



FASTFLOOR COMMERCIAL ARCHETYPE
DESIGN 4-STORY

**VIRGINIA POLYTECHNIC INSTITUTE
AND STATE UNIVERSITY**

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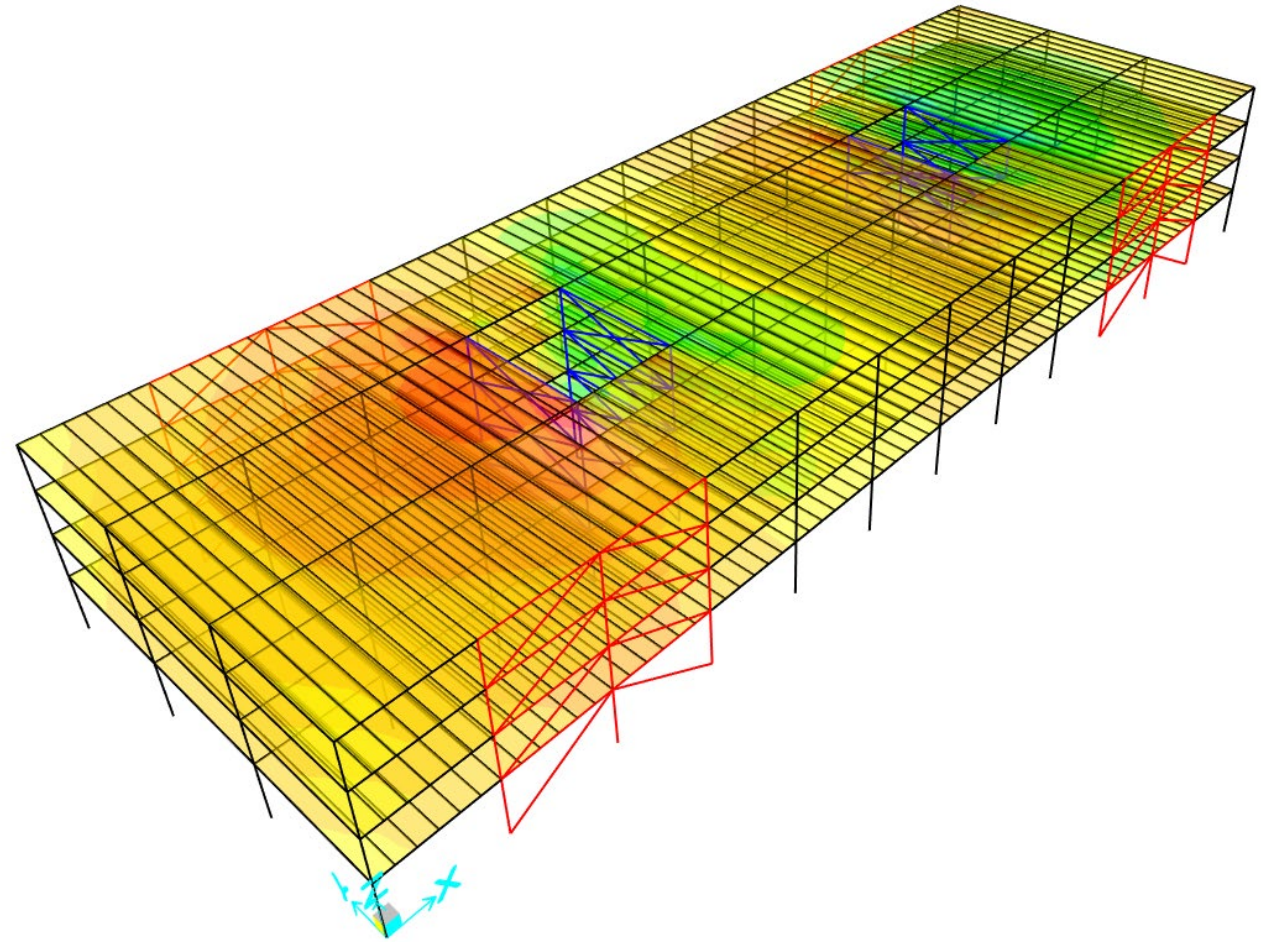
April 2026

Acknowledgements

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FastFloor Commercial Archetype Design 4-Story

Department of Civil and Systems
Engineering
Johns Hopkins University



4/6/2026



IOWA STATE
UNIVERSITY



West Virginia University



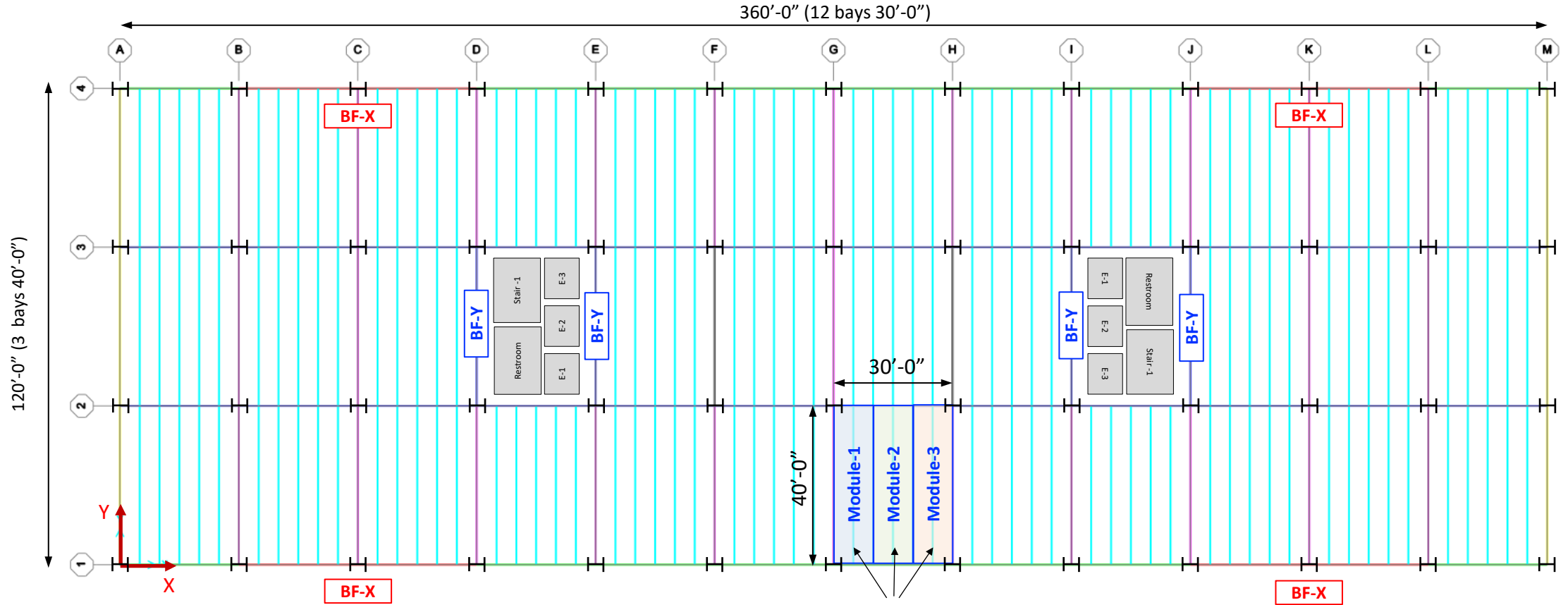
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Building Innovation through Research



Introduction

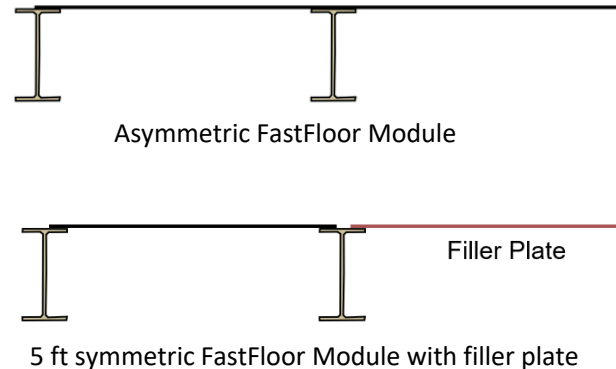
- The building archetypes focus on design aspects and performance of the FastFloor commercial system when implemented in actual buildings. It is known that the building geometry and the location of the lateral force-resisting system could substantially change the diaphragm design and behavior, which could eventually affect the overall building behavior. Further, the floor module-to-module connection could impact the cost, construction time, and diaphragm behavior, and the building archetypes provide the means to study these connections.
- These archetypes focus on diaphragm behavior and detailing. There are ongoing studies on the gravity and vibration behavior of the FastFloor modules, which may slightly change the gravity details. However, we assume the overall behavior of the diaphragms and the lateral force-resisting system would not be affected by a minor change in the module.
- The main goal of this design is to proportionate the main members of the lateral force resisting system (LFRS) and the horizontal force resisting system (HFRS).
- The design of foundation, base plates and other connections is out of scope of this document, but the demands for the connection design are provided.

Project Information (typical plan)



Building Dimensions

Width (E-W)	120.00	(ft)
Length (N-S)	360.00	(ft)
H (First story)	17.00	(ft)
H (Typical story)	13.00	(ft)
Bay size X	30.00	(ft)
Bay size Y	40.00	(ft)
Diaphragm Cantilever	90.00	(ft)
Parapet	3.00	(ft)
Number of Stories	4.00	-
h (Total height)	56	(ft)



Typical floors and roof
Fastfloor Modules
½" Steel plate on W24x94 @ 5'-0" o.c.

Floor and wall loads

Floor dead loads

Load component	Load (psf)	Reference
Steel module	30	Calculated
RAF	15	Estimated
Superimposed dead	30	Estimated
Total	75	

Roof dead loads

Load component	Load (psf)	Reference
Steel module	30	Calculated
MEP	20	Estimated
Roofing	20	Estimated
Total	70	

Floor and roof live loads

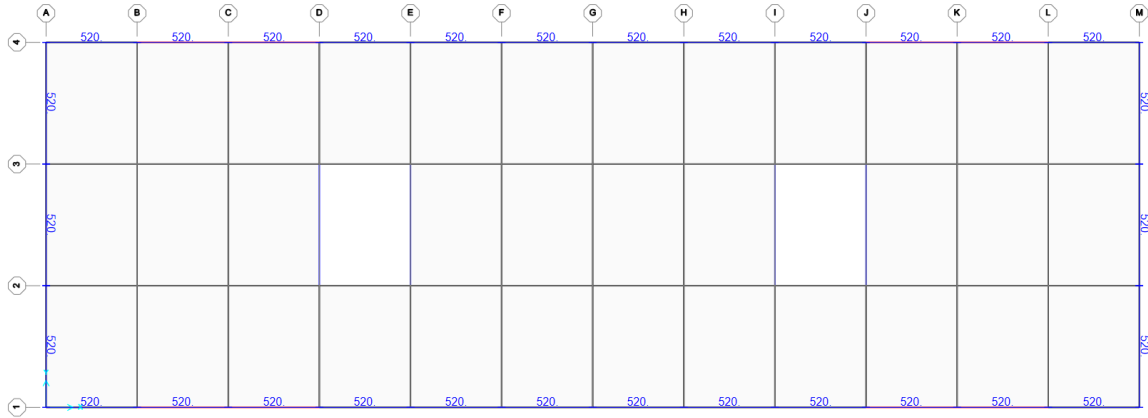
Occupancy or Use	Load (psf)	Reference
Office	50	ASCE 7-22
Partitions (only in offices)	10	ASCE 7-22
Lobbies	100	ASCE 7-22
First floor corridors	100	ASCE 7-22
Corridors above first floor	80	ASCE 7-22
Roof	20	ASCE 7-22
MEP	150	Estimated

Exterior Cladding loads

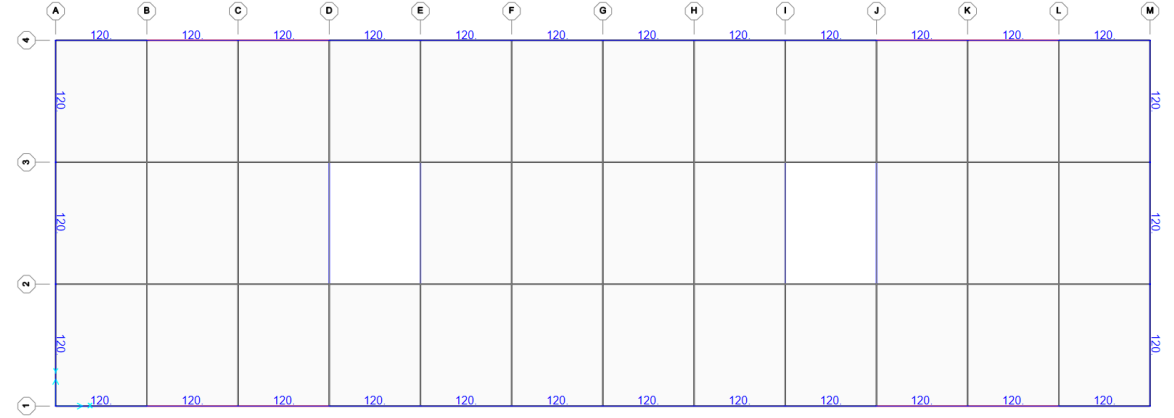
Finish	Load (psf)	Reference
Heavy (Brick or stone)	40	Calculated
Glass curtainwall	15	Estimated

*applied to vertical wall surface

Gravity Loading of the beams (line loads in the model)



Typical floors exterior wall load
(520 plf line load)



Roof parapet load
(120 plf line load)

Seismic Mass

Diaphragm/Deck

RAF weight	15	(psf)
Steel module weight	30	(psf)

Unit Weights

Dead Roof	30	(psf)	Fastfloor Modules
MEP	20	(psf)	
Roofing, mechanical	20	(psf)	(uniform on the roof)
Dead Floor	45	(psf)	
Super-imposed Dead	30	(psf)	(uniform on the floor)
External Walls	40	(psf)	(external wall unit weight)
Partitions (Int. Walls)	10	(psf)	(included in the seismic weight)

