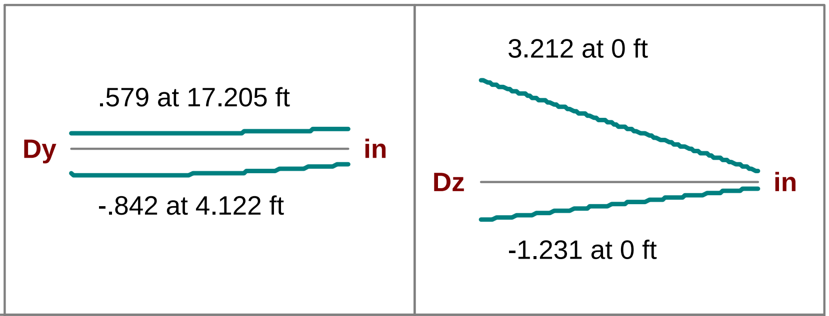
$$R_n/\Omega = \frac{36\text{ ksi}\cdot 0.6(0.75"\cdot 9"\cdot 2) + 58\text{ ksi}\cdot(4"\cdot 0.75")}{2.0}$$

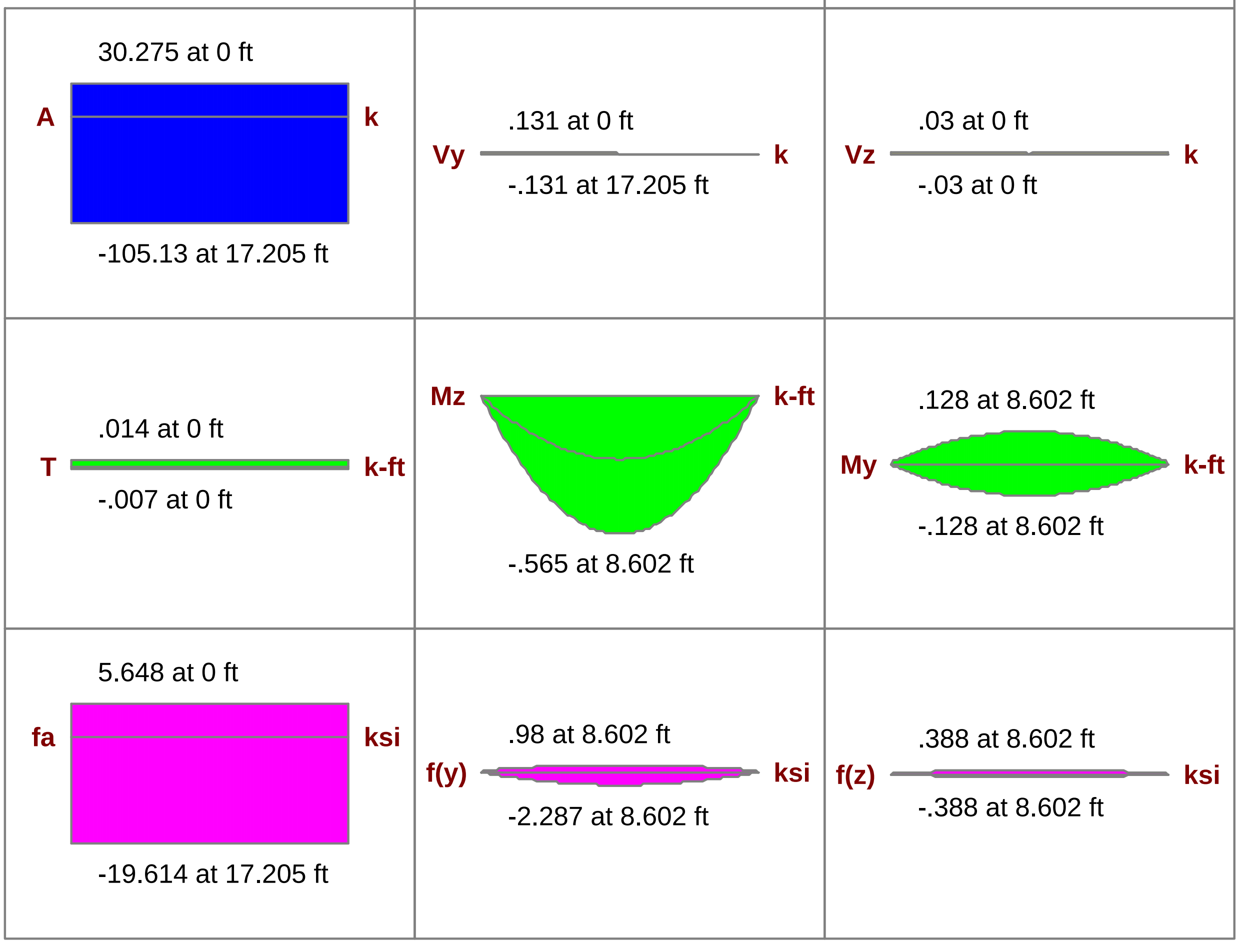
$= 233^{\text{KIPS}}$ OK ✓

$\boxed{\text{USE 3/4" GUSSET}}$

VBrace: *M908A*

Shape: **LL4x3.5x6x6**
Material: **A36 Gr.36**
Length: **17.205 ft**
I Joint: **N24**
J Joint: **N91**
Envelope
Code Check: **0.952 (LC 33)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.952 (LC 33)**	Max Shear Check	**0.018 (y) (LC 27)**
Location	**8.782 ft**	Location	**0 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/1320**
Bending Flange	**Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**36 ksi**	Lb	**16 ft**	**16 ft**
Pnc/om	**33.842 k**	KL/r	**113.642**	**154.292**
Pnt/om	**115.545 k**			
Mny/om	**11.349 k-ft**	L Comp Flange	**16 ft**	
Mnz/om	**8.52 k-ft**	L-torque	**17.205 ft**	
Vny/om	**38.802 k**	Tau_b	**1**	
Vnz/om	**33.952 k**			
Cb	**1**			

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TRUSS STRUT : W8x31

PER RISA, MAX DR = 0.61 < 1.0 OK ✓

CONNECTION DESIGN (ATTACH DIAG. BRACE TO STRUT, SIZE CXN FOR MAX BRACE)

CONFIG #1 - (2) SMALL BRACES, $T_{SEISMIC} = 60^{KIPS} \times 2 = 120^{KIPS}$

CONFIG #2 - (1) MEDIUM-LARGE BRACE, $T_{SEISMIC} = 134^{KIPS}$ ← OPTION #1

CONFIG #3 - (1) LARGE BRACE, $T_{SEISMIC} = 186^{KIPS}$ ← OPTION #2

C OPTION #1, $T_{MAX} = 134^{KIPS}$

$\boxed{\text{USE 1/2" END PL w/ (4) 1"}\varnothing\text{ BOLTS, } T_A/\Omega = 4 \cdot 35.3^k = 141^{KIPS}}$

$L_{WELD} = 134^k / 3.71^k/in = 36"$, W8x31 PERIMETER = 46" OK ✓

USE 1/4" FILLET ALL AROUND

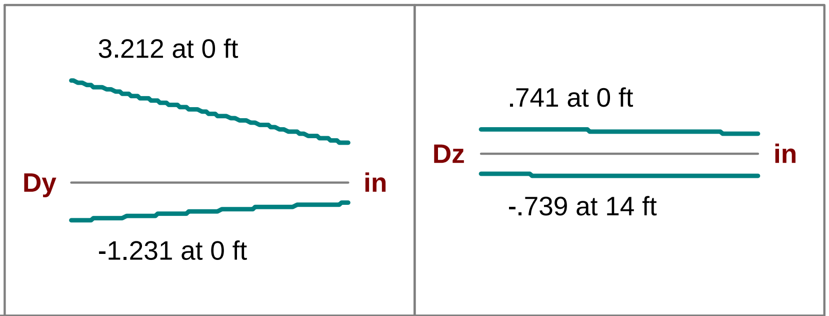
C OPTION #2, $T_{MAX} = 186^{KIPS}$

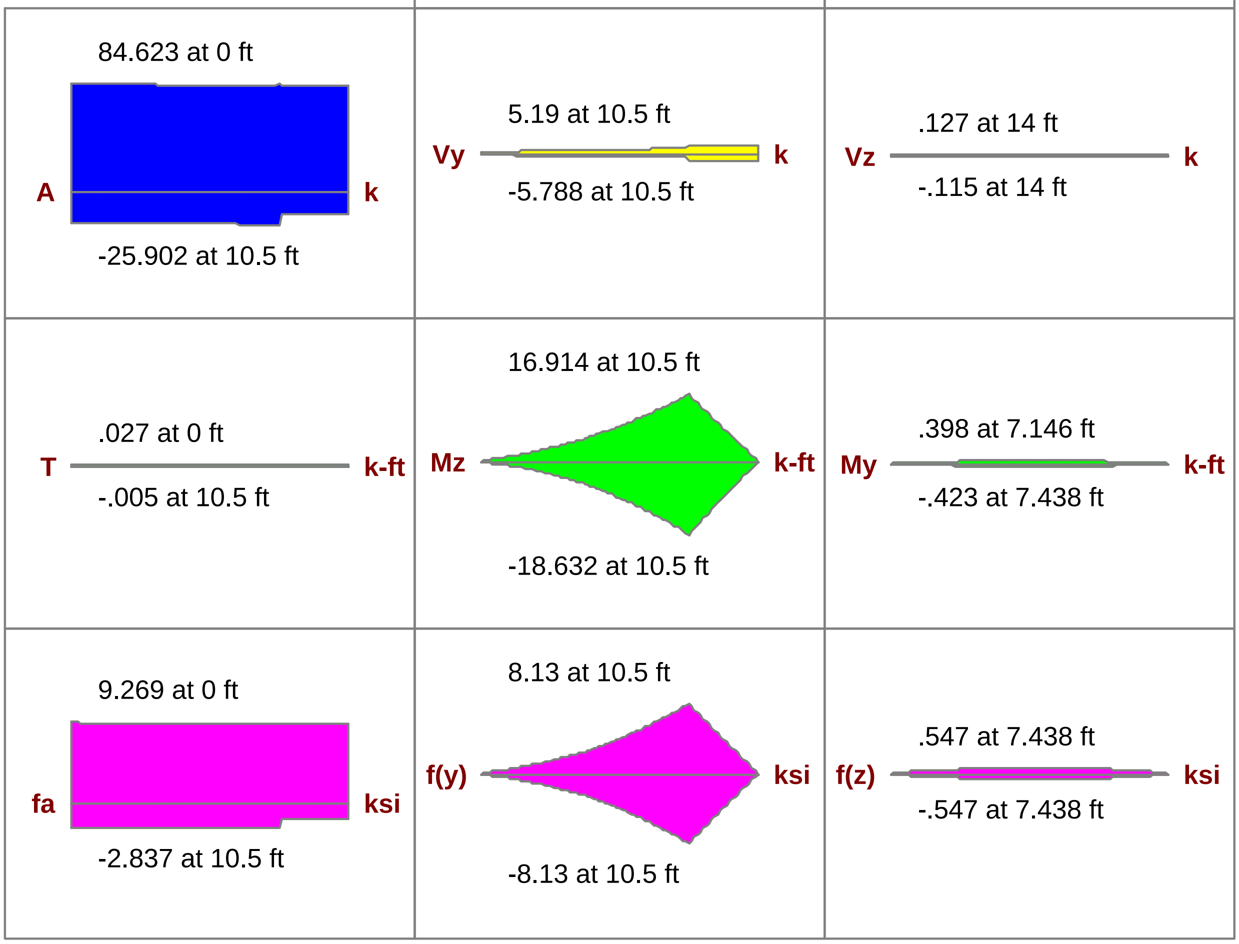
$\boxed{\text{USE 1/2" END PL w/ (4) 1 1/4"}\varnothing\text{ BOLTS, } T_A/\Omega = 4 \cdot 55.2^k = 221^k}$ OK ✓

$L_{WELD} = 184^k / 3.71^k/in = 50"$ → USE 5/16" WELD ALL AROUND

Column: ***M916A***

Shape: **W8x31**
Material: **A572 Gr.50**
Length: **14 ft**
I Joint: **N24**
J Joint: **N14**
Envelope
Code Check: **0.616 (LC 29)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.616 (LC 29)**	Max Shear Check	**0.129 (y) (LC 24)**
Location	**10.5 ft**	Location	**10.5 ft**
Equation	**H1-1a**	Max Defl Ratio	**L/78**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**14 ft**	**14 ft**
Pnc/om	**164.501 k**	KL/r	**83.341**	**48.4**
Pnt/om	**273.353 k**			
Mny/om	**35.124 k-ft**	L Comp Flange	**14 ft**	
Mnz/om	**75.767 k-ft**	L-torque	**14 ft**	
Vny/om	**45.6 k**	Tau_b	**1**	
Vnz/om	**125.03 k**			
Cb	**1.551**			

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REAR TRUSS CHORD CONNECTION

UPPER CHORD TO W24

MAX COMPRESSION = 59^K
MAX TENSION = 37^K

SEISMIC RXNS (2.0 OVERSTRENGTH FACTOR)

MAX COMPRESSION = 31^K
MAX TENSION = $33^K \times 2.0 = 66^{KIPS}$ ← CONTROLS

TRY 1/2" END PL w/ (4) 3/4"Ø BOLTS

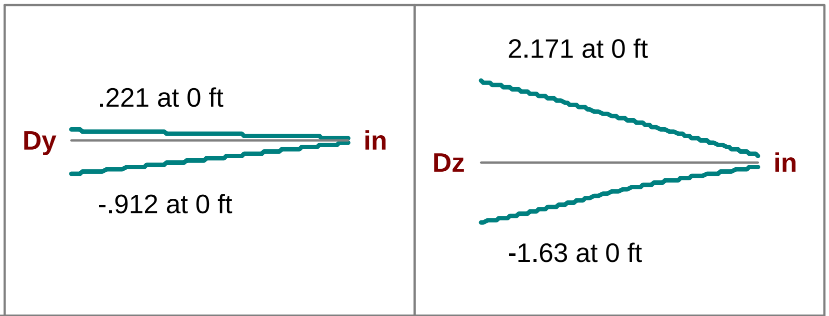
$T_n/\Omega = 4 \cdot 19.9^k/\text{BOLT} = 79.6^{KIPS}$