Level	Assembly	Unit Wt (psf)	Area (ft ²)	Weight (kips)	Story Wt (kips)
Roof	Roof	70	43200	3024.0	
	Ext wall (half+Parapet)	40	9120	364.8	3605
	Int. wall (half)	10	21600	216.0	
Typical floor	Floor	75	43200	3240.0	
	Ext wall	40	12480	499.2	4171
	Int. wall	10	43200	432.0	
First floor	Floor	75	43200	3240.0	
	Ext wall	40	14400	576.0	4248
	Int. wall	10	43200	432.0	
Total Mass, W					16195

Seismic Hazard



ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22

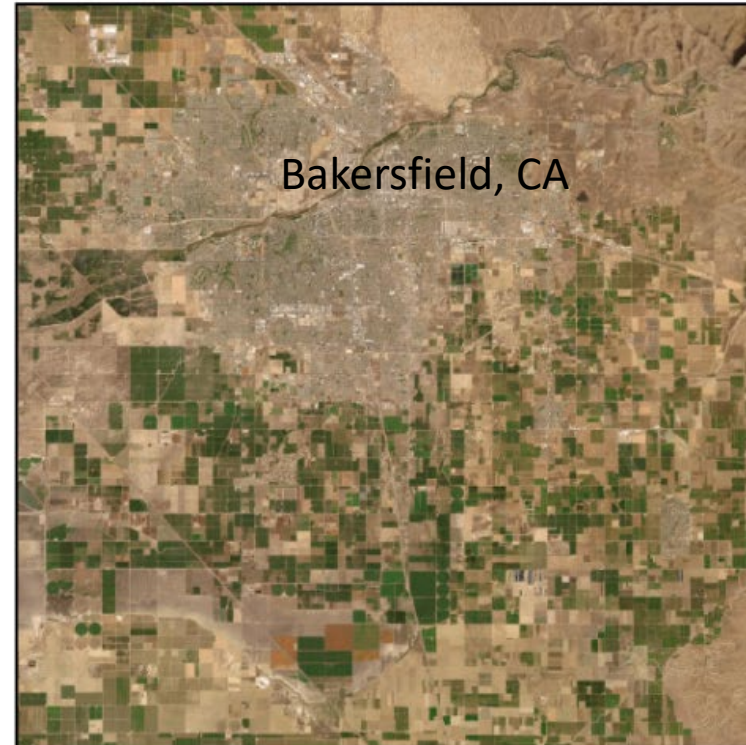
Latitude: 35.280339

Risk Category: II

Longitude: -119.016521

Soil Class: D - Stiff Soil

Elevation: 355.9212035089007 ft
(NAVD 88)



Multi-Period Spectrum



Site Soil Class:

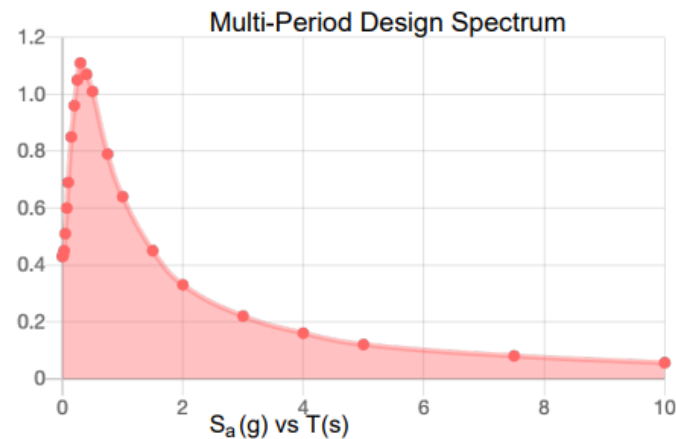
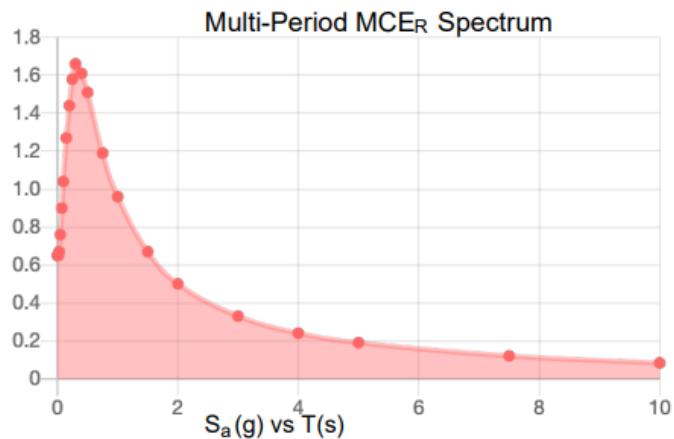
D - Stiff Soil

Results:

PGA_M : 0.53
 S_{MS} : 1.5
 S_{M1} : 0.96
 S_{DS} : 1
 S_{D1} : 0.64

T_L : 12
 S_S : 1.19
 S_1 : 0.37
 V_{S30} : 260

Seismic Design Category: D



Multi-Period Design Spectrum - ASCE7-22

T(s)	S _a (g)
0	0.43
0.01	0.43
0.02	0.44
0.03	0.45
0.05	0.51
0.075	0.6
0.1	0.69
0.15	0.85
0.2	0.96
0.25	1.05
0.3	1.11
0.4	1.07
0.5	1.01
0.75	0.79
1	0.64
1.5	0.45
2	0.33
3	0.22
4	0.16
5	0.12
7.5	0.08
10	0.056

T (sec) used in design
0.573

S_a (g)
0.946

Seismic Analysis

Seismic Analysis (LFRS) per ASCE 7-22

Site Coordinates	35.280339	-119.01652	ASCE/SE 7-22
Risk Category	II	(Table 1.5-1)	Office
$I_e =$	1.00	(Table 1.5-2)	
$S_s =$	1.190	Period of 1 Second for Site Class BC	
$S_1 =$	0.370	Period of 0.2 Second for Site Class BC	
Site Class	D	Chapter-20	
$S_{MS} = F_a S_s =$	1.5	(Eq. 11.4-1)	
$S_{M1} = F_v S_1 =$	0.96	(Eq. 11.4-2)	
$S_{DS} = 2/3 * S_{MS} =$	1.000	(Eq. 11.4-3)	
$S_{D1} = 2/3 * S_{M1} =$	0.640	(Eq. 11.4-4)	
Seismic Category	D	(Table 11.6-1 and -2)	
$T_0 =$	0.128		
$T_s =$	0.640		
$T_L =$	12	(Fig. 22-12)	

Seismic Analysis

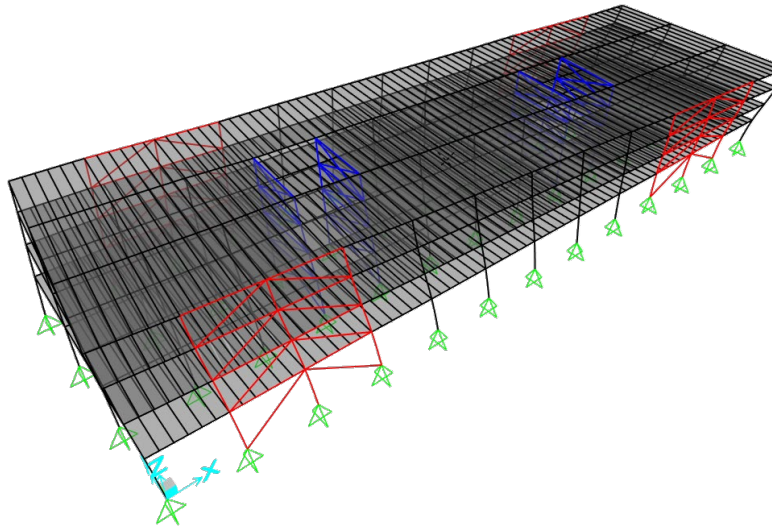
Base and Structural Level Shear, V Calculation

LFRS=

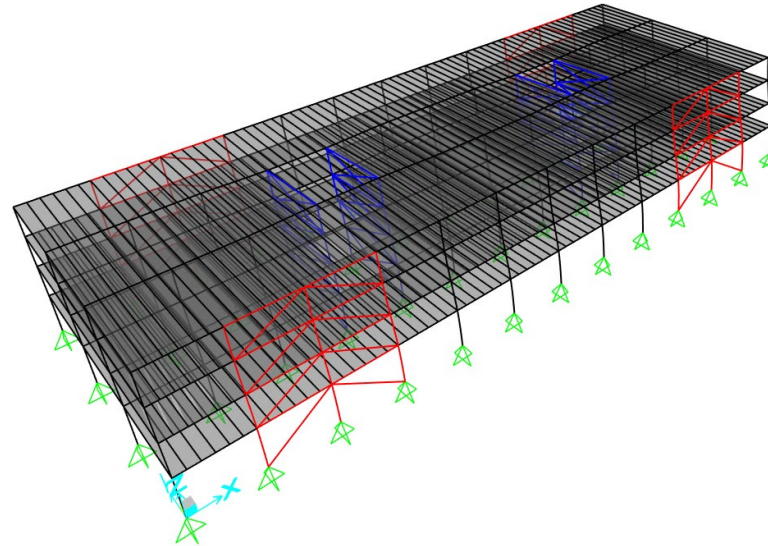
SCBF (Special Concentric Braced Frame)

R	6		
Ω_0	2		
C_d	5		
Max Ht. for SDC=D	160	ft.	
ρ	1	Diaphragm loads	
		Design of collector elements, splices, and their connections for load effects including overstrength factor otherwise (assume perimeter bracing is adequate)	
ρ	1		
$C_u =$	1.4	(Table 12.8.1)	
$C_t =$	0.02	(Table 12.8.2)	
x =	0.75	(Table 12.8.2)	
$T_a =$	0.409	(Eq. 12.8.7)	
$C_u T_a$	0.573	upper limit	
T_{model}	0.667	(least of X and Y direction)	
Using analytical period?	Yes	(Yes or No)	$C_s = \frac{S_a}{\left(\frac{R}{I_e}\right)}$
Used Period	0.573		
$C_{smax} =$	0.18609	(Eq. 12.8-3)	$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$
$C_{smin} =$	0.04400	(Eq. 12.8-6/7)	

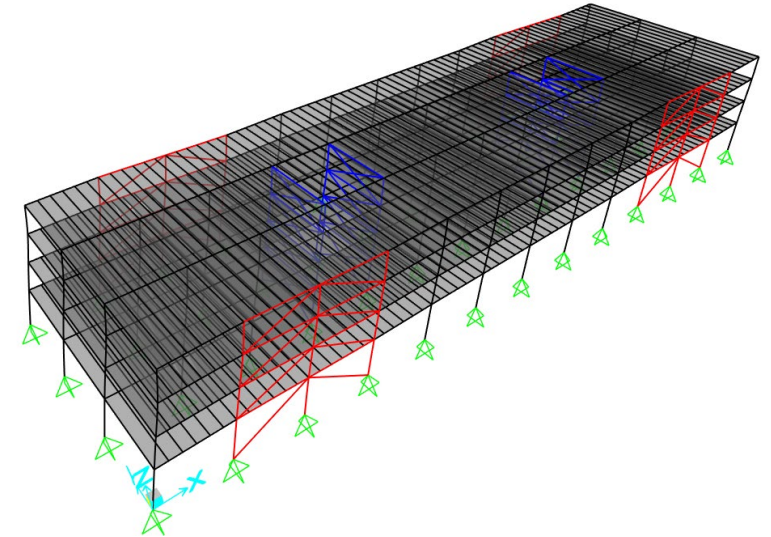
Building Analytical Periods



Mode-1: $T=0.865$ sec.
(torsional)



Mode-2: $T=0.798$ sec.
(Transverse, Y-direction)



Mode-3: $T=0.667$ sec.
(Longitudinal, X-direction)

Seismic Analysis – Base Shear and vertical distribution

$V = C_s W$ (Eq. 12.8.1) for $T \leq T_L$,
 C_s (Method 1) = **0.15760** (Eq. 12.8.2) **Used** $C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e} \right)}$ (12.8-4)
 C_s (Method 2) = **0.16667** (Eq. 12.8-3)
 $V =$ **2552** (kips) **Lower Bounds for C_s for Both Methods:** C_s shall not be less than
 $C_s = 0.044 S_{DS} I_e \geq 0.01$ (12.8-6)

$E_v/D = 0.2 S_{DS}$ **0.2**
 In addition, for structures located where S_1 is equal to or greater than 0.6, C_s shall not be less than

$C_s = 0.5 S_1 / (R/I_e)$ (12.8-7)

Vertical Distribution (12.8.3)

$k =$ **1.037** Sec-12-8-3
 Number of Stories **4**

Level	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)
Roof	3605	56.00	233908	0.354	902
4th	4171	43.00	205829	0.311	794
3rd	4171	30.00	141722	0.214	547
2nd	4248	17.00	80105	0.121	309
Total	16195		661565		

Seismic Analysis in SAP2000 v25.0.0

SAP2000 ASCE7-22 Seismic base shear is 2552.8 kip, which matches the 2552 kip base shear in the hand calculation.

S ASCE 7-22 Seismic Load Pattern

Load Direction and Diaphragm Eccentricity

Global X Direction
 Global Y Direction

Ecc. Ratio (All Diaph.)
 Override Diaph. Eccen.

Time Period

Approx. Period Ct (ft), x =
 Program Calc Ct (ft), x =
 User Defined T =

Lateral Load Elevation Range

Program Calculated
 User Specified

Max Z
 Min Z

Method for Computing Seismic Coefficients

Method 1 Method 2

Parameters - Method 1

Response Spectrum Function
 Spectral Accel, Smax
 Time Period, Tmax
 1 Sec Design Spectral Accel for Site Class B, S1

Factors

Response Modification, R
 System Overstrength, Omega
 Deflection Amplification, Cd
 Occupancy Importance, I

S Base Reactions

File View Edit Format-Filter-Sort Select Options

Units: As Noted Base Reactions

Filter:

	OutputCase	CaseType Text	GlobalFX Kip	GlobalFY Kip	GlobalFZ Kip	GlobalMX Kip-in	GlobalMY Kip-in	GlobalMZ Kip-in	GlobalX in	GlobalY in	GI
	Ex	LinStatic	-2552.795	5.145E-11	-1.473E-12	-2.78E-08	-1276279.19	1838012.682	0	0	
▶	Ey	LinStatic	6.939E-11	-2552.795	5.116E-13	1276279.186	3.3E-08	-5514038	0	0	

Record: << < 2 > >> of 2 Double-click row header to view row details.

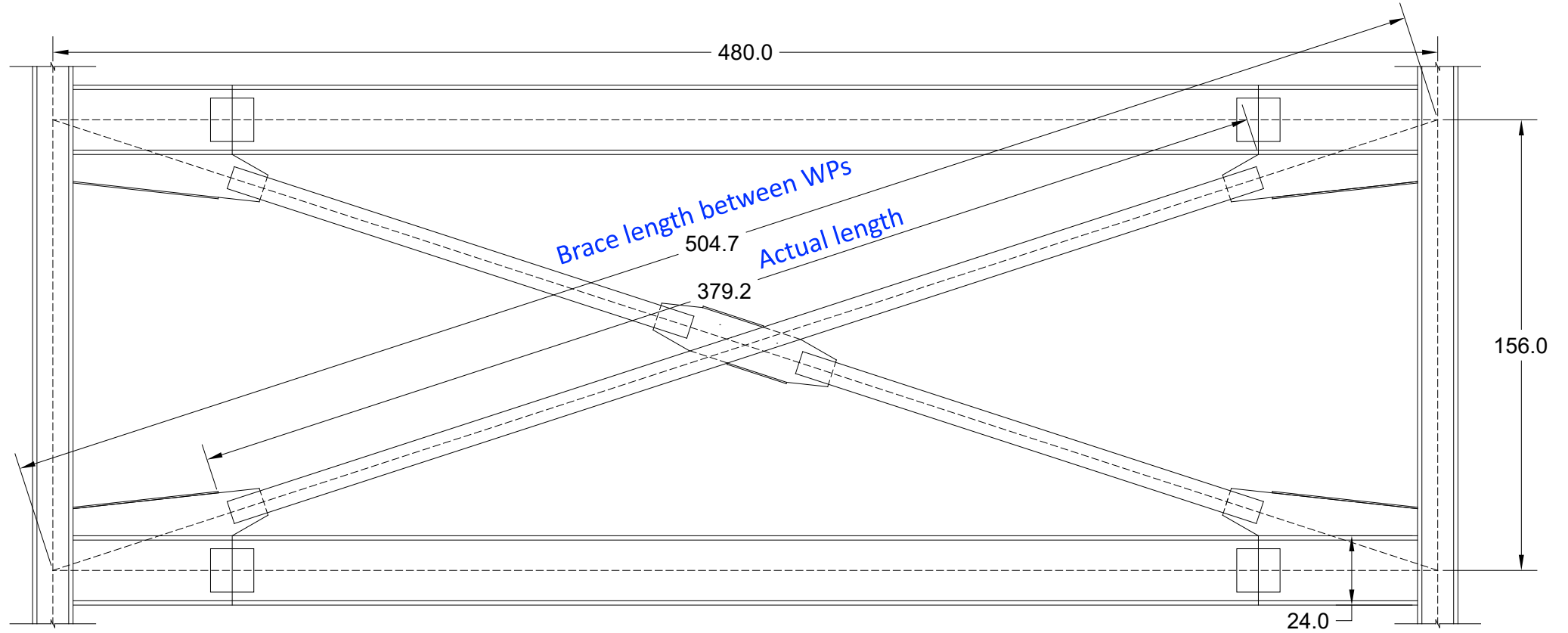
Computer Model Assumptions

- Roof and floor live load are modeled as floor live load for simplicity.
- The loading corresponding to the tributary edge of the slab has been neglected for simplicity.
- Member self-weight and mass is not calculated by the computer program for simplicity.
- It is assumed that wind loading is not governing the design.
- Braces are modeled as pin-ended frame elements, but the actual length of braces considering the gusset plates is considered in design
- Beam are models as pin-ended frame elements, and columns are continuous from base to the roof.
- Columns are modeled as pinned at their bases.
- The program considers an end offset at the end of the beams to incorporate the eccentricity of the beam reaction force on the columns.

Computer Model Assumptions

- Diaphragms are modeled as rigid constrains. For the bare deck roof, the mass of the roof is distributed over the top nodes of the columns.
- Program applies the seismic force to the center of mass. The accidental eccentricity (5%) is applied by moving the center of mass of all floors and roof: $e_x=0.05 \times 360=18'$, and $e_y=0.05 \times 120=6'$.
- P- Δ effect is not considered initially to check the stability coefficient (θ). If stability coefficient is less than 0.1, P- Δ effect is not significant.
- In order to estimate the unbraced length of the brace in the braced Frame, a preliminarily connection detailing is provided in the following slide. The unbraced length of the brace can be finalized after designing the braces and the connections, but the lengths are adequate for the design.
- In the analytical model an end offset is not considered for the braces, which is conservative for the drift results.

X-Brace Connection Configuration



Bay width of 40 ft and story height of 13 ft

Brace Length Factor in Design

Bay width ft	Story Height ft	Brace length ft	Brace length (L) in	Actual length (L _a) in	L _e /L -	Physical length factor -	K for diagonal braces -	BF-X Length Factor -	K for X-braces -	BF-Y Length Factor -
40	17	43.46	522	414	0.79	0.78	1*	0.782	0.7**	0.548
30	17	34.48	414	319	0.77	0.78	1	0.782	0.7	0.548
30	13	32.70	392	287.3	0.73	0.74	1	0.742	0.7	0.519
40	13	42.06	505	379	0.75	0.74	1	0.742	0.7	0.519

* Diagonal brace, out-of-plane buckling, K=1.0

** X-brace out-of-plane buckling, effective length factor of K=0.7 (controlled) applied to the actual length of the brace

Alternatively, for X-braces, effective length factor of K=0.5 could be used for the brace length (between the working points)

The brace length factors shows, both approaches provide similar results for the length factors, and the length factor is around 0.5.

Design Story Drifts

Ex

X-Direction	Story height (ft)	Elastic deformation (ft)	Design interstory drift (ft)	Allowable story drift (ft)	Interstory Drift Ratio (%)	Design Drift Ratio (%)	Allowable Drift Ratio (%)
Story	h_{sx}	Δ_x	Δ_M	Δ_a	θ_x	θ_M	θ_a
4	13	0.087	0.081	0.26	0.13	0.63	2.0
3	13	0.071	0.111	0.26	0.17	0.85	2.0
2	13	0.049	0.109	0.26	0.17	0.84	2.0
1	17	0.027	0.136	0.34	0.16	0.80	2.0

Ex+5%eccentricity

X-Direction	Story height (ft)	Elastic deformation (ft)	Design interstory drift (ft)	Allowable story drift (ft)	Interstory Drift Ratio (%)	Design Drift Ratio (%)	Allowable Drift Ratio (%)
Story	h_{sx}	Δ_x	Δ_M	Δ_a	θ_x	θ_M	θ_a
4	13	0.083	0.077	0.26	0.12	0.59	2.0
3	13	0.068	0.105	0.26	0.16	0.81	2.0
2	13	0.047	0.104	0.26	0.16	0.80	2.0
1	17	0.026	0.130	0.34	0.15	0.76	2.0

Ey

Y-Direction	Story height (ft)	Elastic deformation (ft)	Design interstory drift (ft)	Allowable story drift (ft)	Interstory Drift Ratio (%)	Design Drift Ratio (%)	Allowable Drift Ratio (%)
Story	h_{sx}	Δ_x	Δ_M	Δ_a	θ_x	θ_M	θ_a
4	13	0.132	0.133	0.26	0.21	1.03	2.0
3	13	0.105	0.177	0.26	0.27	1.36	2.0
2	13	0.070	0.189	0.26	0.29	1.45	2.0
1	17	0.032	0.159	0.34	0.19	0.94	2.0

Ey+5%eccentricity

Y-Direction	Story height (ft)	Elastic deformation (ft)	Design interstory drift (ft)	Allowable story drift (ft)	Interstory Drift Ratio (%)	Design Drift Ratio (%)	Allowable Drift Ratio (%)
Story	h_{sx}	Δ_x	Δ_M	Δ_a	θ_x	θ_M	θ_a
4	13	0.177	0.178	0.26	0.27	1.37	2.0
3	13	0.141	0.237	0.26	0.36	1.82	2.0
2	13	0.094	0.249	0.26	0.38	1.92	2.0
1	17	0.044	0.220	0.34	0.26	1.29	2.0

Amplification of Accidental Torsional Moment

Amplification of Accidental Torsional Moment

Story	Max. X- displacement (ft)	Avg. X- disp. (ft)	Torsional Irregularity Ratio	Torsional amplification factor	Max. Y- displacement (ft)	Avg. Y- disp. (ft)	Torsional Irregularity Ratio	Torsional amplification factor
Story	δ_{x-max}	δ_{x-avg}	TIR _x	A _x	δ_{y-max}	δ_{y-avg}	TIR _y	A _y
4	0.083	0.087	0.952	1.00	0.177	0.132	1.342	1.25
3	0.068	0.071	0.953	1.00	0.141	0.105	1.345	1.26
2	0.047	0.049	0.954	1.00	0.094	0.070	1.348	1.26
1	0.026	0.027	0.956	1.00	0.044	0.032	1.381	1.32
Average				1.00				1.27

- Amplification is not required in X direction (only 5% accidental eccentricity is considered)
- Amplification is required in Y direction (27% increase, 6.35% eccentricity is considered)

P-Delta Effects

P-Delta Effects (X-Direction)

$C_d = 5$

$l_e = 1$

	Story height (ft)	Vertical design load	Design story drift	Seismic shear force	Stability coefficient	Max. stability coefficient	Check
Story	h_{sx}	P_x	Δ_x	V_x	θ	θ_{max}	$\theta < \theta_{max}$
4	13	5500	0.081	902	0.008	0.100	Passed
3	13	12661	0.111	1697	0.013	0.100	Passed
2	13	19822	0.109	2243	0.015	0.100	Passed
1	17	27090	0.136	2552	0.017	0.100	Passed

P-Delta Effects (Y-Direction)

$C_d = 5$

$l_e = 1$

	Story height (ft)	Vertical design load	Design story drift	Seismic shear force	Stability coefficient	Max. stability coefficient	Check
Story	h_{sx}	P_x	Δ_x	V_x	θ	θ_{max}	$\theta < \theta_{max}$
4	13	5500	0.133	902	0.012	0.100	Passed
3	13	12661	0.177	1697	0.020	0.100	Passed
2	13	19822	0.189	2243	0.026	0.100	Passed
1	17	27090	0.159	2552	0.020	0.100	Passed

- P-Delta effect is negligible in both directions

Design per AISC 341-22 and AISC 360-22

Designed here:

- Gravity frames (Composite and non-composite joists, girders, and columns)
- Special Concentrically Braced Frames (SCBFs)
- Columns of the SCBFs
- Beams of the SCBFs
- Diaphragm, chord and collectors

Not designed here:

- Design for vibration
- Base plates
- Column splices
- Beam to column connections
- Brace gusset plates and pins
- Continuity plates and stiffeners
- Foundation

Member Design Assumptions in SAP2000

- All members are designed per AISC 360-22 and ASIC 341-22.
- Size of the members in the model are determined after several iteration to fulfill the following requirements:
 - *Strength*: All the members are designed to have Demand-to-capacity ratios under or close to 0.95. Different members may be designed for different load combinations.
 - *Stiffness*: All members are designed to have enough stiffness to meet the deflection requirements. The whole SFRS is designed to have lateral drift less than the lateral drift requirements in ASCE7-22.
 - *Ductility*: All members of the SFRS are designed per requirements of the AISC Seismic Provisions to ensure the building at both system level and member level (local stability) will behave as desired performance under seismic demands.

Member Design Assumptions in SAP2000

- Effective length of all column and beams are assumed to be 1.0 for both minor axis and major axis flexural buckling, $K_{\text{minor}}=K_{\text{major}}=1.0$. All beams are laterally braced at the top flange.
- Effective length of all braces are assumed to be t should be noted that the length of the braces are modified in design to account for the gusset plates.

Bay Width	Story Height	X-brace Length Factor	Y-brace Length Factor
ft	ft	-	-
40	17	0.782	0.548
30	17	0.782	0.548
30	13	0.742	0.519
40	13	0.742	0.519

- All gravity beams and girders of the floors and roof are designed as non-composite members. The elements of the Fastfloor modules are pre-defined and the adequacy is checked in the model, without changing the sizes.
- Columns and beams of the braced frames are design to the requirements of AISC Seismic Provisions. For the purpose of the seismic design, a separate analytical model has been made to find the demand and design the columns and beams of the braced frames.