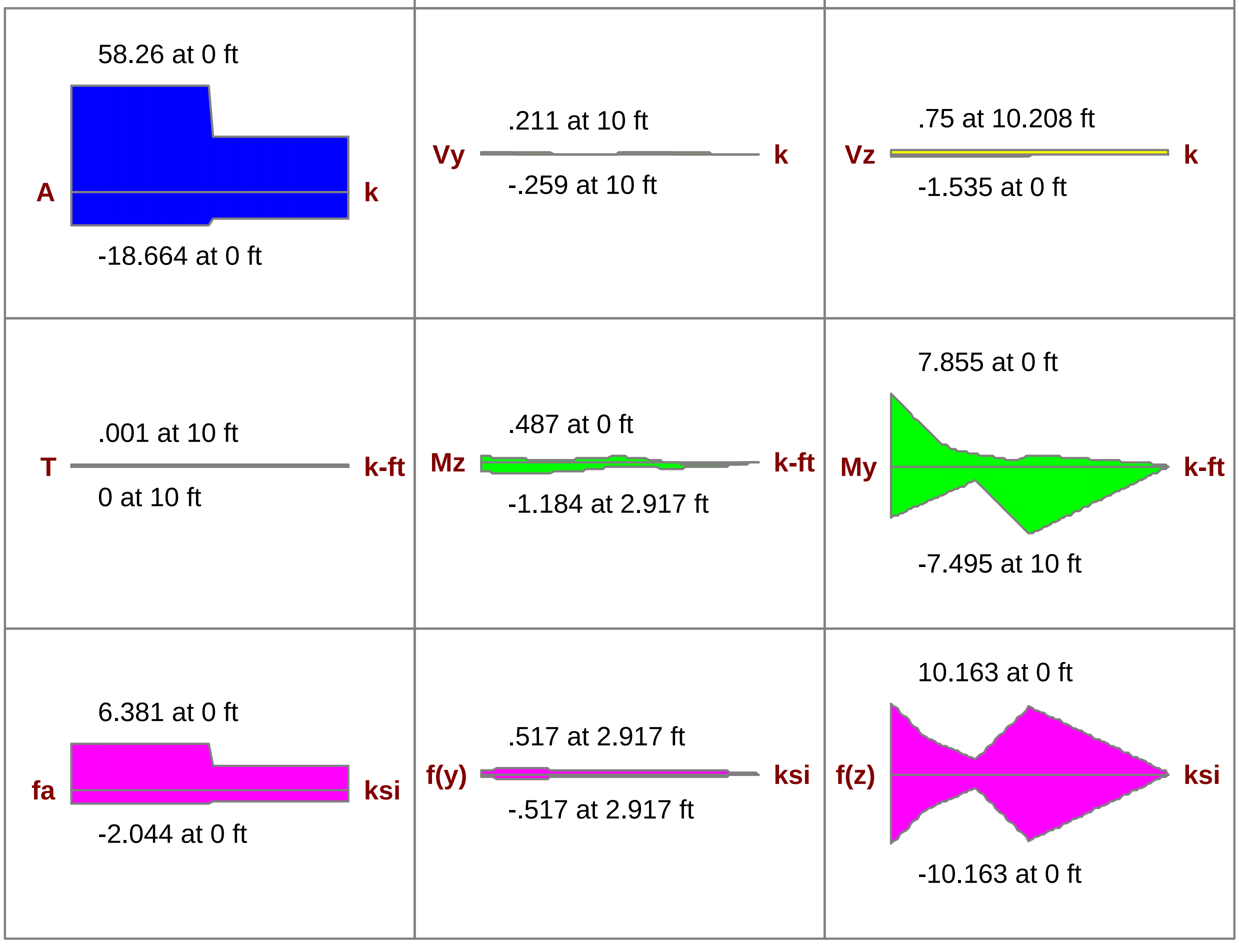
L_{WELD} (1/4" FILLET) $= 66^k / 3.71^k/\text{IN} = \underline{18"}$ ALL AROUND OR BY INSP

* ALIGN W24 STIFFENER AS REQ'D

Beam: ***M1001***

Shape: **W8x31**
Material: **A572 Gr.50**
Length: **20 ft**
I Joint: **N797A**
J Joint: **N545A**
Envelope
Code Check: **0.365 (LC 35)**
Report Based On 97 Sections





AISC 14th(360-10): ASD Code Check
Direct Analysis Method

Max Bending Check	**0.365 (LC 35)**	Max Shear Check	**0.013 (z) (LC 20)**	Max Defl Ratio	**L/10000**
Location	**0 ft**	Location	**0 ft**	Location	**0 ft**
Equation	**H1-1a**			Span	**0**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**10 ft**	**10 ft**
Pnc/om	**191.199 k**	KL/r	**59.529**	**34.572**
Pnt/om	**273.353 k**			
Mny/om	**35.124 k-ft**	L Comp Flange	**10 ft**	
Mnz/om	**71.309 k-ft**	L-torque	**20 ft**	
Vny/om	**45.6 k**	Tau_b	**1**	
Vnz/om	**125.03 k**			
Cb	**1**			

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REAR TRUSS CHORD CONNECTIONS

LOWER CHORD TO W12 COLUMN

MAX COMPRESSION = 72^k) @ WEB CONNECTION 19^k) @ FLANGE CONNECTION
MAX TENSION = 72^k 41^k

SEISMIC RXNS (2.0 OVERSTRENGTH)

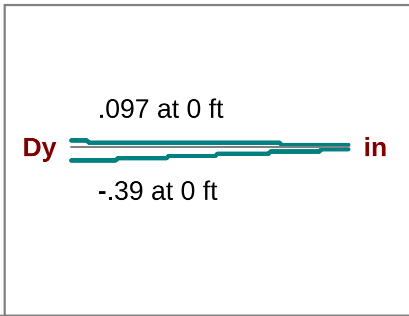
MAX COMPRESSION = 70^k) @ WEB 19^k) @ FLANGE
MAX TENSION = 6^k 19^k

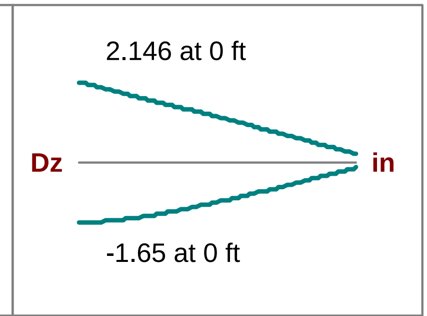
USE 1/2" END "PL" CXN w/ (4) 3/4" ∅ BOLTS

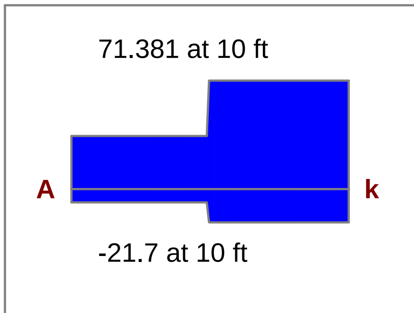
$\boxed{\text{OK BY INSPECTION}}$

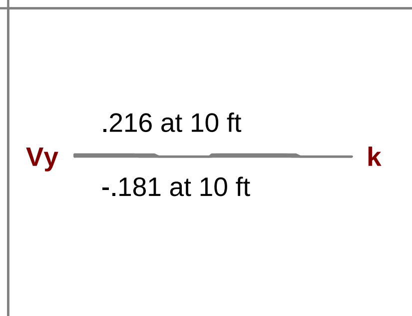
Beam: ***M978C***

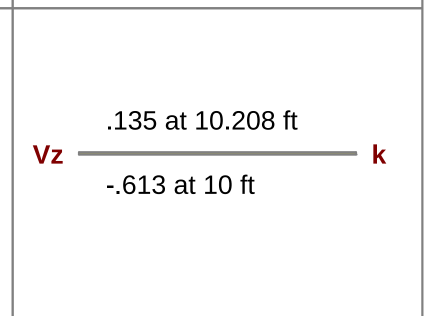
Shape: **W8x31**
Material: **A572 Gr.50**
Length: **20 ft**
I Joint: **N790B**
J Joint: **N787A**
Envelope
Code Check: **0.398 (LC 33)**
Report Based On 97 Sections

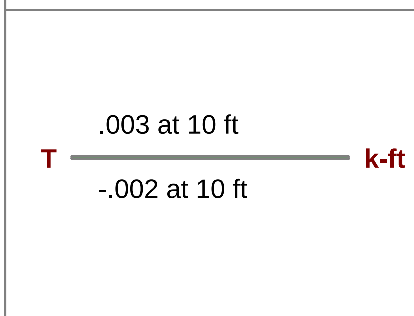


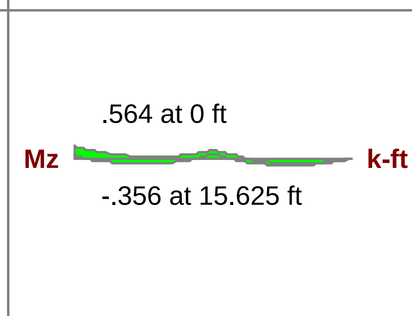


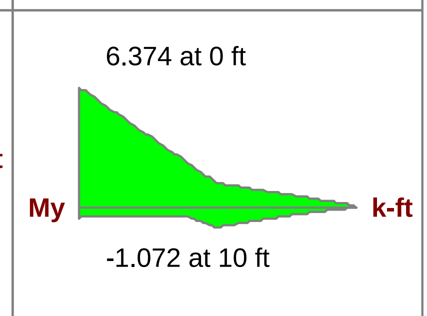


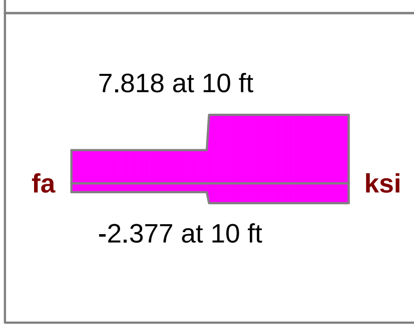


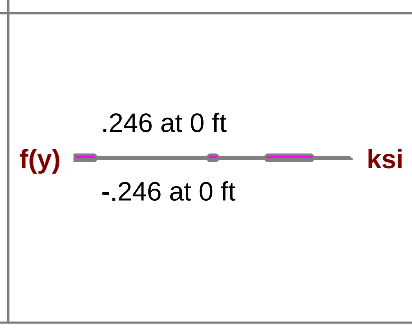


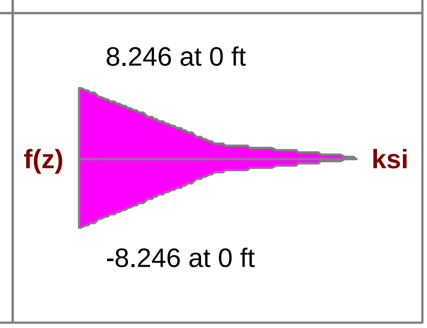












AISC 14th(360-10): ASD Code Check

Direct Analysis Method

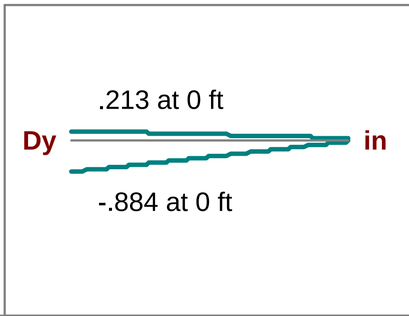
Max Bending Check	**0.398 (LC 33)**	Max Shear Check	**0.007 (z) (LC 41)**	Max Defl Ratio	**L/10000**
Location	**10 ft**	Location	**10 ft**	Location	**0 ft**
Equation	**H1-1a**			Span	**0**

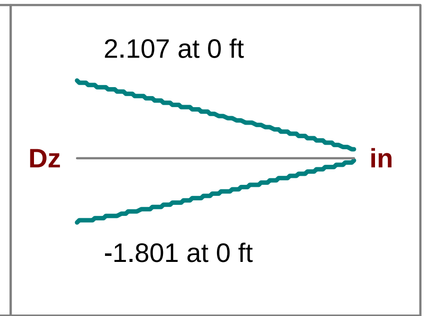
Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

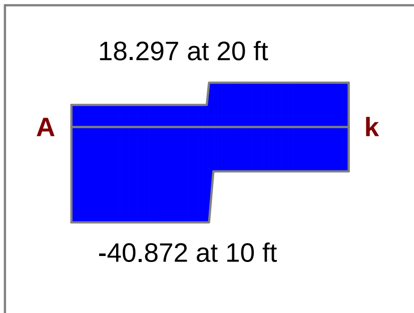
			y-y	z-z
Fy	**50 ksi**	Lb	**10 ft**	**10 ft**
Pnc/om	**191.199 k**	KL/r	**59.529**	**34.572**
Pnt/om	**273.353 k**			
Mny/om	**35.124 k-ft**	L Comp Flange	**10 ft**	
Mnz/om	**75.767 k-ft**	L-torque	**20 ft**	
Vny/om	**45.6 k**	Tau_b	**1**	
Vnz/om	**125.03 k**			
Cb	**2.01**			

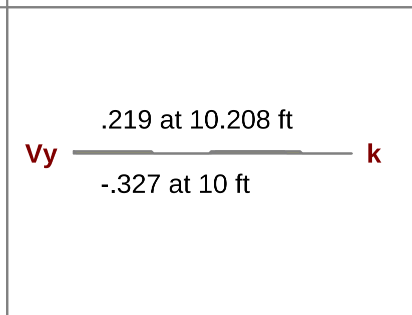
Beam: ***M983B***

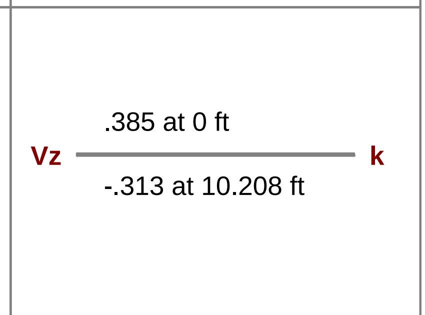
Shape: **W8x31**
Material: **A572 Gr.50**
Length: **20 ft**
I Joint: **N798A**
J Joint: **N786B**
Envelope
Code Check: **0.135 (LC 25)**
Report Based On 97 Sections