Load Combinations (ASCE7-22)

$S_{DS}=1.0$
 $\rho=1$
 $\Omega=2.0$

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L$
 $(0.9 - 0.2S_{DS})D + \rho Q_E$

(Beams, columns,
and braces)

UDSTL1: 1.4D
 UDSTL2: 1.2D+1.6L
 UDSTL3: 1.40D+0.5L+ E_x
 UDSTL4: 1.40D+0.5L- E_x
 UDSTL5: 1.40D+0.5L+ E_y
 UDSTL6: 1.40D+0.5L- E_y
 UDSTL7: 1.40D+0.5L+ $E_{x+5\%}$
 UDSTL8: 1.40D+0.5L- $E_{x+5\%}$
 UDSTL9: 1.40D+0.5L+ $E_{x-5\%}$
 UDSTL10: 1.40D+0.5L- $E_{x-5\%}$
 UDSTL11: 1.40D+0.5L+ $E_{y+5\%}$
 UDSTL12: 1.40D+0.5L- $E_{y+5\%}$
 UDSTL13: 1.40D+0.5L+ $E_{y-5\%}$
 UDSTL14: 1.40D+0.5L- $E_{y-5\%}$
 UDSTL15: 0.70D+ E_x
 UDSTL16: 0.70D- E_x
 UDSTL17: 0.70D+ E_y
 UDSTL18: 0.70D- E_y
 UDSTL19: 0.70D+ $E_{x+5\%}$
 UDSTL20: 0.70D- $E_{x+5\%}$
 UDSTL21: 0.70D+ $E_{x-5\%}$
 UDSTL22: 0.70D- $E_{x-5\%}$
 UDSTL23: 0.70D+ $E_{y+5\%}$
 UDSTL24: 0.70D- $E_{y+5\%}$
 UDSTL25: 0.70D+ $E_{y-5\%}$
 UDSTL26: 0.70D- $E_{y-5\%}$

$(1.2 + 0.2S_{DS})D + \Omega Q_E + 0.5L$
 $(0.9 - 0.2S_{DS})D + \Omega Q_E$

(Amplified load combinations for
bracing columns and collectors)

AUDSTL3: 1.40D+0.5L+2.0 E_x
 AUDSTL4: 1.40D+0.5L-2.0 E_x
 AUDSTL5: 1.40D+0.5L+2.0 E_y
 AUDSTL6: 1.40D+0.5L-2.0 E_y
 AUDSTL7: 1.40D+0.5L+2.0 $E_{x+5\%}$
 AUDSTL8: 1.40D+0.5L-2.0 $E_{x+5\%}$
 AUDSTL9: 1.40D+0.5L+2.0 $E_{x-5\%}$
 AUDSTL10: 1.40D+0.5L-2.0 $E_{x-5\%}$
 AUDSTL11: 1.40D+0.5L+2.0 $E_{y+5\%}$
 AUDSTL12: 1.40D+0.5L-2.0 $E_{y+5\%}$
 AUDSTL13: 1.40D+0.5L+2.0 $E_{y-5\%}$
 AUDSTL14: 1.40D+0.5L-2.0 $E_{y-5\%}$
 AUDSTL15: 0.70D+2.0 E_x
 AUDSTL16: 0.70D-2.0 E_x
 AUDSTL17: 0.70D+2.0 E_y
 AUDSTL18: 0.70D-2.0 E_y
 AUDSTL19: 0.70D+2.0 $E_{x+5\%}$
 AUDSTL20: 0.70D-2.0 $E_{x+5\%}$
 AUDSTL21: 0.70D+2.0 $E_{x-5\%}$
 AUDSTL22: 0.70D-2.0 $E_{x-5\%}$
 AUDSTL23: 0.70D+2.0 $E_{y+5\%}$
 AUDSTL24: 0.70D-2.0 $E_{y+5\%}$
 AUDSTL25: 0.70D+2.0 $E_{y-5\%}$
 AUDSTL26: 0.70D-2.0 $E_{y-5\%}$

AISC 341-22 general provisions (summary of the applicable provisions)

Section B5.1

Diaphragms and chords shall be designed for the loads and load combinations in the applicable building code. Collectors shall be designed for the load combinations in the applicable building code, including overstrength.

Section D1.4a. Required Strength

- The required strength of columns in the SFRS shall be determined from the greater of the capacity-limited seismic load effect and the the compressive axial strength and tensile strength as determined using the overstrength seismic load.

Section D1.5 Composite Slab Diaphragms (applicable to the connection design)

- Details shall be provided to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizontal framing system.

AISC 341-22 requirements for SCBFs (summary of the applicable provisions)

Section F2.1

- Special concentrically braced frames (SCBF) of structural steel shall be designed in conformance with this section.

Section F2.2

- SCBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

Section F2.3

- The capacity-limited horizontal seismic load effect, E_{cl} , shall be taken as the larger force determined from the following analyses:
 - (a) An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension
 - (b) An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength.
- The expected brace strength in tension is $R_y F_y A_g$, where A_g is the gross area.

AISC 341-22 requirements for SCBFs (summary of the applicable provisions)

- Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.
- The expected brace strength in compression is permitted to be taken as the lesser of $R_y F_y A_g$ and $(1/0.877) F_{cre} A_g$, where F_{cre} is determined from *Specification* Chapter E using the equations for F_{cr} , except that the expected yield stress, $R_y F_y$, is used in lieu of F_y .
- The brace length used for the determination of F_{cre} shall not exceed the distance from brace end to brace end.
- The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression.

Section F2.4

- Along any line of braces, braces shall be deployed in alternate directions such that, for either direction of force parallel to the braces, at least 30% but no more than 70% of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the overstrength seismic load.

AISC 341-22 requirements for SCBFs (summary of the applicable provisions)

Section F2.5a

- Columns, beams and braces shall satisfy the requirements of Section D1.1 for highly ductile members.

Section F2.5b

Braces shall have a slenderness ratio of $L_c/r \leq 200$,

Where, L_c = effective length of brace = KL , r = governing radius of gyration

Section F2.5c

The protected zone of SCBF shall satisfy Section D1.3, and shall include the following:

- (a) For braces, the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling
- (b) Elements that connect braces to beams and columns


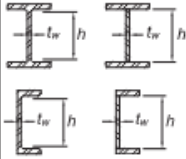
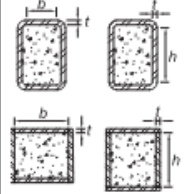

Material properties

Material	Designation
Columns and beams	ASTM A992 ($F_y / F_u = 50/65$ ksi, $E=29000$ ksi)
Braces (HSS)	ASTM A1085 ($F_y / F_u = 50/65$ ksi, $E=29000$ ksi) CVN=25 ft.-lbs. @ 40° F
Diaphragm (FastFloor)	ASTM A529 ($F_y / F_u = 50/65$ ksi, $E=29000$ ksi)

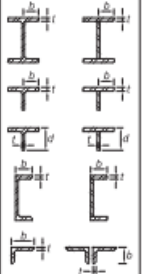
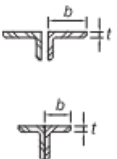
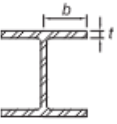
Width-to-thickness ratio per Table D1.1

Width-to-thickness ratio of all Beams and columns of SCBF system shall satisfy the requirements of Section D1.1 for Highly ductile members.

Flanges of beam and columns are checked as “Unstiffened Elements” and webs are “Stiffened Elements”. Beams are under axial force (P_u) projected from the braces and h/t_w limiting ratio is determined based on $C_a = P_u / (\phi_c R_y F_y A_g)$

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Example
			λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members	
3	Walls of round HSS ^(a)	D/t	$0.053 \frac{E}{R_y F_y}$	$0.062 \frac{E}{R_y F_y}$	
4	Webs of rolled or built-up I-shaped sections and channels	h/t_w	$1.49 \sqrt{\frac{E}{R_y F_y}}$	$1.49 \sqrt{\frac{E}{R_y F_y}}$	
5	Walls of filled rectangular HSS and box sections ^(a)	b/t h/t	$1.4 \sqrt{\frac{E}{R_y F_y}}$	$2.26 \sqrt{\frac{E}{R_y F_y}}$	
6	Walls of filled round HSS sections ^(a)	D/t	$0.076 \frac{E}{R_y F_y}$	$0.15 \frac{E}{R_y F_y}$	

^(a) The design wall thickness shall be used in the calculations involving the wall thickness of hollow structural sections (HSS), as defined in *Specification* Section B4.2.

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Example
			λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members	
7	(1) Flanges of rolled or built-up I-shaped sections	b/t d/t	$0.30 \sqrt{\frac{E}{R_y F_y}}$	$0.38 \sqrt{\frac{E}{R_y F_y}}$	
	(2) Flange and stem of rolled or built-up tees				
	(3) Flanges of rolled or built-up channels				
	(4) Legs of single angles or double-angle members with separators				
	(5) Outstanding legs of pairs of angles in continuous contact				
8	Horizontal legs of double-angle members with separators or in continuous contact	b/t	$0.47 \sqrt{\frac{E}{R_y F_y}}$	$0.54 \sqrt{\frac{E}{R_y F_y}}$	
9	Flanges of H-pile sections per Section D4	b/t	not applicable	$0.45 \sqrt{\frac{E}{R_y F_y}}$	

Width-to-thickness ratio of SCBF columns

Table D1.1 limiting ratios								Member check			
Story	Braced frame	Cross-section	F_y	E	R_y	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
3,4	X- Direction	W14x68	50	29000	1.1	36.05	7.35	27.50	Passed	7.0	Passed
1,2	X- Direction	W14x159	50	29000	1.1	36.05	7.35	15.30	Passed	6.5	Passed
3,4	X-Direction (Zipper)	W14x82	50	29000	1.1	36.05	7.35	22.40	Passed	5.9	Passed
1,2	X-Direction (Zipper)	W14x193	50	29000	1.1	36.05	7.35	12.80	Passed	5.5	Passed
3,4	Y-Direction	W14x82	50	29000	1.1	36.05	7.35	22.40	Passed	5.9	Passed
1,2	Y-Direction	W14x211	50	29000	1.1	36.05	7.35	11.60	Passed	5.1	Passed

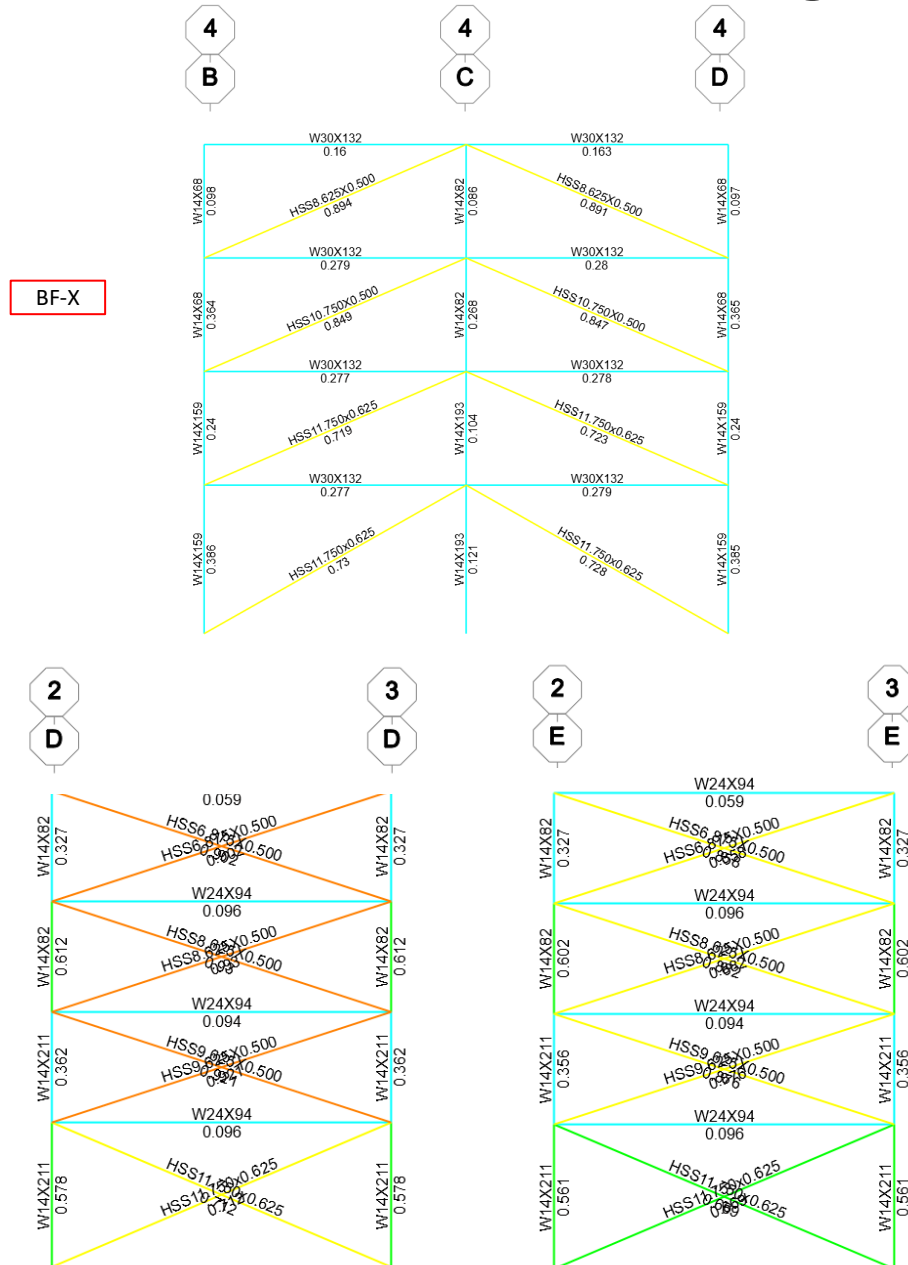
Width-to-thickness ratio of SCBF beams

Story	Braced frame	Cross-section	F_y (ksi)	E (ksi)	R_y	P_u^* (kips)	C_a	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
4	Bay of X- Direction	W30x132	50	29000	1.1	52	0.03	57.35	7.35	43.9	Passed	5.27	Passed
3		W30x132	50	29000	1.1	307	0.16	50.92	7.35	43.9	Passed	5.27	Passed
2		W30x132	50	29000	1.1	322	0.17	50.77	7.35	43.9	Passed	5.27	Passed
1		W30x132	50	29000	1.1	399	0.21	49.96	7.35	43.9	Passed	5.27	Passed
4	Bay of Y- Direction	W24x94	50	29000	1.1	193	0.14	51.31	7.35	41.9	Passed	5.18	Passed
3		W24x94	50	29000	1.1	400	0.29	48.26	7.35	41.9	Passed	5.18	Passed
2		W24x94	50	29000	1.1	413	0.3	48.07	7.35	41.9	Passed	5.18	Passed
1		W24x94	50	29000	1.1	491	0.36	46.92	7.35	41.9	Passed	5.18	Passed

* Force associated with the strength of the SCBFs. Source: Ultimate models

** Designed for Highly Ductile Members

SCBF Design (DCR ratios)



Story	Brace -X	Brace-Y
	HSS	HSS
4	8.625x0.500	6.875x0.500
3	10.750x0.500	8.625x0.500
2	11.750x0.625	9.625x0.500
1	11.750x0.625	11.750x0.625

SCBF Design load combination

$$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L$$

$$(0.9 - 0.2S_{DS})D + \rho Q_E$$

Design for Brace Strength

Using thermal deformation to make tension and compression in the braces.

$$E=1 \text{ ksi}$$

$$\alpha=1/^\circ\text{F}$$

$$\Delta T = 0.3(1/0.877) F_{cre}$$

(Compression)

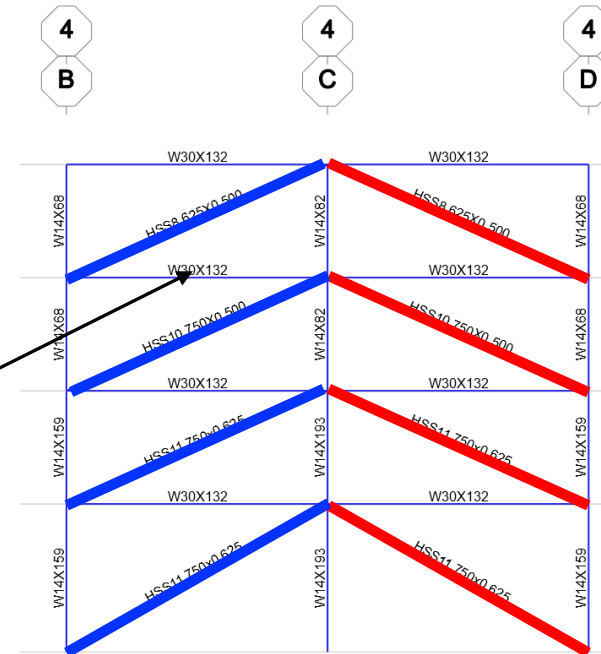
$$\Delta T = R_y F_y = -62.5 \text{ ksi}$$

(Tension)

The force will be combined with dead and live loads as follows:

$$(1.2D+0.2S_{ds})+0.5L+\text{brace_force}$$

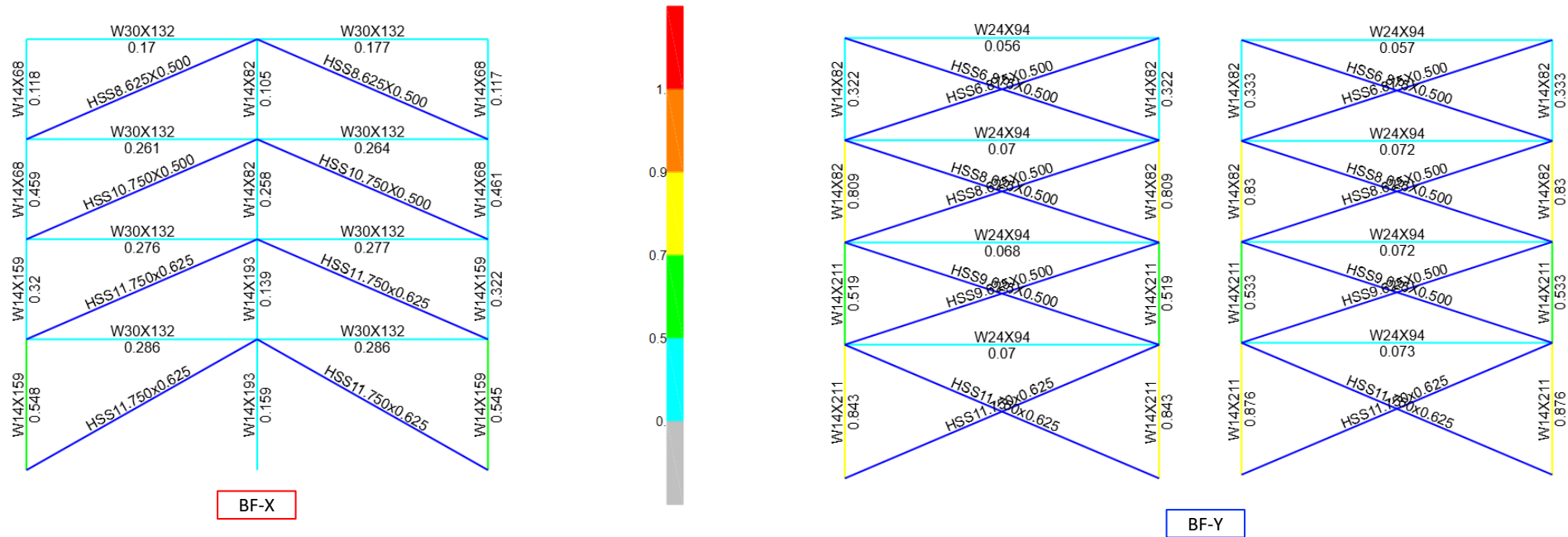
Longitudinal spring along the beams,
K=1000 kip/ft



Brace Strength in Tension and Compression

Level	Direction	Type	Manual Label	F_y	R_y	D/t limit	D/t	Compactness	KL/r	F_e	F_y/F_e	F_{cre}	P_n	$(1/0.877)F_{cre}$	$0.3(1/0.877)F_{cre}$	$R_y F_y$
				ksi						ksi		ksi	kip	ksi	ksi	ksi
1	X	HSS	HSS11.750X0.625	50	1.25	24.6	20.2	Compact	82	42.8	1.46	33.9	692	38.7	11.6	62.5
2	X	HSS	HSS11.750X0.625	50	1.25	24.6	20.2	Compact	74	52.8	1.18	38.1	777	43.4	13.0	62.5
3	X	HSS	HSS10.750X0.500	50	1.25	24.6	23.1	Compact	80	44.8	1.39	34.9	523	39.8	11.9	62.5
4	X	HSS	HSS8.625X0.500	50	1.25	24.6	18.5	Compact	101	28.3	2.21	24.8	295	28.2	8.5	62.5
1	Y	HSS	HSS11.750X0.625	50	1.25	24.6	20.2	Compact	72	55.0	1.14	38.8	792	44.3	13.3	62.5
2	Y	HSS	HSS9.625X0.500	50	1.25	24.6	20.7	Compact	81	43.7	1.43	34.3	460	39.2	11.7	62.5
3	Y	HSS	HSS8.625X0.500	50	1.25	24.6	18.5	Compact	91	34.7	1.80	29.4	350	33.6	10.1	62.5
4	Y	HSS	HSS6.625X0.500	50	1.25	24.6	14.2	Compact	120	19.8	3.16	17.3	156	19.8	5.9	62.5

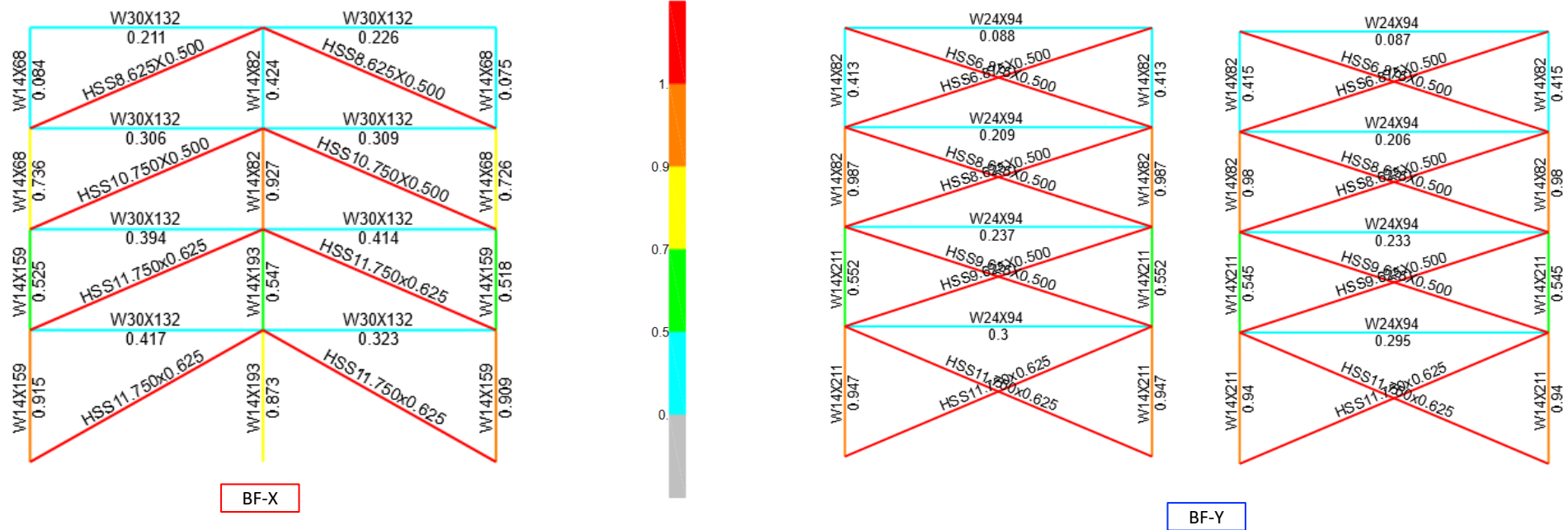
Bracing Column and Beam Design (DCR ratios)



amplified load combinations

Bracing frame beams columns are designed for the maximum of the adjusted strength of the SCBFs and the amplified load combinations. (AISC 341-22: Chapter D.4a)

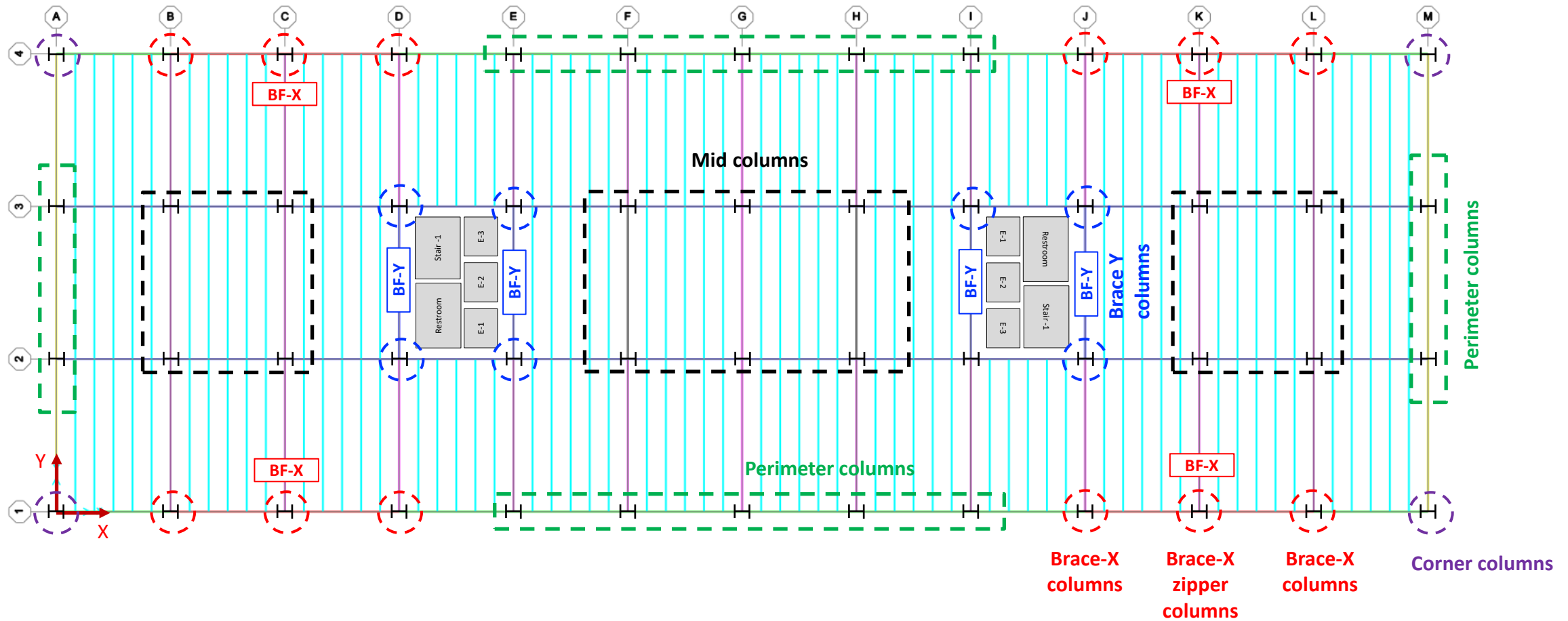
Bracing Column and Beam Design (DCR ratios)



Strength of SCBs

Bracing frame beams columns are designed for the maximum of the adjusted strength of the SCBFs and the amplified load combinations. (AISC 341-22: Chapter D.4a)

Column Design Summary



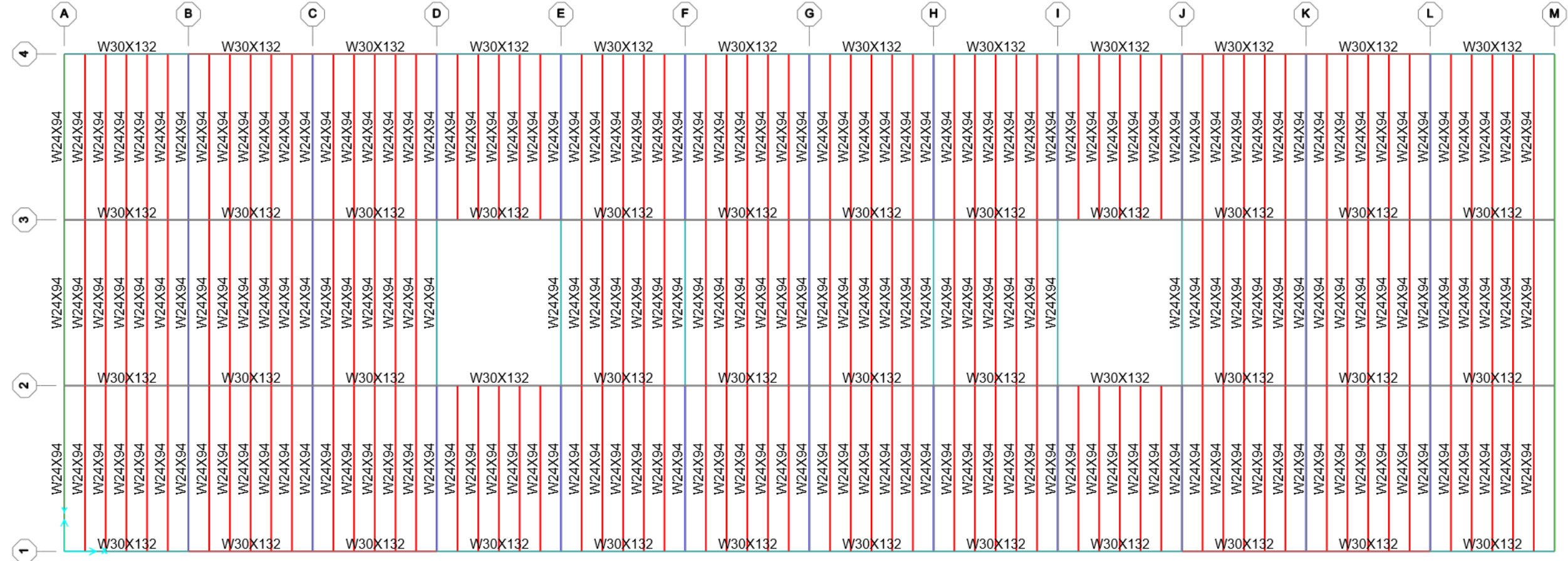
Story	Perimeter column	Mid column	Corner Columns	Brace-X column	Brace-X Zipper	Brace-Y column
4						
3	W14x38	W14x53	W14x30	W14x68	W14x82	W14x82
2						
1	W14x68	W14x90	W14x53	W14x159	W14x193	W14x211

Column Design Summary

Notes:

- Columns are spliced every other stories. Accordingly, column size kept unchanged for two consequent stories.
- Perimeter columns, Mid columns, and Corner columns are mostly gravity columns.
- Brace-X, Brace-X-Zipper, and Brace-Y columns are seismic columns and designed for the greater of the load effect resulting from adjusted brace forces and the compressive axial strength and tensile strength as determined using the overstrength seismic load.