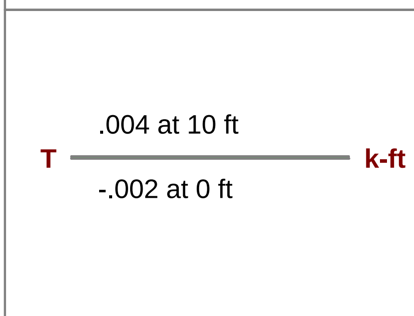


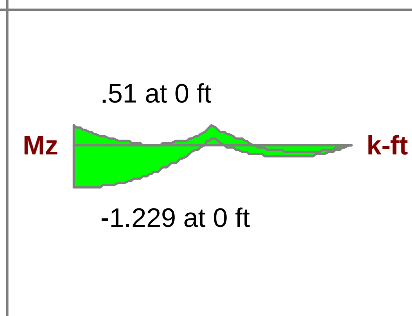


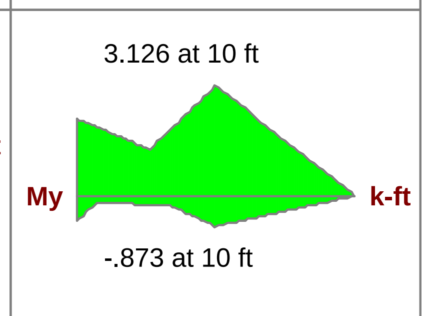


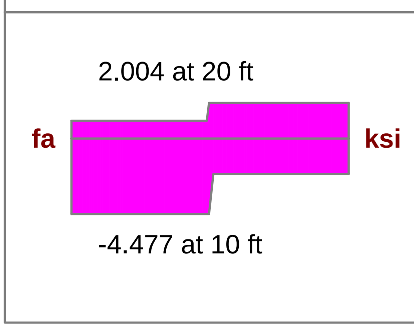


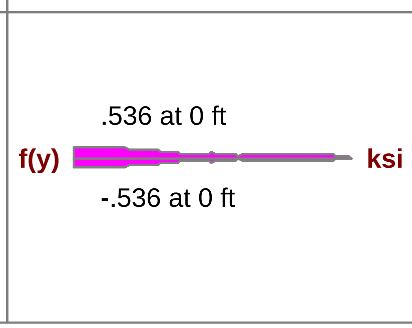


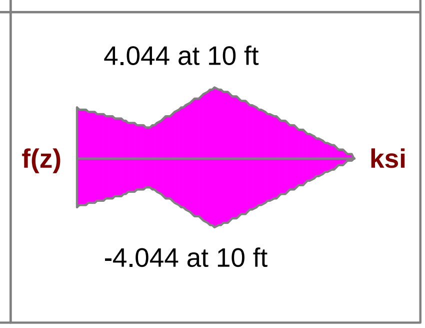












AISC 14th(360-10): ASD Code Check
Direct Analysis Method

Max Bending Check	**0.135 (LC 25)**	Max Shear Check	**0.008 (y) (LC 33)**	Max Defl Ratio	**L/10000**
Location	**10 ft**	Location	**10 ft**	Location	**0 ft**
Equation	**H1-1b**			Span	**0**

Bending Flange	**Non-Compact**	Compression Flange	**Non-Slender**
Bending Web	**Compact**	Compression Web	**Non-Slender**

			y-y	z-z
Fy	**50 ksi**	Lb	**10 ft**	**10 ft**
Pnc/om	**191.199 k**	KL/r	**59.529**	**34.572**
Pnt/om	**273.353 k**			
Mny/om	**35.124 k-ft**	L Comp Flange	**10 ft**	
Mnz/om	**75.767 k-ft**	L-torque	**20 ft**	
Vny/om	**45.6 k**	Tau_b	**1**	
Vnz/om	**125.03 k**			
Cb	**1.737**			

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DBL TRUSS H-BRACE/STRUT : L4x3x5/16

PER RISA, MAX. DR = 0.88 < 1.0 OK✓

CONNECTION DESIGN

MAX. COMPRESSION = 9.5 KIPS
MAX. TENSION = 19 KIPS

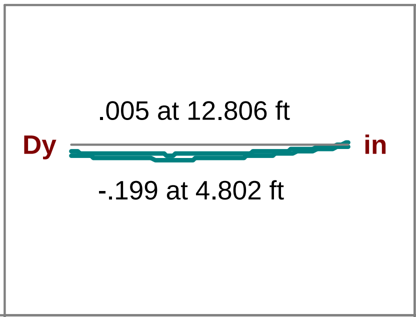
MAX. COMPRESSION (SEISMIC) = 9 KIPS x 2.0 = 18 KIPS
MAX. TENSION (SEISMIC) = 19 KIPS x 2.0 = 38 KIPS

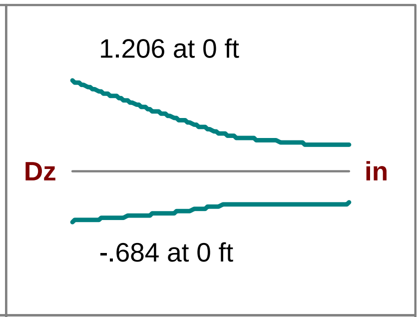
USE (2) 1"Ø BOLTS

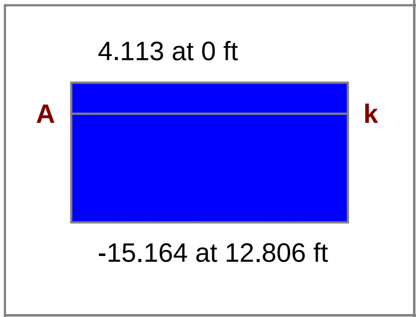
HBrace: ***M1085***

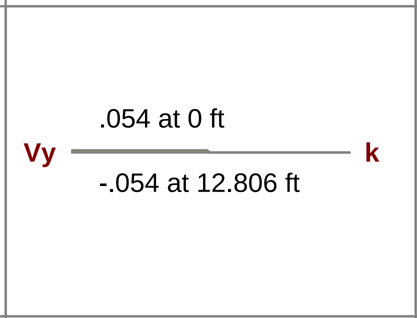
Shape:	**L4x3x5**
Material:	**A36 Gr.36**
Length:	**12.806 ft**
I Joint:	**N141A**
J Joint:	**N786B**

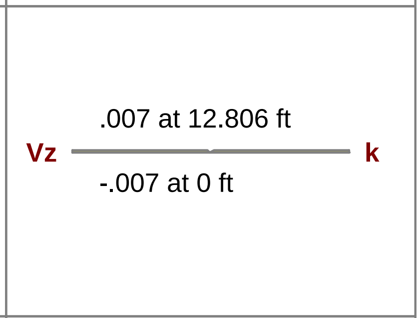
Envelope
Code Check: **0.337 (LC 52)**
Report Based On 97 Sections

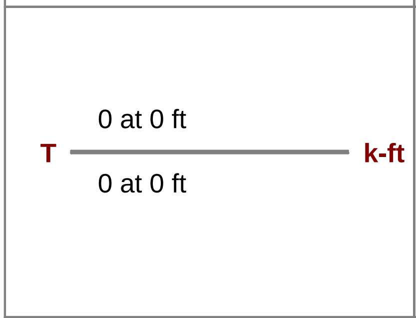


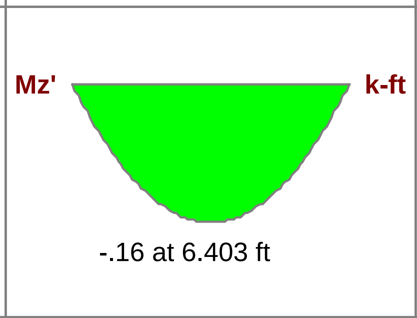


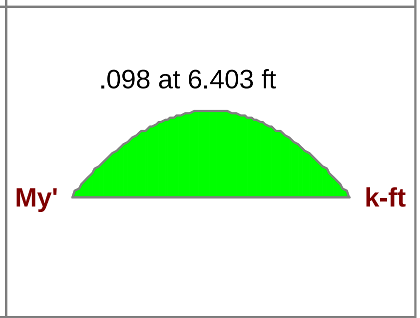


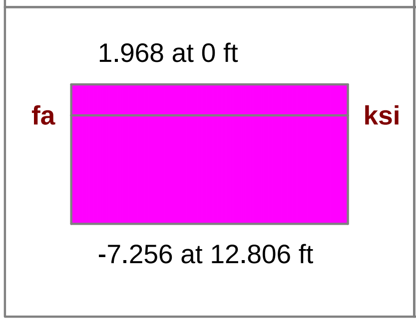


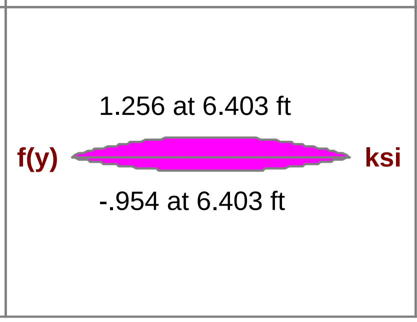


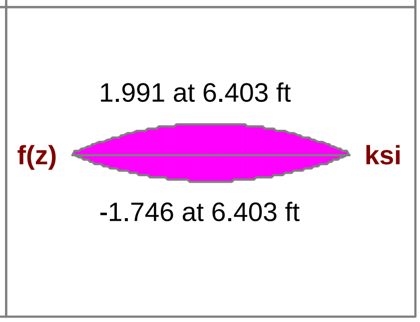












AISC 14th(360-10): ASD Code Check

Direct Analysis Method

Max Bending Check	**0.337 (LC 52)**	Max Shear Check	**0.005 (y) (LC 40)**
Location	**12.806 ft**	Location	**0 ft**
Equation	**H2-1***	Max Defl Ratio	**L/10000**

Bending Flange	**Compact**	Compression Flange	**Non-Slender**	**Qs=.997**
Bending Web	**Compact**	Compression Web	**Slender**	**Qa=1**

			y-y'	z-z'
Fy	**36 ksi**	Lb	**12.806 ft**	**12.806 ft**
Pnc/om	**5.415 k**	KL/r	**240.87**	**109.33**
Pnt/om	**45.054 k**			
Mny'/om	**1.514 k-ft**	L Comp Flange	**12.806 ft**	
Mnz'/om	**2.953 k-ft**	L-torque	**12.806 ft**	
Vny/om	**16.168 k**	Tau_b	**1**	
Vnz/om	**12.126 k**			
Cb	**1.136**			

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

Bin Wall / Footing Design

NOTE: WIND LOAD FAR EXCEEDS SEISMIC

Loading Scenarios

1. Max Column Vertical Load (Corner Column Controls)
2. Max Column Uplift (Corner Column Controls)
3. Max Material Surcharge + Lateral Load

Controlling Column Reactions Per Model

CONSERVATIVE, MODEL POLISHED TO 169 KIP RXN

① $P_{MAX} = 172^{KIPS}$ @ Building #1, Corner Controls (Adjacent Column = $30^{K}\downarrow$)
② $P_{UPLIFT} = 42^{KIPS}$ @ Building #1, Corner Controls (Adjacent Column = $20^{K}\downarrow$)

③ $P_{LATERAL} = 19.0^{KIPS}$ @ Building #1, Negative Pressure Adds to Surcharge,
$P_{DL} = 10^{KIPS}$ P_{WIND} = Horizontal @ 20' OC
P_{DL} = 20' OC

MAX. COMPRESSIVE STRESS = 172000 LBS (12" x 36") = 398 psi, OK BY INSPECTION

Material Surcharge Load

$\gamma_{GRAVEL} = 105$ PCF $H = 12'$ (Wall Height)

$\phi = 33^{\circ}$ Angle of Repose = 37° ($\sim\beta$)

Per Log Spiral, $K_a = 0.45$

$P_A = K_a \cdot \gamma \cdot H^2/2 = 3402$ PLF @ $H/3 = 4'$ Above Footing

WORST CASE LOADING IS BUILDING WIND REACTION + UNEQUAL SURCHARGE FROM THE ASPHALT STOCKPILE. THE WALL WAS MODELED AS A 40FT LONG CANTILEVER RETAINING WALL WITH FULL HEIGHT SURCHARGE FROM THE ASPHALT MATERIAL. THIS IS VERY CONSERVATIVE AS THE STOCKPILE CAN ONLY REACH THE FULL HEIGHT OVER A SHORT LENGTH OF THE WALL AND THIS NEGLECTS ANY OUT OF PLANE RESISTANCE FROM THE ADACENT WALL OR SOIL EMBEDMENT ON THE OPPOSITE SIDE OF THE WALL.

SMITH & MONROE & GRAY ENGINEERS, INC.