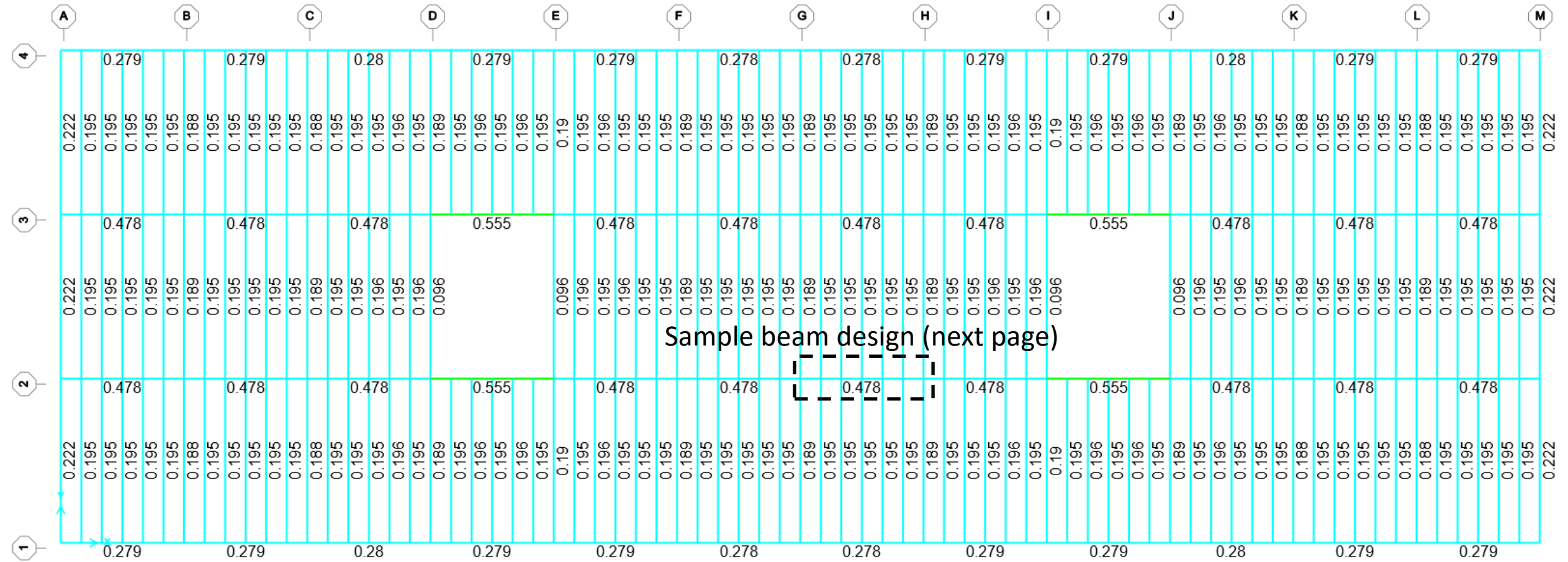
Beam (joists) and Girder design Summary



All beams and joists are taken Fastfloor Commercial modules.

*Exterior beams (joists) and girders may subject to change after designing chords and collectors.

Beam (joists) and Girder design Summary (DCRs)



All beams and joists are taken Fastfloor Commercial modules. The beam/joist sizes are controlled by vibration design and DCRs are well under 1.0 for all floor elements.

Beam and joist deflection limits for Dead+Live and Live loads are L/240 and L/360, respectively.

*Exterior beams (joists) and girders may subject to change after designing chords and collectors.

Beam (joists) and Girder design Summary (SAP2000 Design)

STRESS CHECK FORCES & MOMENTS (Combo UDFL2)

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
15.	0.234	783.09	-0.007	-18.599	-7.703E-05	-1.127E-04

PMM DEMAND/CAPACITY RATIO (H1.2,H1-1b)

D/C Ratio: 0.478 = 0. + 0.478 + 0.
 = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

COMPACTNESS

Slenderness	Lambda	Lambda_p	Lambda_r	Lambda_s	Compactness
Major/Flange	5.25	9.152	24.083		Compact
/Web	43.902	90.553	137.274	232.	Compact
Minor/Flange	5.25	9.152	24.083		Compact
/Web	43.902	90.553	137.274	232.	Compact
Axial/Flange	5.25		13.487		Compact
/Web	43.902		35.884		Slender

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1.2,H1-1b)

Factor	L	K1	K2	B1	B2	Cm
Major Bending	0.961	1.	1.	1.	1.	1.
Minor Bending	6.243E-09	1.	1.	1.	1.	1.

	Lltb	Kltb	Cb
LTB	6.243E-09	1.	1.

	Pu	phi*Pnc	phi*Pnt
Axial	Force	Capacity	Capacity
	0.234	1563.568	1746.

	Mu	phi*Mn	phi*Mn	phi*Mn
Major Moment	Moment	Capacity	No LTB	Cb=1
	783.09	1638.75	1638.75	1638.75
Minor Moment	-0.007	219.		

SHEAR CHECK

	Vu	phi*Vn	Stress	Status
Major Shear	Force	Capacity	Ratio	Check
	18.599	559.035	0.033	OK
Minor Shear	7.703E-05	567.	0.	OK

DEFLECTION CHECK (ENVELOPE)

Deflection	Load	Station	Deflection	Deflection	Deflection	Check
Type	Combo	Location	Value	Limit	Ratio	Status
Dead Load	D-1	15.	0.023	0.25	0.092	OK
Super DL+LL	D-1	15.	0.018	0.25	0.074	OK
Live Load	D-1	15.	0.018	0.125	0.147	OK
Total Load	D-1	15.	0.041	0.083	0.498	OK
Total-Camber	D-1	15.	0.041	0.125	0.332	OK

Sample beam design in
SAP2000
Beam size W30x132

Beam and joist deflection
limits for Dead+Live and Live
loads are L/240 and L/360,
respectively.

Diaphragm Design

Diaphragms are design for the following demands:

- 1) ASCE 7-22, $R_s=1.0$
- 2) ASCE 7-22 Standard Method

Following parts of the diaphragms are designed:

- Shear design
- Collector design
- Chord design

Diaphragm Flexibility

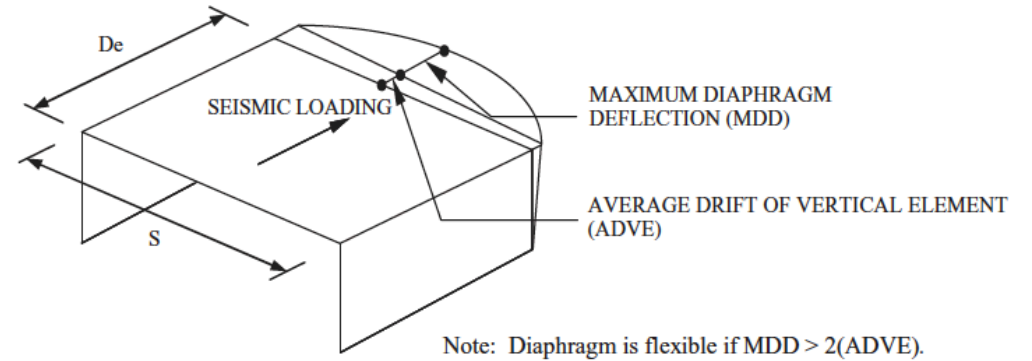
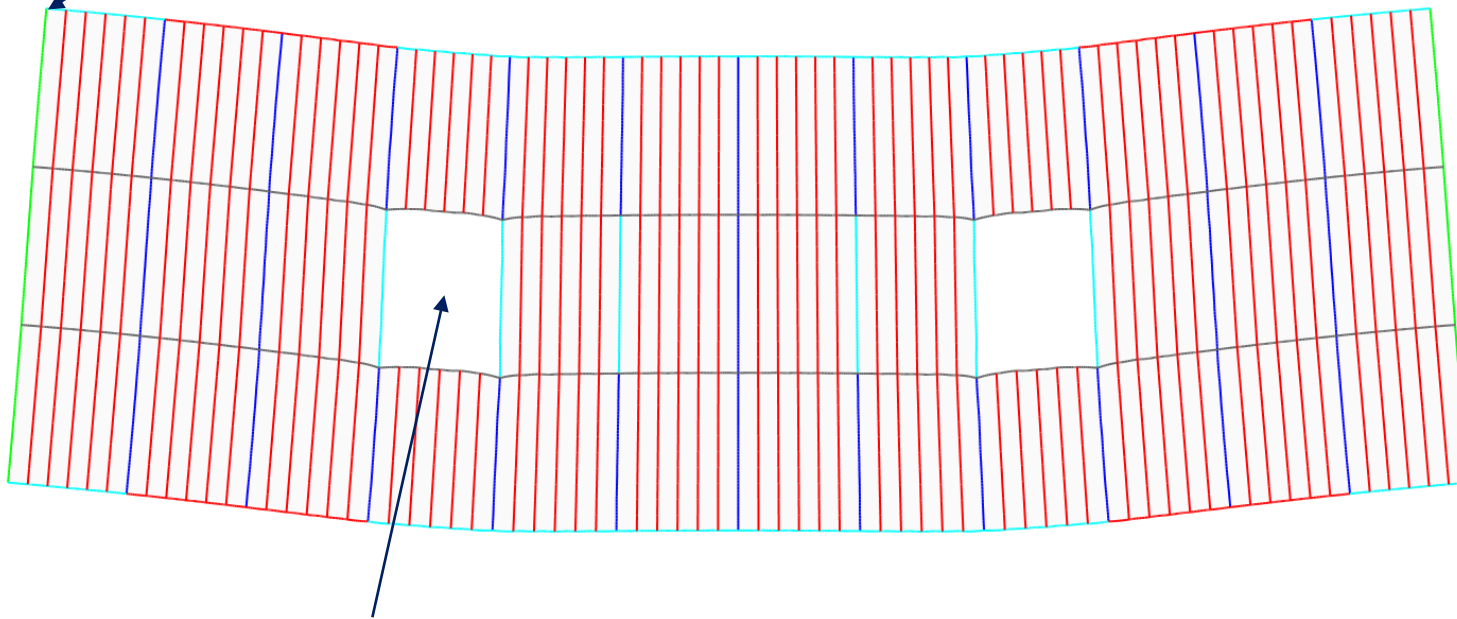


FIGURE 12.3-1 Flexible Diaphragm

For roof level, Maximum Diaphragm Deflection (MDD) = 0.058



For roof level, Average drift of vertical element (ADVE) = 0.312 in (elastic)

Diaphragm Flexibility

Diaphragm Stiffness

Level	F_v (kips)	δ_{MDD} (in)	Δ_{ADVF} (in)	$\delta_{MDD}/\Delta_{ADVF}$		
Roof	902	0.058	0.312	0.186	<2	not Flexible
4th	794	0.050	0.409	0.123	<2	not Flexible
3rd	547	0.034	0.442	0.077	<2	not Flexible
2nd	309	0.023	0.360	0.063	<2	not Flexible
Total	2552					

Diaphragm Stiffness - No LFRS in X direction

Level	F_v (kips)	δ_{MDD} (in)	Δ_{ADVF} (in)	$\delta_{MDD}/\Delta_{ADVF}$		
Roof	902	0.058	0.312	0.186	<2	not Flexible
4th	794	0.053	0.409	0.130	<2	not Flexible
3rd	547	0.035	0.442	0.080	<2	not Flexible
2nd	309	0.025	0.360	0.070	<2	not Flexible
Total	2552					

Change in diaphragm deformationms due to removing LFRS in X-direction

Level	δ_{MDD} (in)
Roof	1.00
4th	1.06
3rd	1.03
2nd	1.12
Total	

This comparison shows that the LFRS in the perpendicular direction (X-Braces in this archetype) could add additional stiffness to the diaphragm. This effect is estimated to be between 0% to 12%, in this archetype, and it will be conservative to ignore this effect. (See slide 72 for more details)

Diaphragms are not flexible per ASCE7-22. However, definition of Rigid diaphragm in ASCE 7 is prescriptive and only includes concrete slabs or concrete-filled metal decks with certain conditions provide in 12.3.1.2