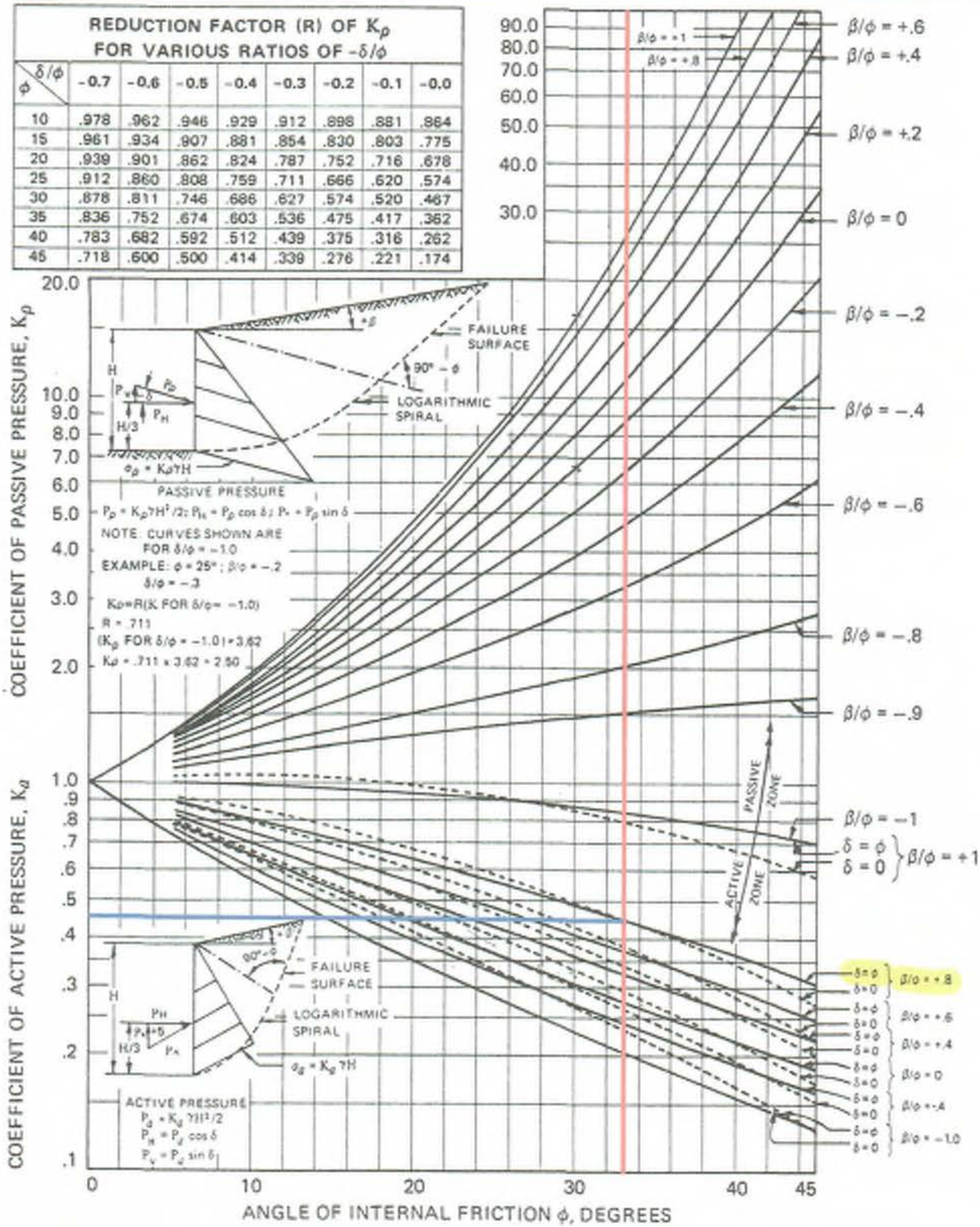
CIVIL & STRUCTURAL ENGINEERS
MATERIAL HANDLING & MECHANICAL

CLIENT ________________
PROJECT ________________

BY __________ DATE __________ REV __________
JOB NO __________ SHEET ______ OF ______

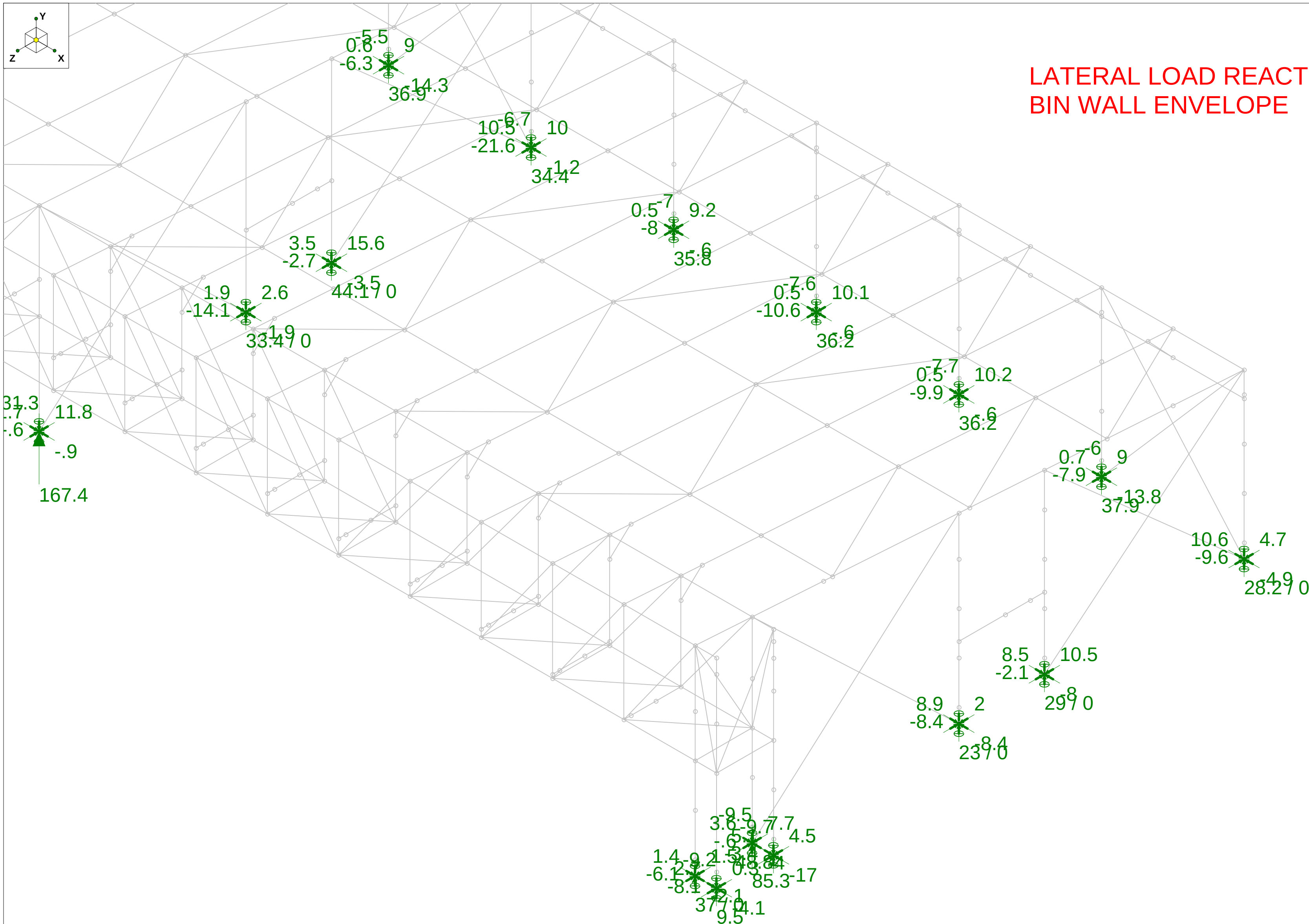
REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	−0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



Active and passive coefficients with wall friction and sloping backfill (after Caquot and Kerisel[8])

SHEET B125

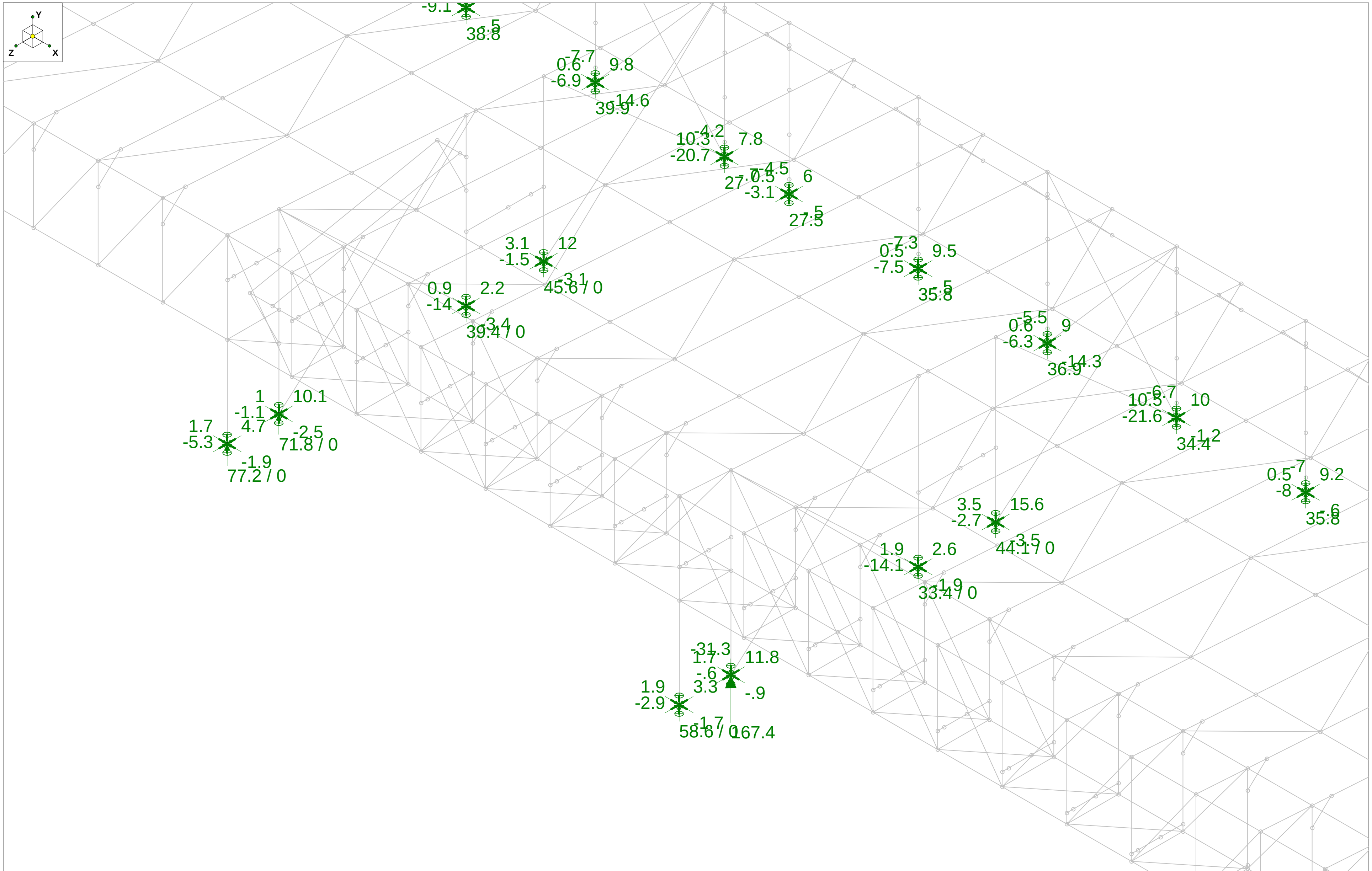


LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

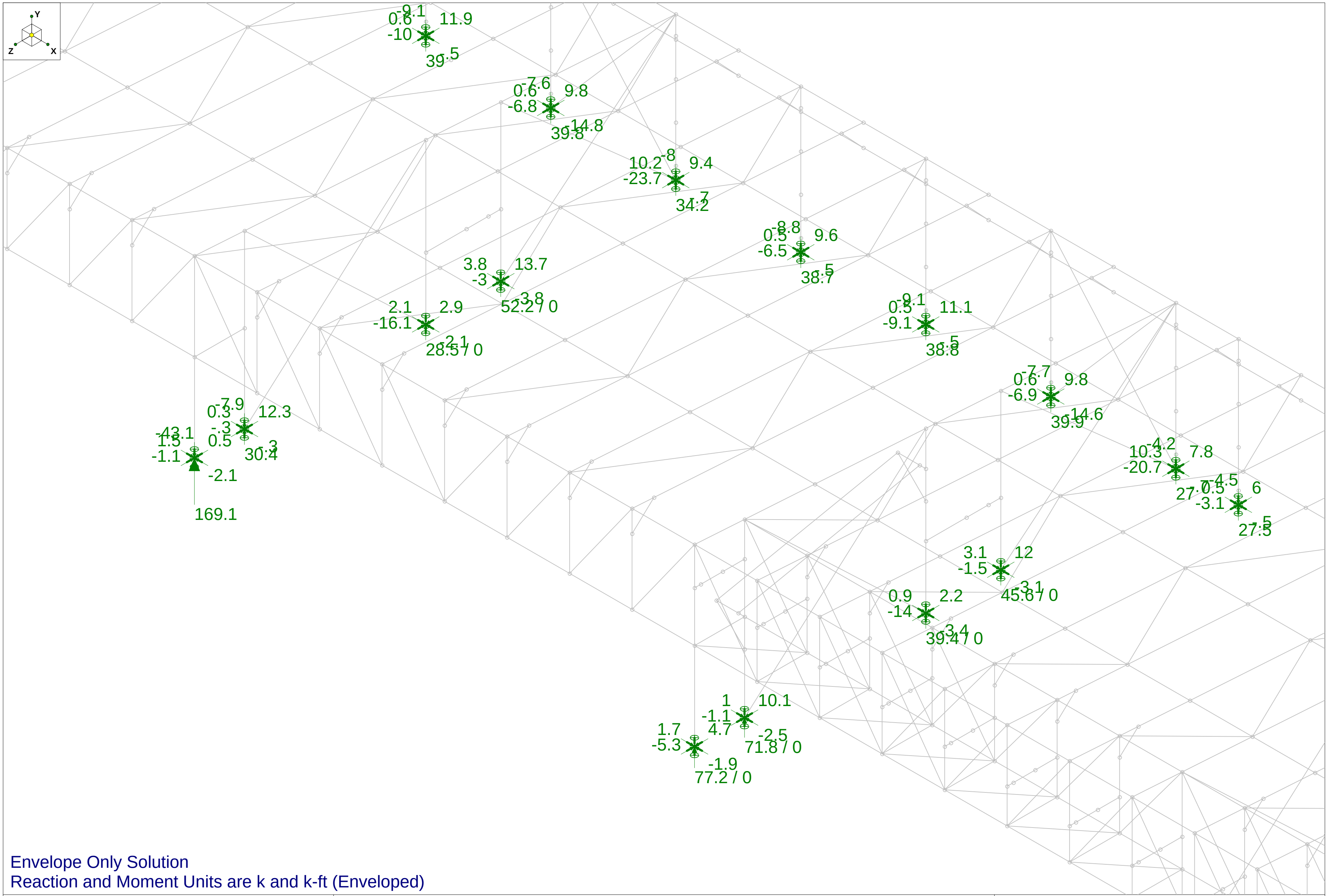
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 2
BS		Apr 8, 2019 at 4:34 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B126



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

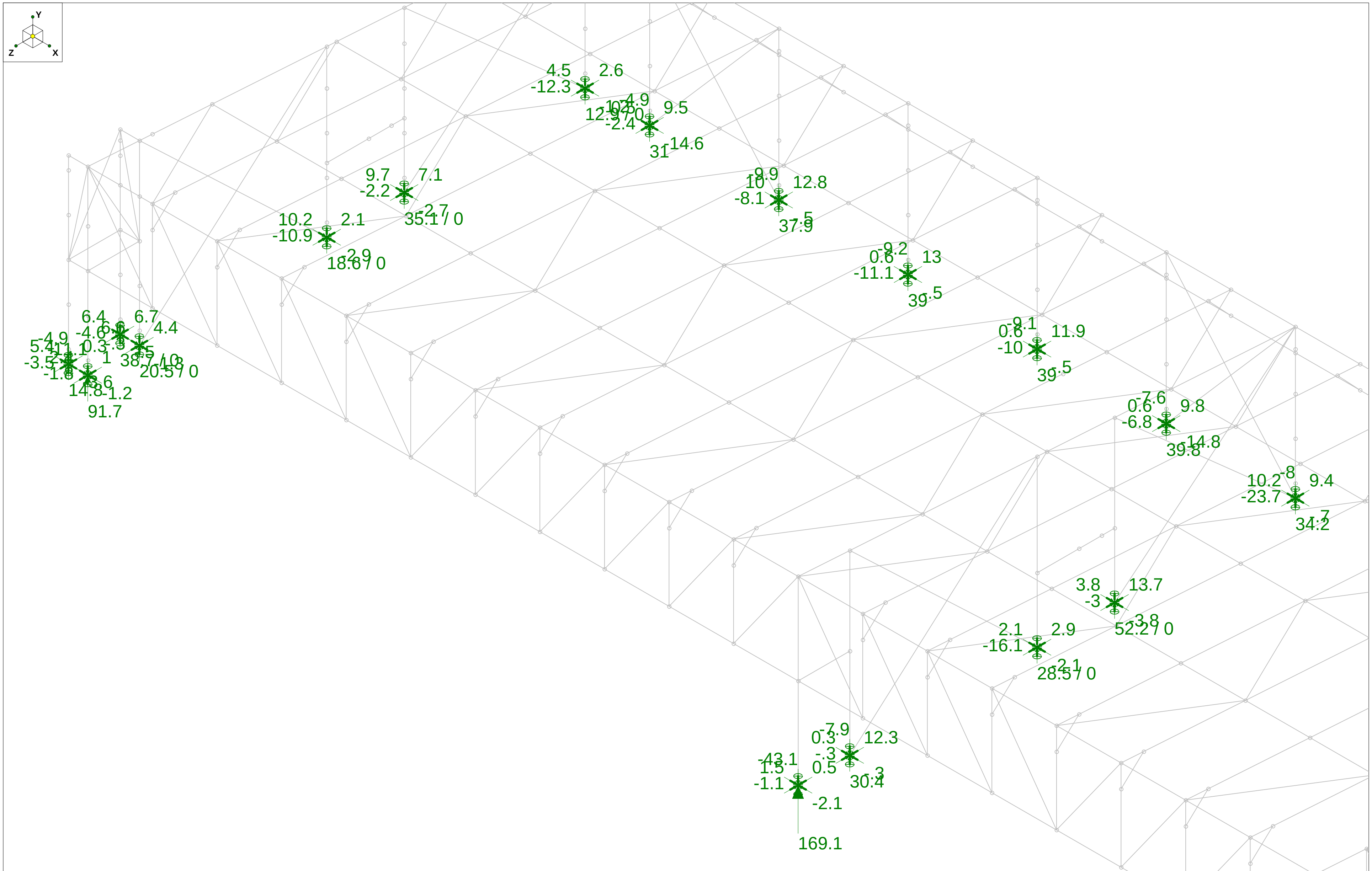
SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 3
BS		Apr 8, 2019 at 4:34 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 4
BS		Apr 8, 2019 at 4:35 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