Seismic Analysis – Diaphragm Force

Diaphragm Force, ASCE7-22 Alternative, $R_s=1$

R_s	1.00	elastic		
I_e	1.00		$C_{p0} = 0.4S_{DS}I_e$	(12.10-6)
N	4.00		$C_{pi} = \sqrt{(\Gamma_{m1}\Omega_0C_s)^2 + (\Gamma_{m2}C_{s2})^2} \geq C_{pi}$	(12.10-7)
S_{Ds}	1.000		Design acceleration coefficient, C_{pi} , shall be the greater of the values given by Equations (12.10-8) and (12.10-9):	
S_{D1}	0.640		$C_{pi} = 0.8C_{p0}$	(12.10-8)
C_{p0}	0.4		$C_{pi} = 0.9\Gamma_{m1}\Omega_0C_s$	(12.10-9)
C_{pi}	0.41		$C_{s2} = (0.15N + 0.25)I_eS_{DS}$	(12.10-10)
C_{pn}	0.63		$C_{s2} = I_eS_{DS}$	(12.10-11)
Γ_{m1}	1.375		For $N \geq 2$ $C_{s2} = \frac{I_eS_{D1}}{0.03(N-1)}$	(12.10-12a)
Γ_{m2}	0.506		For $N = 1$ $C_{s2} = 0$	(12.10-12b)
C_s	0.167		$\Gamma_{m1} = 1 + \frac{z_s}{2} \left(1 - \frac{1}{N}\right)$	(12.10-13)
z_s	1.0	for "SCBF"	$\Gamma_{m2} = 0.9z_s \left(1 - \frac{1}{N}\right)^2$	(12.10-14)
C_{s2}	0.85			
C_{s2_1}	0.85			
C_{s2_2}	1.00			
C_{s2_3}	7.11	$N \geq 2$	$F_{px} = 0.2S_{DS}I_eW_{px}$	(12.10-5)
C_{s2_4}	0	$N = 1$		
Collector design factor	1.5		$F_{px} = \frac{C_{px}}{R_s}W_{px}$	(12.10-4)

Seismic Analysis – Diaphragm Force

Diaphragm Force, ASCE7-22 Alternative, $R_s=1$

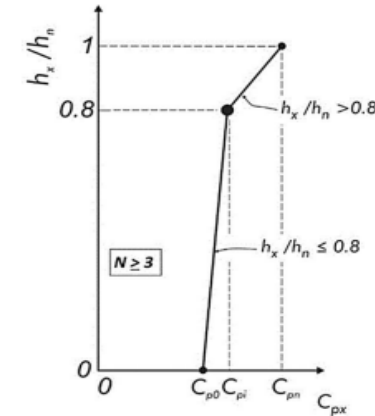
Level	F_i (k)	W_i (k)	h_x/h_n (ft)	C_{Px}	F_{Px} (k)	F_{P-min} (k)	F_P (k) design	$R_s=2$ F_P (k) design
Roof	902	3605	1.00	0.611	2202	721	2202	1101
4th	794	4171	0.77	0.390	1629	834	1629	834
3rd	547	4171	0.54	0.393	1641	834	1641	834
2nd	309	4248	0.30	0.396	1683	850	1683	850

Diaphragm Force, ASCE7-22 Standard

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n W_i} W_{px} \quad (12.10-1)$$

$$F_{px} = 0.2 S_{DS} I_e W_{px} \quad (12.10-2)$$

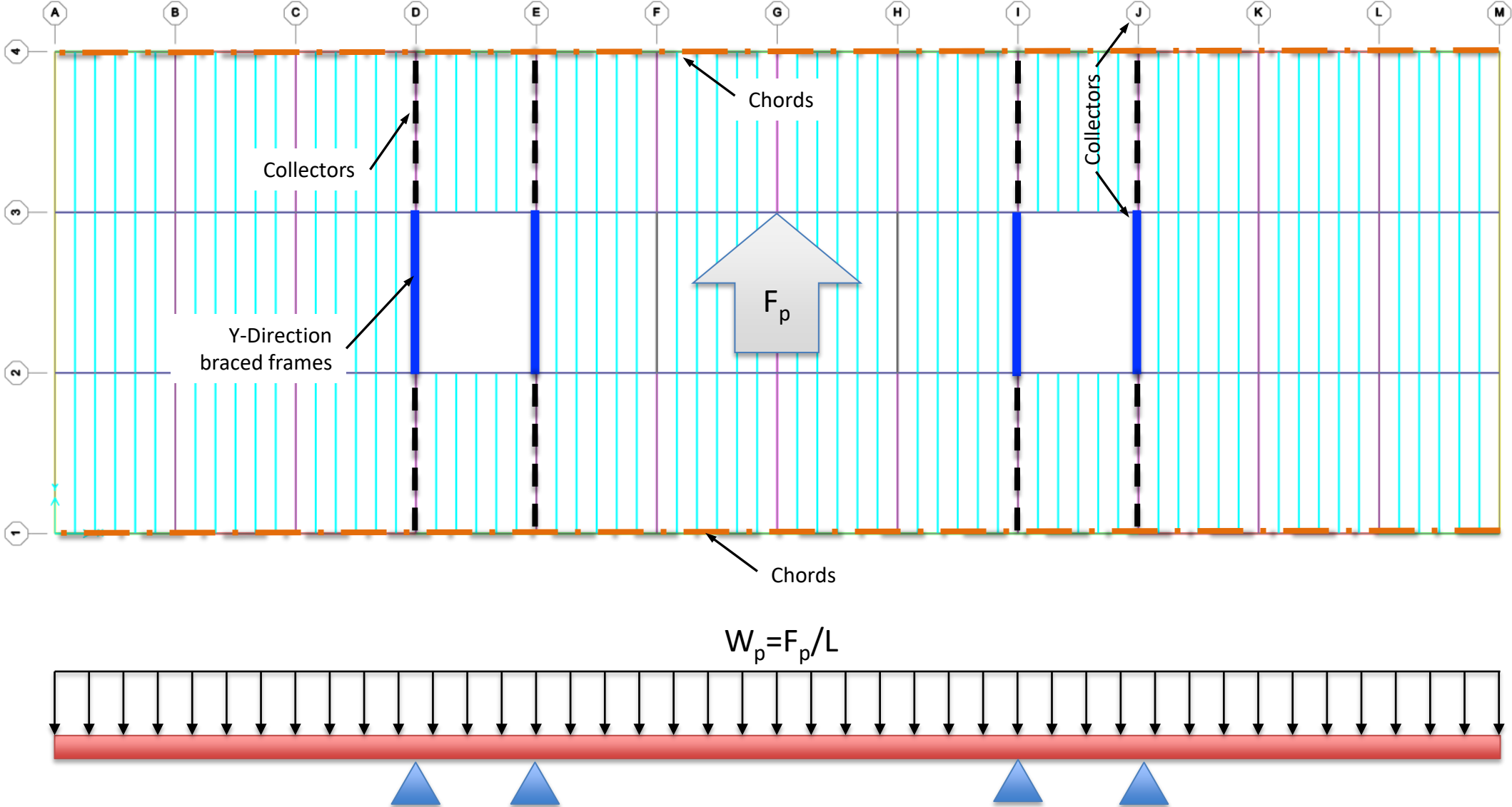
$$F_{px} = 0.4 S_{DS} I_e W_{px} \quad (12.10-3)$$



Level	F_i (k)	W_i (k)	F_P (k)	F_{P-min} (k)	F_{P-max} (k)	F_P (k) design
Roof	902	3605	902.43	720.96	1442	902
4th	794	4171	910.05	834.24	1668	910
3rd	547	4171	783.21	834.24	1668	834
2nd	309	4248	669.48	849.60	1699	850

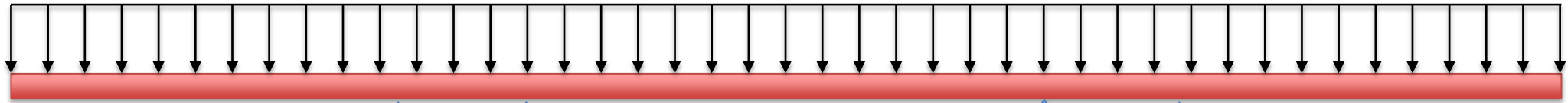
Close to $R_s=2$

Diaphragm Analysis (Simplified Beam Analogy)



Diaphragm demands (ASCE7-22, $R_s=1$)

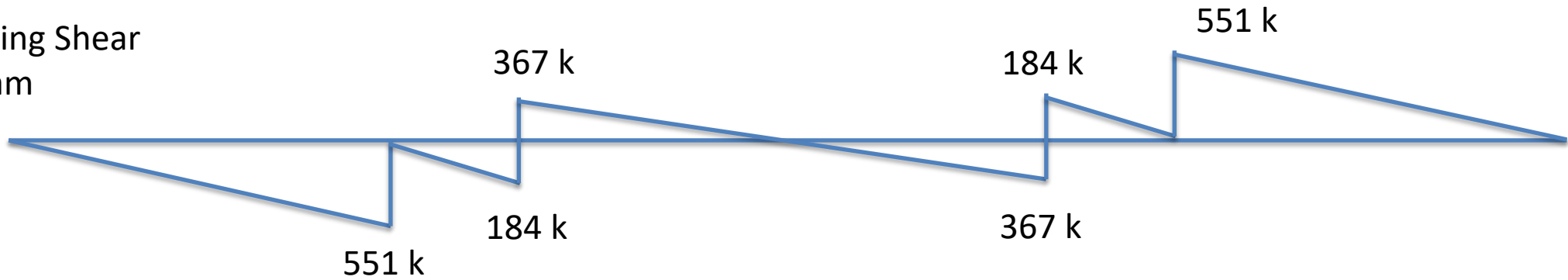
$$W_p = F_p / L = 2202 / 360 = 6.12 \text{ kip/ft}$$



Frame forces assuming rigid diaphragm behavior and no accidental torsion

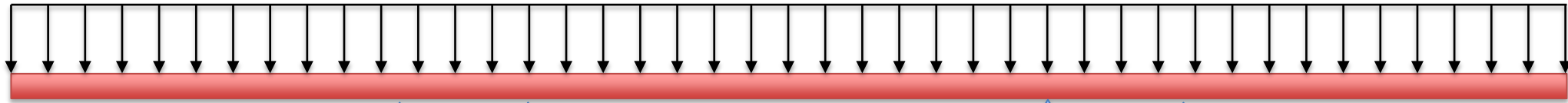


Resulting Shear Diagram



Diaphragm demands (ASCE7-22, traditional method)

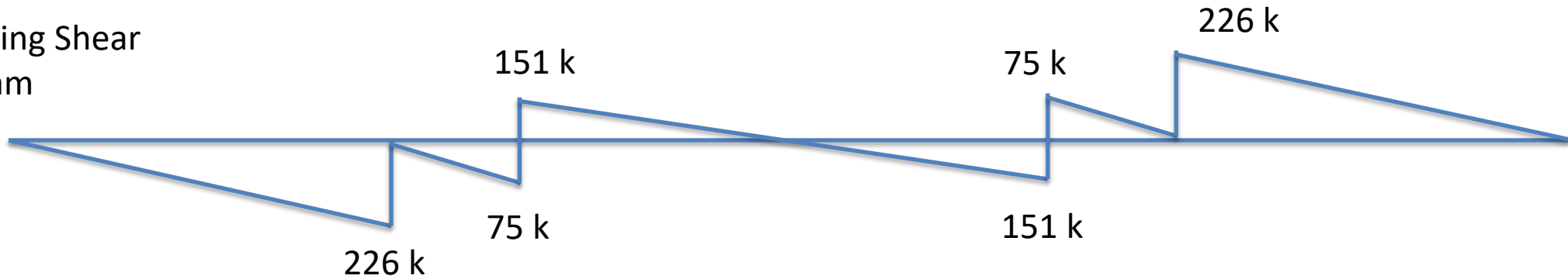
$$W_p = F_p / L = 902 / 360 = 2.51 \text{ kip/ft}$$



Frame forces assuming rigid diaphragm behavior and no accidental torsion



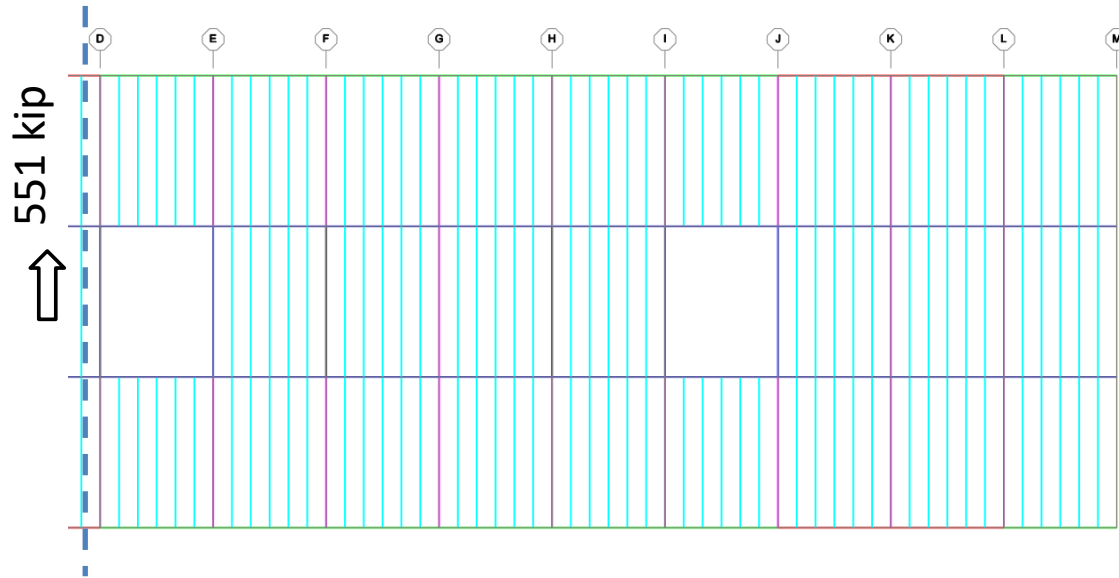
Resulting Shear Diagram



Chord and Collector Design

- Diaphragm chords and collectors are designed as beams under gravity loads and axial load of the diaphragm. Chord and collector are designed as beam-columns.
- Beams are assumed to be fully braced in minor axis direction and LTB. The beams can still experience torsional-flexural buckling.
- Collectors are designed under amplified seismic loads as moderately ductile members.
- Chord and collectors are designed for two different demands in ASCE7-22, including ASCE7-22 standard and ASCE7-22 Alternative design method with $R_s=1$.
- The beam of the braced frame, between grids 2 and 4 on grids A and M for the long direction (Y-direction seismic force) and beams between grids B and C on grids 1 and 5 are designed as non-composite beams connected to the diaphragms via shear studs.