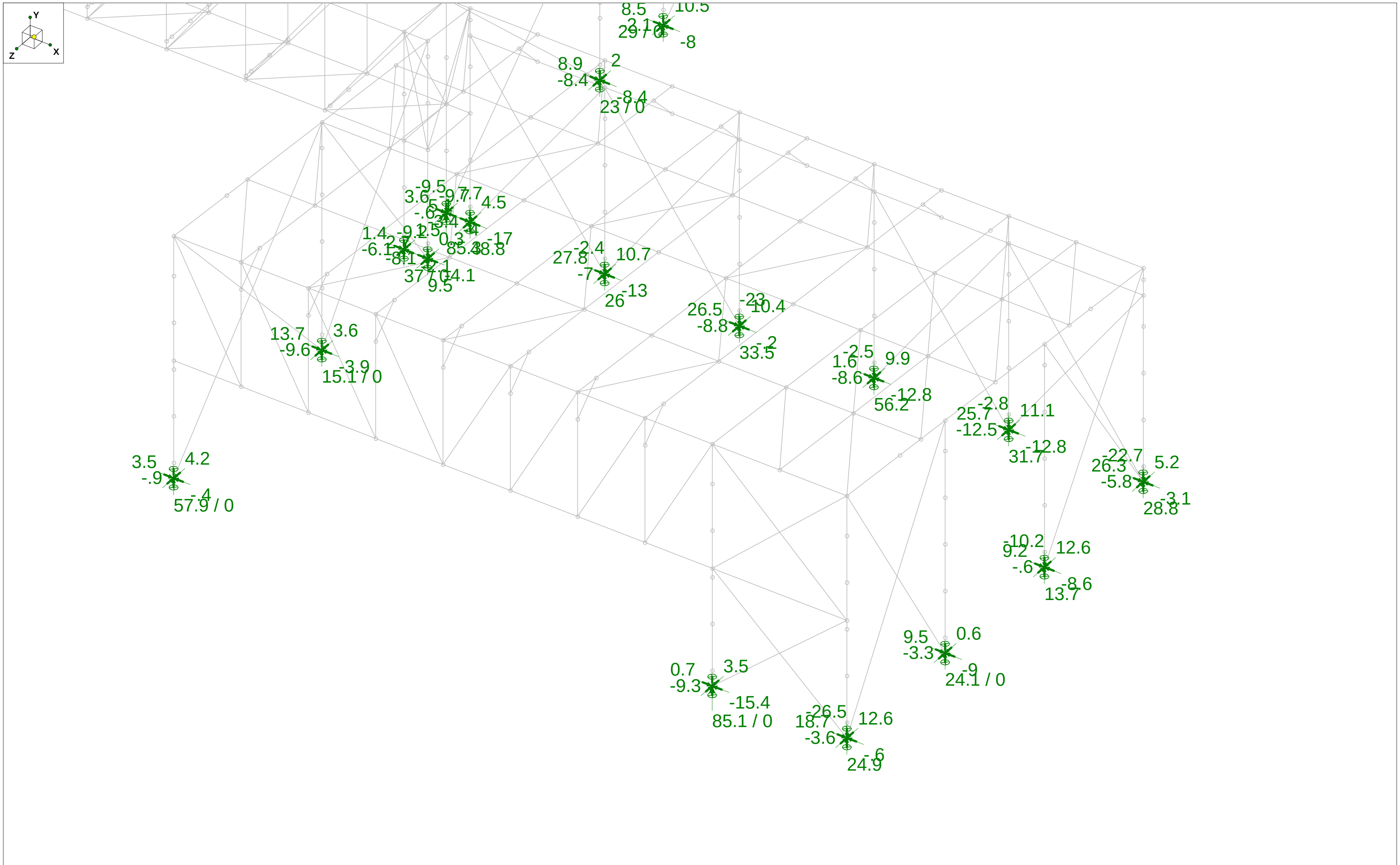
SHEET B128



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 5
BS		Apr 8, 2019 at 4:35 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

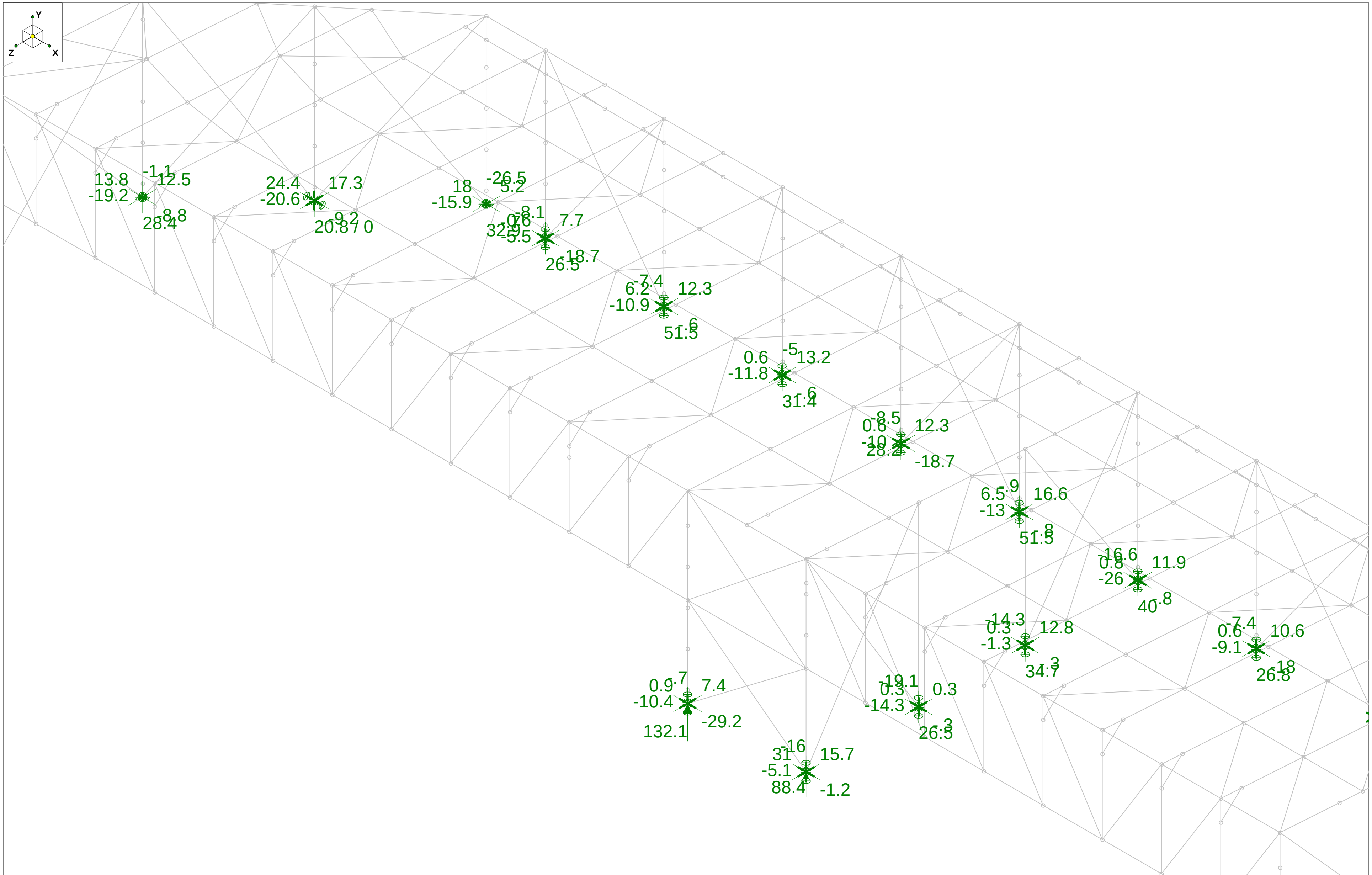
SHEET B129



Envelope Only Solution
Member Axial Forces (k) (Enveloped)
Reaction and Moment Units are k and k-ft (Enveloped)

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 1
BS		Apr 8, 2019 at 4:33 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

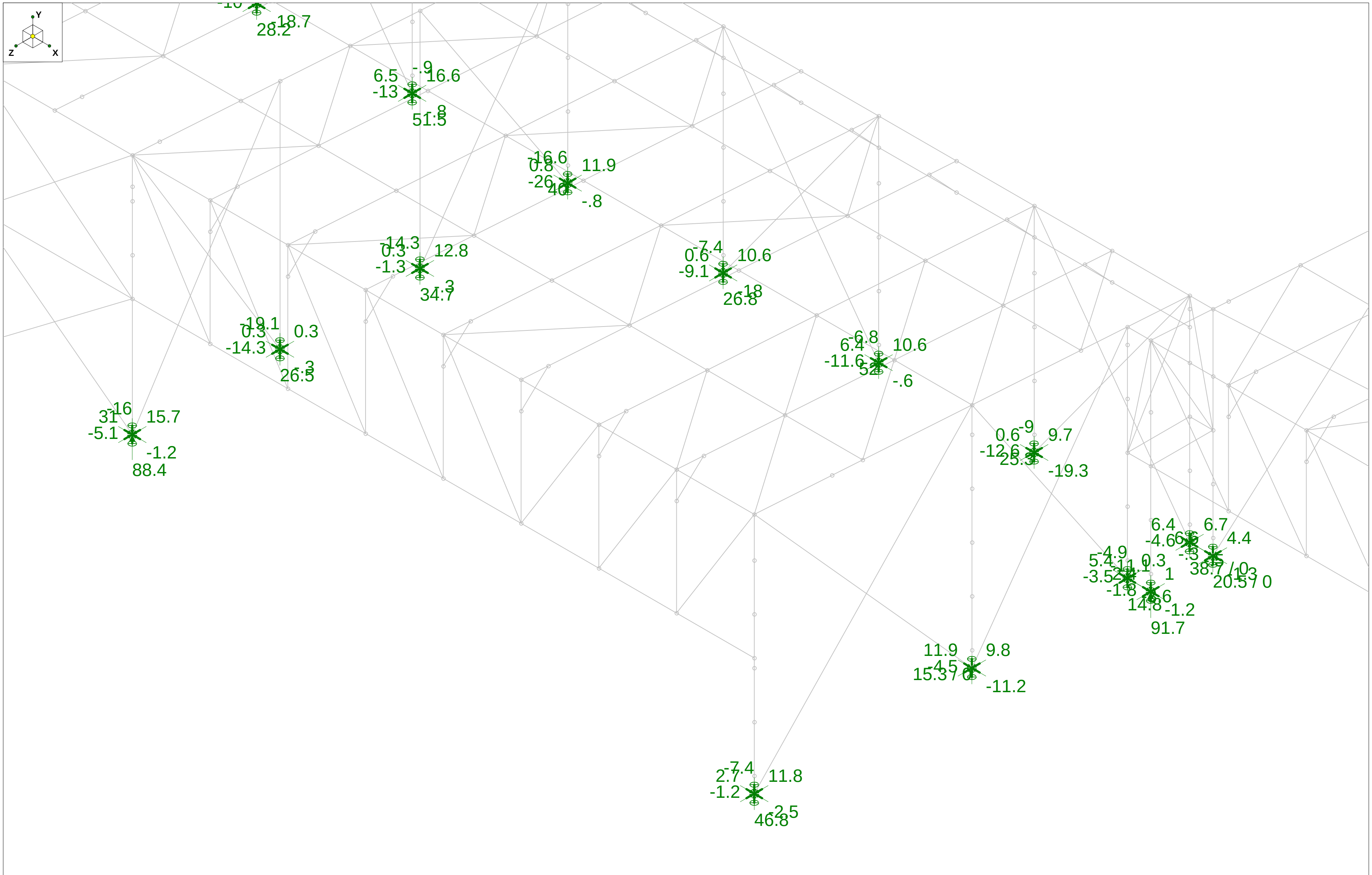
SHEET B130



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 6
BS		Apr 8, 2019 at 4:36 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

SHEET B131



Envelope Only Solution
Reaction and Moment Units are k and k-ft (Enveloped)

SMG ENGINEERS	AGGREGATE STORAGE BUILDING	SK - 7
BS		Apr 8, 2019 at 4:36 PM
18-183B		BLDG 123 Model Rev0 11.1.18.r3d

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N668	max	1.381	69	171.718	65	.629	61	0	55	.001	72	0	55
2		min	-1.706	70	-41.369	66	-1.318	66	0	55	-.001	59	0	55
3	N785A	max	3.598	69	160.49	65	15.471	71	0	55	.002	70	0	55
4		min	-2.389	70	-28.224	66	-.704	65	0	55	-.023	69	0	55
5	N6 1	max	1.084	69	108.717	65	9.491	61	0	55	.026	70	0	55
6		min	-30.618	70	1.733	66	-12.075	60	0	55	-.266	59	0	55
7	N582A	max	33.118	69	102.279	59	9.9	61	0	55	.037	70	0	55
8		min	-1.02	70	-14.151	68	-12.678	60	0	55	-.234	59	0	55
9	N3	max	2.308	69	93.095	65	.737	71	0	55	.013	65	0	55
10		min	-.838	70	-11.262	66	-1.856	72	0	55	-.006	62	0	55
11	N544A	max	3.418	63	89.47	65	10.333	71	0	55	.009	64	0	55
12		min	-3.87	62	-6.06	66	-.61	72	0	55	-.008	61	0	55
13	N564A	max	.383	69	85.525	67	6.145	61	0	55	.003	62	0	55
14		min	-18.804	70	-3.408	66	-5.325	66	0	55	-.005	59	0	55
15	N783A	max	3.168	69	75.87	65	10.518	71	0	55	2.941	71	0	55
16		min	-4.087	70	2.759	68	-.691	72	0	55	-2.699	72	0	55
17	N676A	max	1.619	69	73.291	65	2.97	71	0	55	.009	62	0	55
18		min	-1.441	70	7.037	66	-7.378	72	0	55	-.015	59	0	55
19	N6	max	1.75	69	70.768	65	4.869	62	0	55	0	71	0	55
20		min	-1.177	70	40.781	66	-10.883	65	0	55	-.002	72	0	55
21	N4 2	max	2.459	63	70.29	65	13.252	62	0	55	0	62	0	55
22		min	-2.61	60	-10.59	68	-1.067	64	0	55	-.003	65	0	55
23	N170A	max	3.261	68	69.595	65	16.348	62	0	55	.002	64	0	55
24		min	-.567	70	-10.071	68	-1.176	66	0	55	0	62	0	55
25	N3 1	max	3.384	68	63.964	65	10.126	62	0	55	.003	59	0	55
26		min	-.397	70	-3.565	68	-.947	66	0	55	-.002	62	0	55
27	N78 1	max	.808	69	55.207	69	10.319	68	0	55	0	61	0	55
28		min	-15.708	70	-.227	70	-6.885	59	0	55	0	70	0	55
29	N70 1	max	.784	69	53.093	69	11.42	68	0	55	.003	59	0	55
30		min	-15.465	70	-2.695	70	-8.691	59	0	55	0	55	0	55

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N668	max	1.381	69	171.718	65	.629	61	0	55	.001	72	0	55
2		min	-1.706	70	-41.369	66	-1.318	66	0	55	-.001	59	0	55
3	N785A	max	3.598	69	180.49	65	15.471	71	0	55	.002	70	0	55
4		min	-2.389	70	-28.224	66	-.704	65	0	55	-.023	69	0	55
5	N86A	max	18.012	66	32.343	61	5.207	68	0	68	.003	69	0	65
6		min	-.705	67	-26.288	66	-15.32	59	0	65	-.001	66	0	68
7	N2 1	max	26.618	69	30.62	70	6.123	68	0	55	0	67	0	55
8		min	-4.07	60	-24.246	69	-17.681	59	0	55	0	62	0	55
9	N551B	max	5.823	63	17.14	68	.309	71	0	55	.02	65	0	55
10		min	-2.856	60	-22.588	59	-13.495	59	0	55	-.006	70	0	55
11	N66 1	max	28.718	69	38.371	70	10.425	68	0	55	.003	59	0	55
12		min	-.814	70	-21.064	69	-7.054	59	0	55	0	70	0	55
13	N4 1	max	17.713	69	32.81	59	12.915	61	0	55	0	66	0	55
14		min	-1.682	60	-20.297	68	-4.201	60	0	55	0	62	0	55
15	N74 1	max	26.842	69	37.455	70	11.277	68	0	55	.002	62	0	55
16		min	-.842	70	-18.542	69	-8.507	59	0	55	0	70	0	55
17	N582A	max	33.118	69	102.279	59	9.9	61	0	55	.037	70	0	55
18		min	-1.02	70	-14.151	68	-12.678	60	0	55	-.234	59	0	55
19	N549A	max	3.326	68	12.213	68	.29	71	0	55	.011	66	0	55
20		min	-3.434	70	-12.569	59	-10.816	59	0	55	-.01	65	0	55
21	N1 1	max	1.855	69	47.925	69	6.224	68	0	55	0	68	0	55
22		min	-17.669	70	-11.708	70	-3.861	59	0	55	0	70	0	55
23	N3	max	2.308	69	93.095	65	.737	71	0	55	.013	65	0	55
24		min	-.838	70	-11.262	66	-1.856	72	0	55	-.006	62	0	55
25	N750A	max	11.455	69	38.247	65	9.77	68	0	55	.011	69	0	55
26		min	-.523	70	-10.792	66	-7.414	59	0	55	0	70	0	55
27	N4 2	max	2.459	63	70.29	65	13.252	62	0	55	0	62	0	55
28		min	-2.61	60	-10.59	68	-1.067	64	0	55	-.003	65	0	55
29	N170A	max	3.261	68	69.595	65	16.348	62	0	55	.002	64	0	55
30		min	-.567	70	-10.071	68	-1.176	66	0	55	0	62	0	55

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N582A	max	33.118	69	102.279	59	9.9	61	0	55	.037	70	0	55
2		min	-1.02	70	-14.151	68	-12.678	60	0	55	-.234	59	0	55
3	N66 1	max	28.718	69	38.371	70	10.425	68	0	55	.003	59	0	55
4		min	-.814	70	-21.064	69	-7.054	59	0	55	0	70	0	55
5	N74 1	max	26.842	69	37.455	70	11.277	68	0	55	.002	62	0	55
6		min	-.842	70	-18.542	69	-8.507	59	0	55	0	70	0	55
7	N2 1	max	26.618	69	30.62	70	6.123	68	0	55	0	67	0	55
8		min	-4.07	60	-24.246	69	-17.681	59	0	55	0	62	0	55
9	N85	max	24.386	66	20.83	65	16.735	62	0	70	0	55	0	55
10		min	-8.968	61	1.697	66	-19.982	59	0	62	0	55	0	55
11	N86A	max	18.012	66	32.343	61	5.207	68	0	68	.003	69	0	65
12		min	-.705	67	-26.288	66	-15.32	59	0	65	-.001	66	0	68
13	N4 1	max	17.713	69	32.81	59	12.915	61	0	55	0	66	0	55
14		min	-1.682	60	-20.297	68	-4.201	60	0	55	0	62	0	55
15	N84	max	13.772	66	28.467	66	11.964	62	0	66	0	65	0	61
16		min	-8.652	67	-1.949	70	-21.146	59	0	61	0	61	0	66
17	N558	max	13.672	66	16.289	65	8.63	62	0	55	.008	70	0	55
18		min	-3.856	63	6.48	66	-12.428	59	0	55	-.014	69	0	55
19	N2	max	13.345	69	28.532	70	4.909	68	0	55	0	63	0	55
20		min	-4.262	62	-2.494	66	-14.921	59	0	55	0	62	0	55
21	N5	max	12.429	69	34.215	61	9.951	68	0	55	0	69	0	55
22		min	-1.005	59	-6	66	-25.587	59	0	55	0	70	0	55
23	N91 1	max	11.92	63	16.666	65	11.024	62	0	55	.016	70	0	55
24		min	-11.218	62	6.675	66	-15.301	59	0	55	-.007	69	0	55
25	N675A	max	11.914	69	28.238	70	7.813	68	0	55	.083	71	0	55
26		min	-.691	70	-6.851	66	-19.483	59	0	55	0	67	0	55
27	N667	max	11.669	69	33.874	61	9.436	68	0	55	.001	62	0	55
28		min	-.718	70	-8.434	66	-23.487	59	0	55	0	70	0	55
29	N750A	max	11.455	69	38.247	65	9.77	68	0	55	.011	69	0	55
30		min	-.523	70	-10.792	66	-7.414	59	0	55	0	70	0	55

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N6 1	max	1.084	69	108.717	65	9.491	61	0	55	.026	70	0	55
2		min	-30.618	70	1.733	66	-12.075	60	0	55	-.266	59	0	55
3	N564A	max	.383	69	85.525	67	6.145	61	0	55	.003	62	0	55
4		min	-18.804	70	-3.408	66	-5.325	66	0	55	-.005	59	0	55
5	N1 1	max	1.855	69	47.925	69	6.224	68	0	55	0	68	0	55
6		min	-17.669	70	-11.708	70	-3.861	59	0	55	0	70	0	55
7	N78 2	max	.579	69	31.787	69	9.979	68	0	55	.002	68	0	55
8		min	-17.348	70	-5.69	70	-6.405	59	0	55	-.002	70	0	55
9	N5 1	max	.868	69	36.474	67	17.686	68	0	55	.081	59	0	55
10		min	-16.444	70	-5.424	60	-14.371	59	0	55	-.01	70	0	55
11	N70 2	max	.622	69	30.878	69	12.703	68	0	55	.005	65	0	55
12		min	-16.263	70	-4.225	70	-10.543	59	0	55	0	70	0	55
13	N78 1	max	.808	69	55.207	69	10.319	68	0	55	0	61	0	55
14		min	-15.708	70	-.227	70	-6.885	59	0	55	0	70	0	55
15	N551A	max	.627	69	28.364	69	7.567	68	0	55	.11	59	0	55
16		min	-15.658	70	-4.917	70	-5.085	59	0	55	-.072	61	0	55
17	N70 1	max	.784	69	53.093	69	11.42	68	0	55	.003	59	0	55
18		min	-15.465	70	-2.695	70	-8.691	59	0	55	0	55	0	55
19	N756	max	.585	69	40.155	67	9.745	68	0	55	.008	69	0	55
20		min	-14.857	70	-7.121	66	-6.76	59	0	55	-.002	70	0	55
21	N742	max	.533	69	31.204	67	4.975	68	0	55	.011	69	0	55
22		min	-14.633	70	-3.94	66	-2.419	60	0	55	0	70	0	55
23	N748	max	.574	69	40.418	67	9.715	68	0	55	.008	69	0	55
24		min	-14.621	70	-7.431	66	-6.686	59	0	55	-.001	70	0	55
25	N62	max	.868	68	36.807	65	7.852	68	0	55	.007	69	0	55
26		min	-14.278	70	-4.86	66	-3.952	59	0	55	-.001	70	0	55
27	N78	max	.799	61	38.135	65	7.921	68	0	55	.003	69	0	55
28		min	-13.866	70	-5.32	66	-3.796	59	0	55	-.002	70	0	55
29	N91 1	max	11.92	63	16.666	65	11.024	62	0	55	.016	70	0	55
30		min	-11.218	62	6.675	66	-15.301	59	0	55	-.007	69	0	55

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N134	max	3.454	69	47.542	59	19.886	71	0	55	0	67	0	55
2		min	-3.453	70	13.059	68	-2.742	72	0	55	-.03	70	0	55
3	N5_1	max	.868	69	36.474	67	17.686	68	0	55	.081	59	0	55
4		min	-16.444	70	-5.424	60	-14.371	59	0	55	-.01	70	0	55
5	N157A	max	10.52	69	42.236	61	17.567	68	0	55	.068	59	0	55
6		min	-.836	70	-8.745	66	-14.445	59	0	55	-.006	70	0	55
7	N85	max	24.386	66	20.83	65	16.735	62	0	70	0	55	0	55
8		min	-8.968	61	1.697	66	-19.982	59	0	62	0	55	0	55
9	N170A	max	3.261	68	69.595	65	16.348	62	0	55	.002	64	0	55
10		min	-.567	70	-10.071	68	-1.176	66	0	55	0	62	0	55
11	N785A	max	3.598	69	160.49	65	15.471	71	0	55	.002	70	0	55
12		min	-2.389	70	-28.224	66	-.704	65	0	55	-.023	69	0	55
13	N133	max	8.47	63	33.368	59	13.407	71	0	55	.077	63	0	55
14		min	-7.972	60	12.58	71	-2.076	72	0	55	-.009	62	0	55
15	N681A	max	3.078	69	44.487	59	13.346	71	0	55	.755	71	0	55
16		min	-3.077	70	16.878	68	-1.832	72	0	55	-.64	72	0	55
17	N4_2	max	2.459	63	70.29	65	13.252	62	0	55	0	62	0	55
18		min	-2.61	60	-10.59	68	-1.067	64	0	55	-.003	65	0	55
19	N673	max	3.83	69	52.046	59	13.248	71	0	55	.008	69	0	55
20		min	-3.83	70	18.742	68	-3.036	72	0	55	-.032	59	0	55
21	N4_1	max	17.713	69	32.81	59	12.915	61	0	55	0	66	0	55
22		min	-1.682	60	-20.297	68	-4.201	60	0	55	0	62	0	55
23	N70_2	max	.622	69	30.878	69	12.703	68	0	55	.005	65	0	55
24		min	-16.263	70	-4.225	70	-10.543	59	0	55	0	70	0	55
25	N89A	max	.659	69	30.983	67	12.088	68	0	55	.003	62	0	55
26		min	-.633	70	-4.285	66	-9.318	59	0	55	-.001	70	0	55
27	N66_2	max	.629	69	30.964	67	12.072	68	0	55	.007	65	0	55
28		min	-.617	70	-4.174	66	-9.373	59	0	55	0	70	0	55
29	N892A	max	9.163	63	29.808	59	12.034	62	0	55	.04	63	0	55
30		min	-8.624	62	-9.492	62	-.616	72	0	55	-.005	62	0	55

LATERAL LOAD REACTION TO BIN WALL ENVELOPE

Envelope Joint Reactions

	Joint		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N5	max	12.429	69	34.215	61	9.951	68	0	55	0	69	0	55
2		min	-1.005	59	-6	66	-25.587	59	0	55	0	70	0	55
3	N667	max	11.669	69	33.874	61	9.436	68	0	55	.001	62	0	55
4		min	-.718	70	-8.434	66	-23.487	59	0	55	0	70	0	55
5	N84	max	13.772	66	28.467	66	11.964	62	0	66	0	65	0	61
6		min	-8.652	67	-1.949	70	-21.146	59	0	61	0	61	0	66
7	N85	max	24.386	66	20.83	65	16.735	62	0	70	0	55	0	55
8		min	-8.968	61	1.697	66	-19.982	59	0	62	0	55	0	55
9	N675A	max	11.914	69	28.238	70	7.813	68	0	55	.083	71	0	55
10		min	-.691	70	-6.851	66	-19.483	59	0	55	0	67	0	55
11	N124	max	1.895	69	34.499	61	2.619	71	0	55	.008	70	0	55
12		min	-1.896	70	2.827	60	-17.975	72	0	55	-.002	69	0	55
13	N2 1	max	26.618	69	30.62	70	6.123	68	0	55	0	67	0	55
14		min	-4.07	60	-24.246	69	-17.681	59	0	55	0	62	0	55
15	N671	max	2.087	69	28.15	61	2.894	71	0	55	.032	59	0	55
16		min	-2.087	70	5.372	72	-16.175	72	0	55	.002	67	0	55
17	N86A	max	18.012	66	32.343	61	5.207	68	0	68	.003	69	0	65
18		min	-.705	67	-26.288	66	-15.32	59	0	65	-.001	66	0	68
19	N91 1	max	11.92	63	16.666	65	11.024	62	0	55	.016	70	0	55
20		min	-11.218	62	6.675	66	-15.301	59	0	55	-.007	69	0	55
21	N2	max	13.345	69	28.532	70	4.909	68	0	55	0	63	0	55
22		min	-4.262	62	-2.494	66	-14.921	59	0	55	0	62	0	55
23	N157A	max	10.52	69	42.236	61	17.567	68	0	55	.068	59	0	55
24		min	-.836	70	-8.745	66	-14.445	59	0	55	-.006	70	0	55
25	N5 1	max	.868	69	36.474	67	17.686	68	0	55	.081	59	0	55
26		min	-16.444	70	-5.424	60	-14.371	59	0	55	-.01	70	0	55
27	N679	max	1.033	69	40.845	61	2.189	71	0	55	2.048	71	0	55
28		min	-3.097	70	13.697	60	-13.813	72	0	55	-1.931	72	0	55
29	N551B	max	5.823	63	17.14	68	.309	71	0	55	.02	65	0	55
30		min	-2.856	60	-22.588	59	-13.495	59	0	55	-.006	70	0	55

Smith Monroe Gray
ENGINEERS, INC.

CLIENT
PROJECT
BY DATE REV.
JOB NO. SHEET OF

BIN WALL / FOOTING DESIGN (CONT'D)

FOOTING DESIGN PER RISA PL MODEL

* MAX DEMAND OCCURS @ LC1 (MAX VERTICAL LOAD, LRFD)

$M_{MAX} = M_U = 12.9^{k\text{-}ft}/ft$

$V_{MAX} = V_U = 8.8^{k\text{-}ft}/ft$

> USE 18" FOOTING W/ #6's @ 12 OC
> SEE SPREADSHEET ON NEXT PAGE

WALL DESIGN PER RISA PL MODEL

* MAX DEMAND OCCURS @ LC3 (SURCHARGE + WIND)

$M_{MAX} = M_U = 17.8^{k\text{-}ft}$

$V_{MAX} = V_U = 4^{KIPS}$

> USE 36" WALL W/ #6's @ 12" OC
> SEE SPREADSHEET

MAX. SOIL BEARING PER MODEL

$P_{MAX} = 700^{\#}$ @ 6" SQ

$P_{MAX} = 700^{\#} / 0.25\ ft^2 = 2800\ PSF < 3000\ PSF$ OK ✓

MIN STEEL (TEMP / SHRINKAGE)

MIN. VERTICAL, $\rho_l = 0.0015 \cdot 36" \cdot 12" = 0.65\ in^2$ (2) #6's @ 12" OC = 0.88 in²

HORIZ, $\rho_t = 0.0025 \cdot 36" \cdot 12" = 1.08\ in^2$ (2) #7's @ 12" OC = 1.2 in²

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES
Job #: 18-183B
By: BS

Project:
Date: 9/27/2018 Sheet of

Concrete Slab Design per ACI 318-08

IN COMPLIANCE W/ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 4 kips
Ultimate Moment, M_u = 17.8 ft-kips

Slab Properties:

Width = 12 in
Depth = 36 in
Cover = 3 in.
d = 32.63 in.
f'_c = 4000 psi
β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$
$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$
$\Phi V_c = \Phi V_n$ = 37.14 kips

Bending: $\phi = 0.9$
$\Phi M_n = \phi(As*fy*(d-a/2))$
ΦM_n = 63.96 k-ft.

Longitudinal Reinforcement:

Bar Size = 6
Spacing = 12 inches o.c.
f_y = 60000 psi

A_s = 0.44 in^2
a = 0.65 in
c = 0.76 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018
A_s min = 0.70 in^2 NO GOOD
max. spacing = 18.0 in

Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$
ε_t = 0.1256 **> 0.005, OK**

Demand Ratios:		
$V_u/\Phi V_n$ =	0.11	**SLAB IS OK IN SHEAR**
$M_u/\Phi M_n$ =	0.28	**SLAB IS OK IN BENDING**

Smith Monroe Gray
ENGINEERS, INC.

Client: LAKESIDE INDUSTRIES

Job #: 18-183B

By: BS

Project:

Date: 9/27/2018 Sheet of

Concrete Slab Design per ACI 318-08

IN COMPLIANCE
W/ACI 318-14

Applied Forces:

Ultimate Shear, V_u = 8.8 kips

Ultimate Moment, M_u = 12.9 ft-kips

Slab Properties:

Width = 12 in

Depth = 18 in

Cover = 3 in.

d = 14.63 in.

f'_c = 4000 psi

β_1 = 0.85

Capacity:

Shear: $\phi = 0.75$

$\Phi V_c = \Phi V_n = \phi*2*b*d*\sqrt{f'c}$

$\Phi V_c = \Phi V_n$ = 16.65 kips

Bending: $\phi = 0.9$

$\Phi M_n = \phi(As*fy*(d-a/2))$

ΦM_n = 28.32 k-ft.

Longitudinal Reinforcement:

Bar Size = 6

Spacing = 12 inches o.c.

f_y = 60000 psi

A_s = 0.44 in^2

a = 0.65 in

c = 0.76 in

Shrinkage and Temperature Reinforcing

Min. reinf. ratio = 0.0018

A_s min = 0.32 in^2 OK

max. spacing = 18.0 in

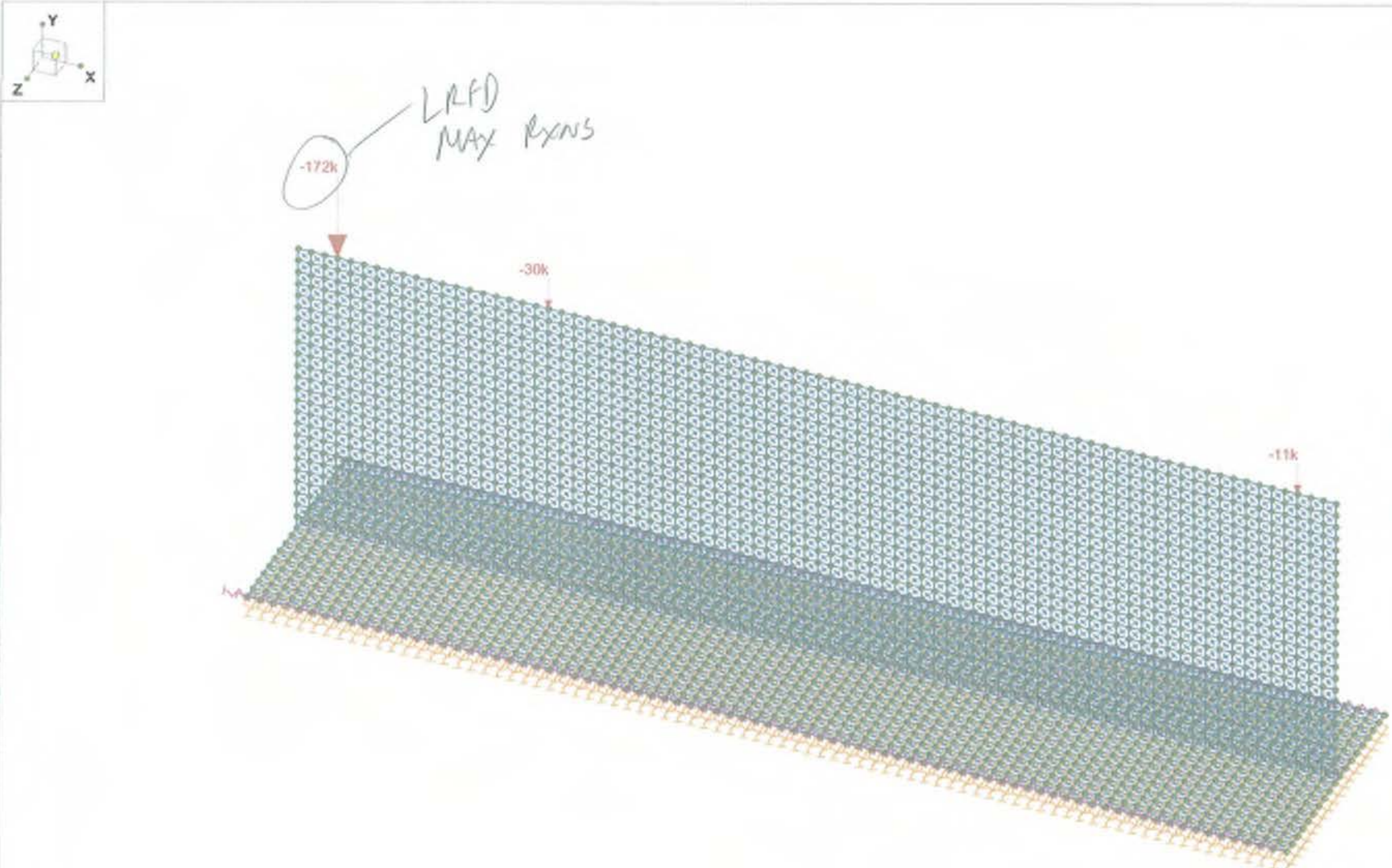
Check Tension Controlled (ACI 10.3.4)

$\varepsilon_t = [(d-c)/c]*0.003$

ε_t = 0.0546 > 0.005, OK

Demand Ratios:

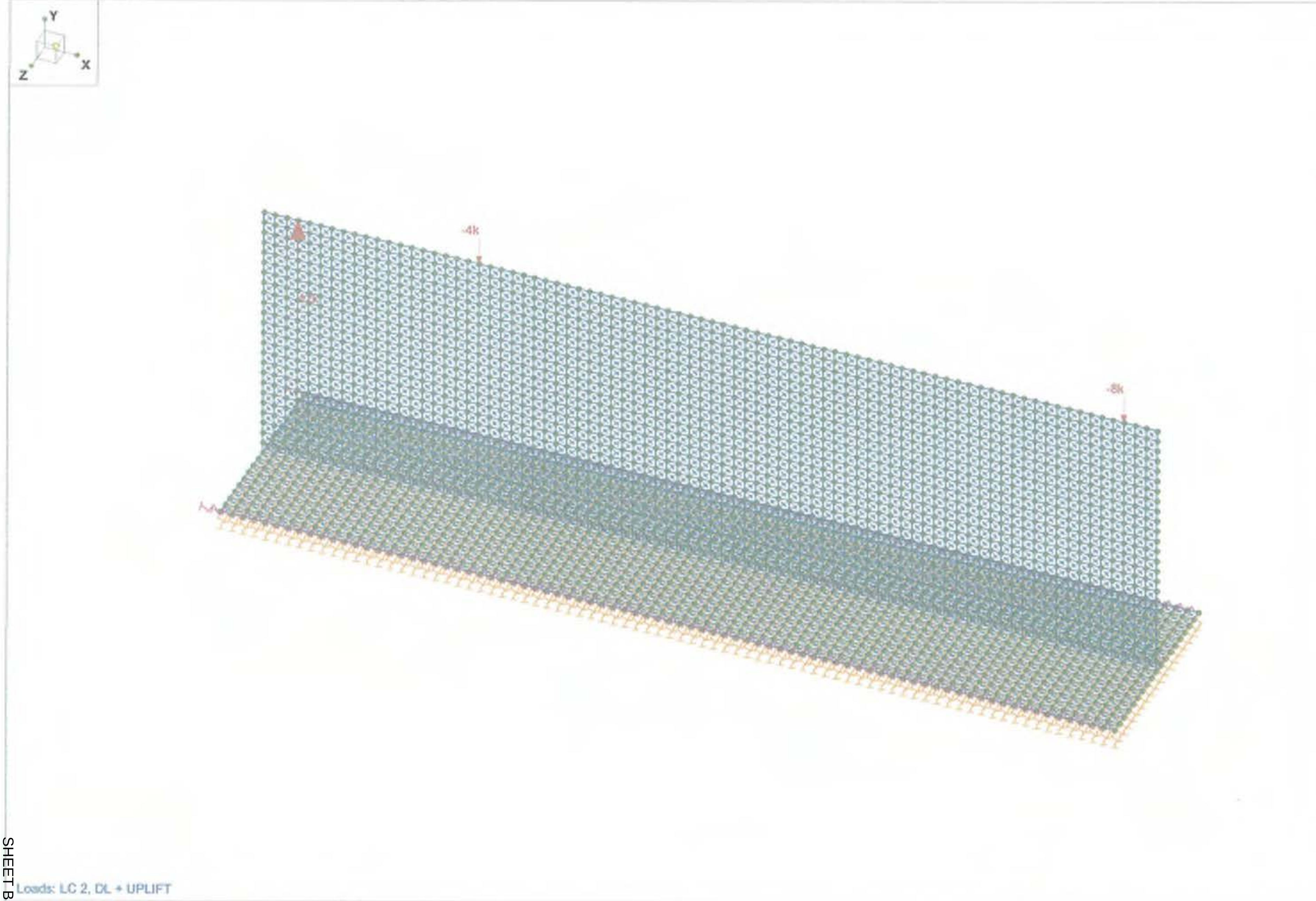
$V_u/\Phi V_n$ =	0.53	**SLAB IS OK IN SHEAR**
$M_u/\Phi M_n$ =	0.46	**SLAB IS OK IN BENDING**



Loads: LC 1, DL + MAX COLUMN LOAD

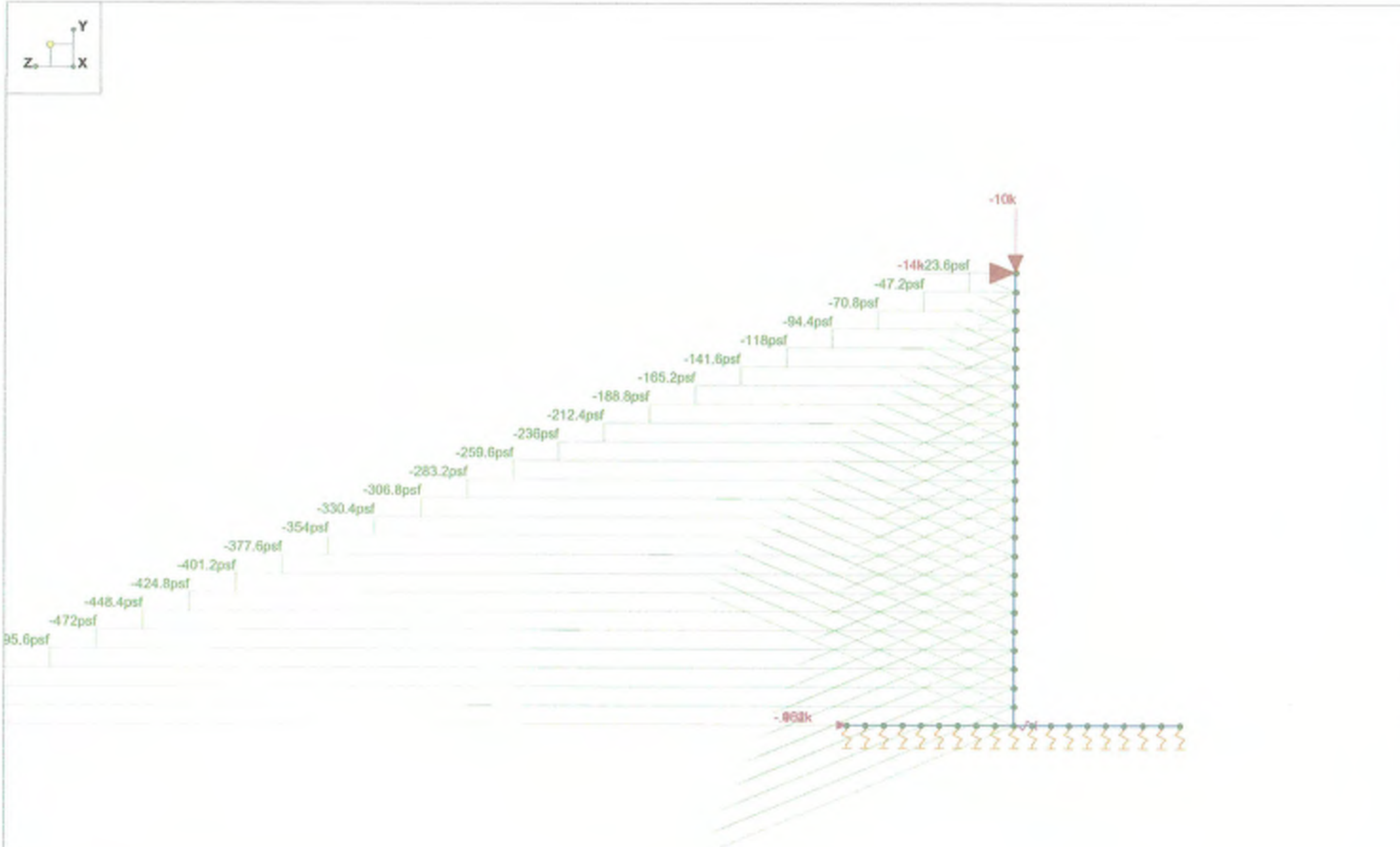
SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:14 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

SHEET B142



SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:14 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

SHEET B143



Loads: LC 3, DL + SURCHARGE + WIND

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:15 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

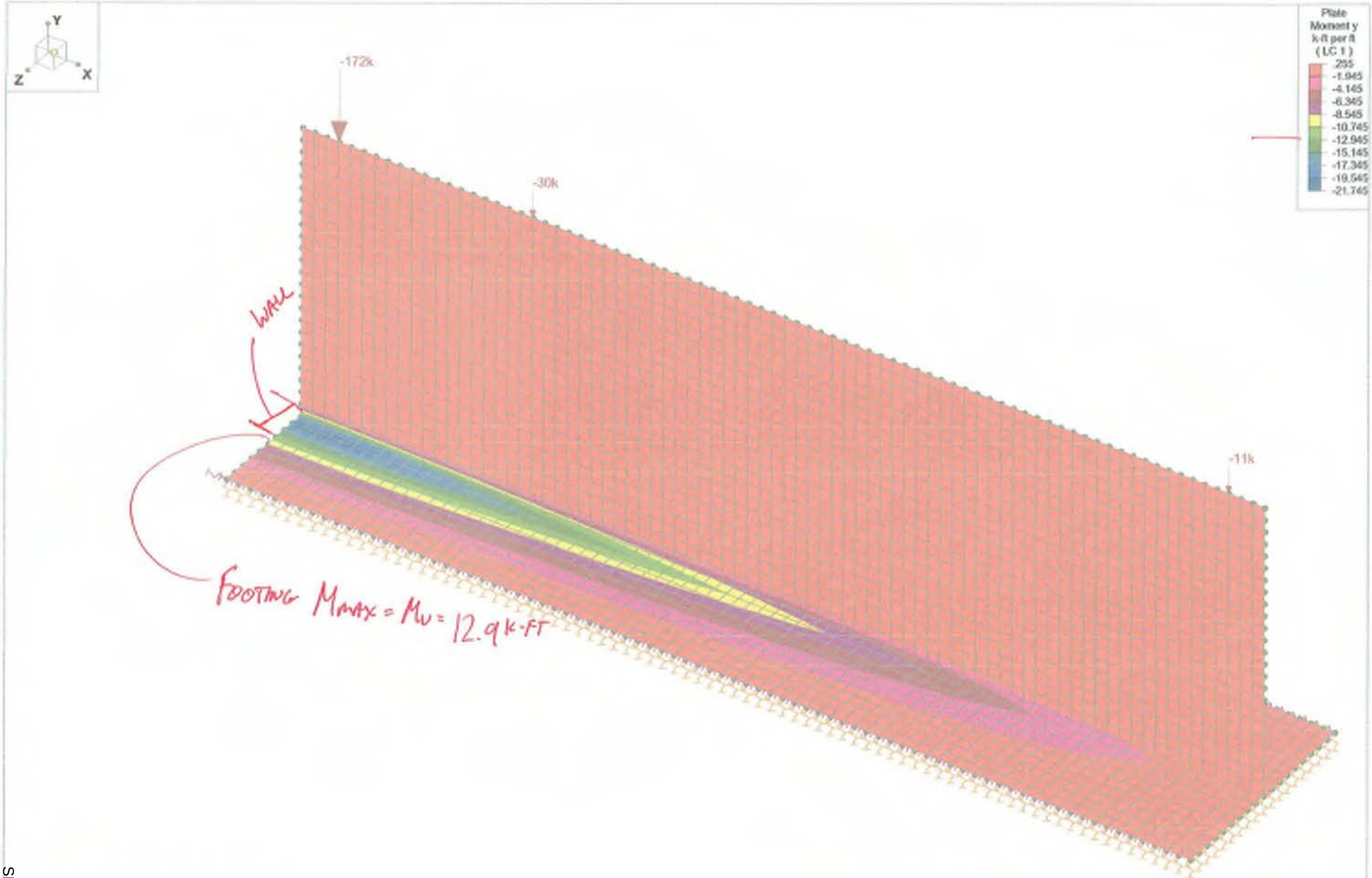
SHEET B144

Load Combinations

	Description	S...	PDelta	S...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...	B...	Fa...
1	DL + MAX COLUMN LOAD	Yes	Y		1	1	3	1																
2	DL + UPLIFT	Yes	Y		1	1	4	1																
3	DL + SURCHARGE + WIND	Yes	Y		1	1	6	1	7	1														

Basic Load Cases

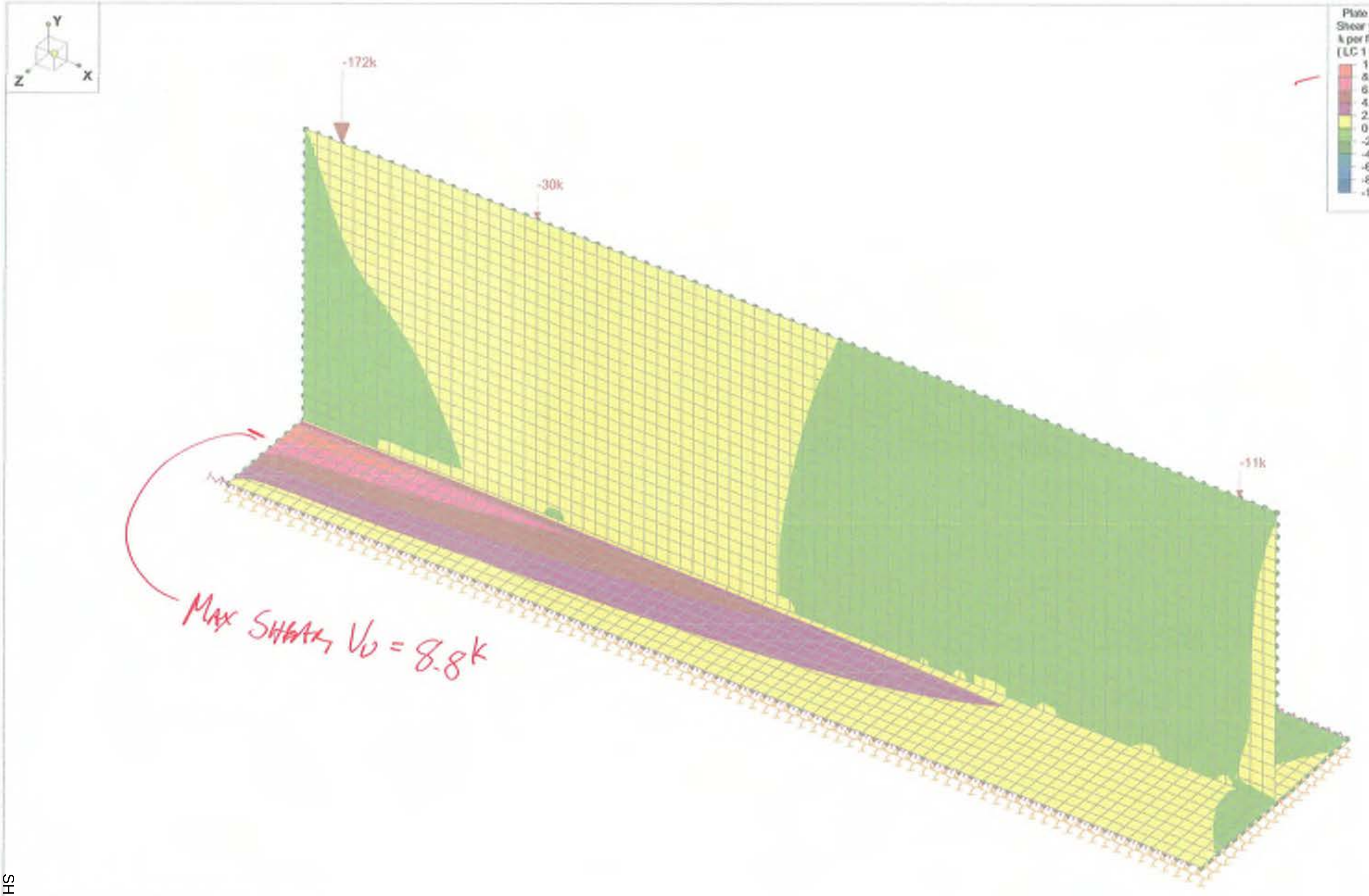
	BLC Description	Category	X Gr...	Y Gr...	Z Gra...	Joint	Point	Distri...	Area(...	Surfa...
1	SELF WEIGHT	None		-1						
3	LRFD MAX VERTICAL LOAD	None				3				
4	UPLIFT LOAD	None				3				
6	SURCHARGE	None				84				24000
7	WIND REACTION	None				4				



Loads: LC 1, DL + MAX COLUMN LOAD
Results for LC 1, DL + MAX COLUMN LOAD

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:19 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

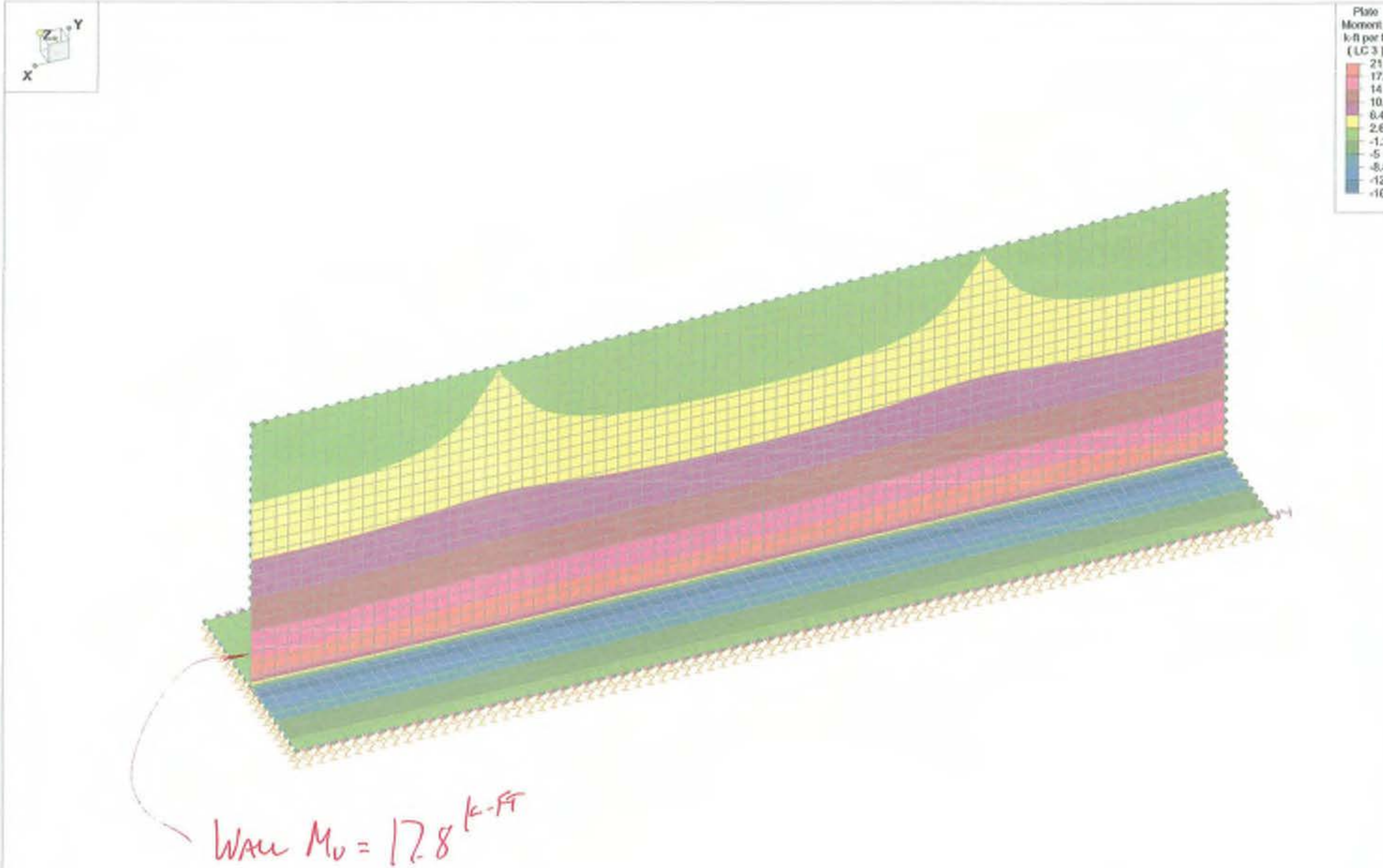
SHEET B147



Loads: LC 1, DL + MAX COLUMN LOAD
Results for LC 1, DL + MAX COLUMN LOAD

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:21 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

SHEET B148



Results for LC 3, DL + SURCHARGE + WIND

SMG ENGINEERS	AGG STORAGE FOUNDATION	SK -
BS		Sept 27, 2018 at 7:22 PM
18-183B		AGG BUILDING FOUNDATION Rev0.r3d

SHEET B149