Diaphragm shear demands (ASCE7-22, $R_s=1$)



Design for shear buckling and shear yielding

Shear Strength of Webs

The nominal shear strength, V_n , is

$$V_n = 0.6F_y A_w C_{v1} \quad (G2-1)$$

(i) When $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad (G2-4)$$

(ii) For webs with transverse stiffeners

$$k_v = 5 + \frac{5}{(a/h)^2} \quad (G2-5)$$

$$= 5.34 \text{ when } a/h > 3.0$$

where

a = clear distance between transverse stiffeners, in. (mm)

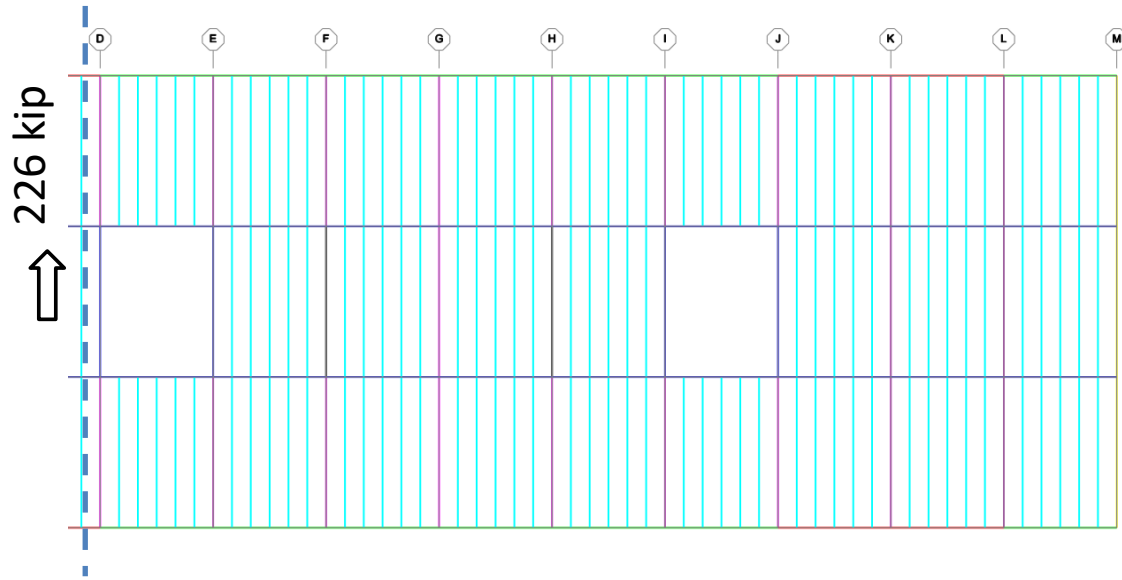
- The diaphragm shear demand is $551 \text{ kip} / 120 \text{ ft} = 4.59 \text{ kip/ft}$.
- Without considering tension field action, the shear capacity can be calculated as follow per AISC 360-22, assuming that joist acting as stiffeners and the steel plate is the web in shear:

- $\frac{a}{h} = \frac{5}{40} = 0.125$
- $k_v = 5 + \frac{5}{0.125^2} = 325$
- $\frac{h}{t_w} = \frac{40 \times 12}{0.5} = 960 > 1.1\sqrt{k_v E/F_y} = 477$, then $C_{v1} = 0.496$
- $\phi V_n = \phi 0.6 F_y A_w C_{v1} = 0.9 \times 0.6 \times 50 \times 80 \times 12 \times 0.5 \times 0.496 = 6428 \text{ kip}$
- $DCR_{shear} = 0.09$

Note: Effect of discrete perimeter fasteners on the shear buckling loads needs to be considered if applicable.

- Although, the shear stress would not increase the plate size, the in-plane shear force needs to be considered in the design of perimeter connectors (welded or bolted)

Diaphragm shear demands (ASCE7-22, Standard)



- The diaphragm shear demand is $226 \text{ kip} / 120 \text{ ft} = 1.88 \text{ kip/ft}$.
- Without considering tension field action, the shear capacity can be calculated as follow per AISC 360-22, assuming that joist acting as stiffeners and the steel plate is the web in shear:
 - $\phi V_n = \phi 0.6 F_y A_w C_{v1} = 0.9 \times 0.6 \times 50 \times 80 \times 12 \times 0.5 \times 0.496 = 6428 \text{ kip}$
 - $DCR_{shear} = 0.04$

Note: Effect of discrete perimeter fasteners on the shear buckling loads needs to be considered in applicable.

- Although, the shear stress would not increase the plate size, the in-plane shear force needs to be considered in the design of perimeter connectors (welded or bolted)

Diaphragm chord demands (ASCE7-22, $R_s=1$)

Diaphragm Chord Demand (Short direction)

$L_{diaphragm} =$	120.00ft
$h_{diaphragm} =$	360.00ft
$L_{cantilever} =$	90.00ft

Level	F_p (k) design	w (k/ft)	M (k-ft)	Chord F.(k)
Roof	2202	18.347	33024	92
4th	1629	13.572	24430	68
3rd	1641	13.673	24611	68
2nd	1683	14.026	25248	70

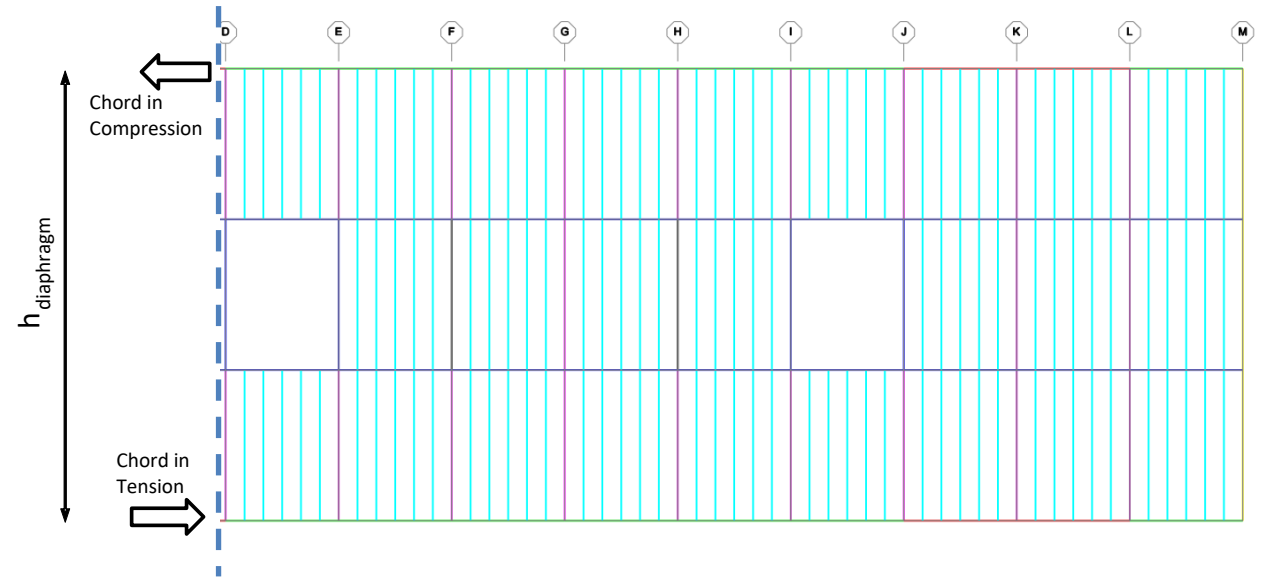
Diaphragm Chord Demand (Long direction)

$L_{diaphragm} =$	360.00ft
$h_{diaphragm} =$	120.00ft
$L_{cantilever} =$	90.00ft

End of Cantilever

Level	F_p (k) design	w (k/ft)	M (k-ft)	Chord F.(k)
Roof	2202	6.116	24768	206
4th	1629	4.524	18323	153
3rd	1641	4.558	18458	154
2nd	1683	4.675	18936	158

- Although, the beam/girder cross-section is not changed, the chord force needs to be considered in the connection design.



- The calculated chord forces are calculated assuming there is no contribution from the other girders or the top steel plate. Accordingly, the chord forces deem to be conservative.
- The chord force of 206 kip in W30x132 perimeter girder with axial capacity of 1563 kip, increases the DCR by 0.13, while the exiting DCR is less than 0.30 and the axial force does not change the cross-section.
- The chord force of 92 kip in W24x94 joist with axial capacity of 1029 kip, increases the DCR by 0.09, while the exiting DCR is less than 0.22 and the axial force does not change the cross-section.

Diaphragm chord demands (ASCE7-22, Standard)

Diaphragm Chord Demand (Short direction) - Standard

$L_{diaphragm} =$	120.00ft
$h_{diaphragm} =$	360.00ft
$L_{cantilever}$	90.00ft

Level	F_p (k) design	w (k/ft)	M (k-ft)	Chord F.(k)
Roof	902	7.520	13536	38
4th	910	7.584	13651	38
3rd	834	6.952	12514	35
2nd	850	7.080	12744	35

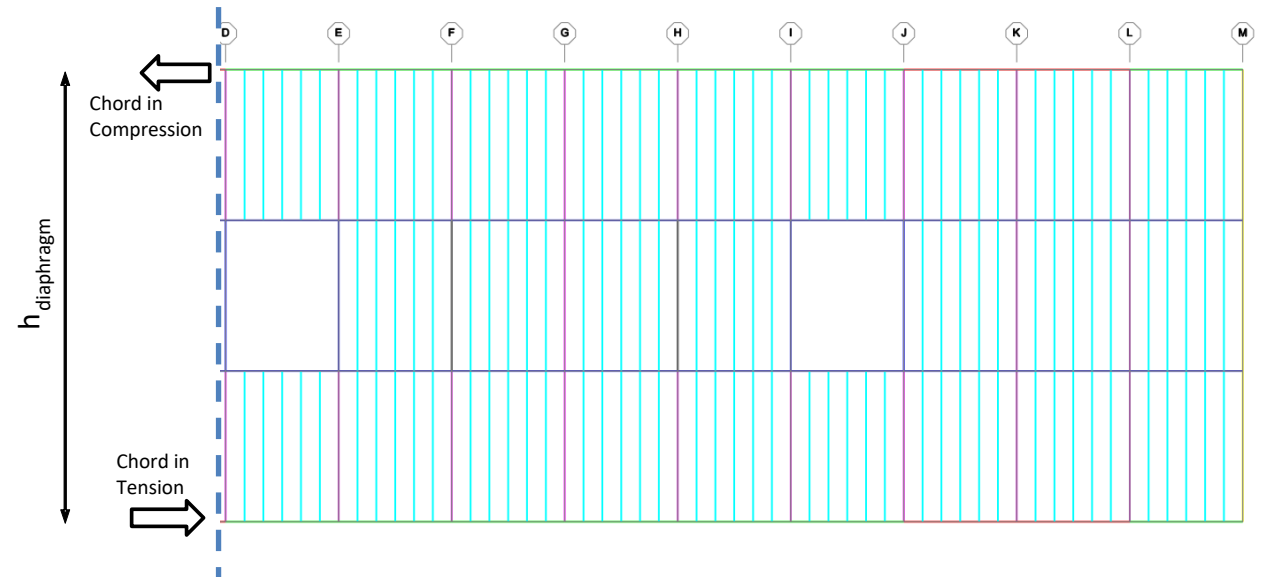
Diaphragm Chord Demand (Long direction) - Standard

$L_{diaphragm} =$	360.00ft
$h_{diaphragm} =$	120.00ft
$L_{cantilever}$	90.00ft

End of Cantilever

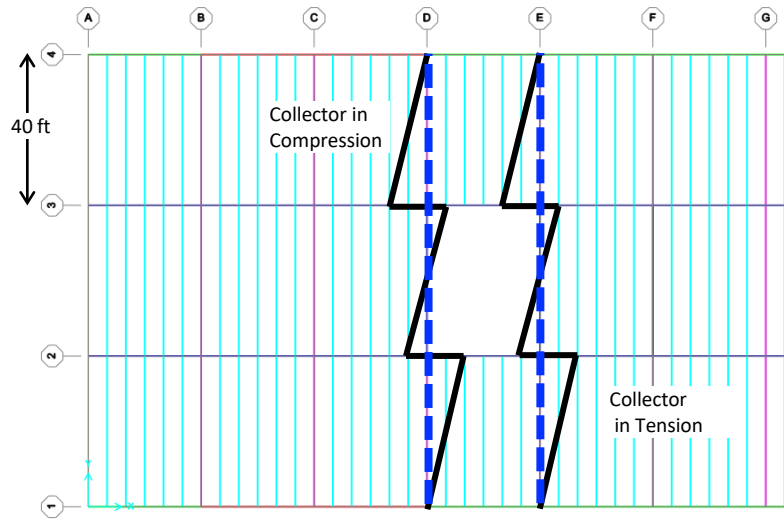
Level	F_p (k) design	w (k/ft)	M (k-ft)	Chord F.(k)
Roof	902	2.507	10152	85
4th	910	2.528	10238	85
3rd	834	2.317	9385	78
2nd	850	2.360	9558	80

- Although, the beam/girder cross-section is not changed, the chord force needs to be considered in the connection design.



- The calculated chord forces are calculated assuming there is no contribution from the other girders or the top steel plate. Accordingly, the chord forces deem to be conservative.
- The chord force of 85 kip in W30x132 perimeter girder with axial capacity of 1563 kip, increases the DCR by 0.055, while the exiting DCR is less than 0.30 and the axial force does not change the cross-section.
- The chord force of 38 kip in W24x94 joist with axial capacity of 1029 kip, increases the DCR by 0.037, while the exiting DCR is less than 0.22 and the axial force does not change the cross-section.

Diaphragm collector demands (ASCE7-22, $R_s=1$) – Long Direction



Collector Design in tension and compression (Long direction)

$L_{diaphragm} =$	360.00
$h_{diaphragm} =$	120.00
Lines of LFRS	4.00
Number of bays	1.00

$$(1.2 + 0.2S_{DS})D + 1.5Q_E + 0.5L$$

Level	1.5Fp (kips)	v (lb/ft)
Roof	3302	6880
4th	2443	5090
3rd	2461	5127
2nd	2525	5260

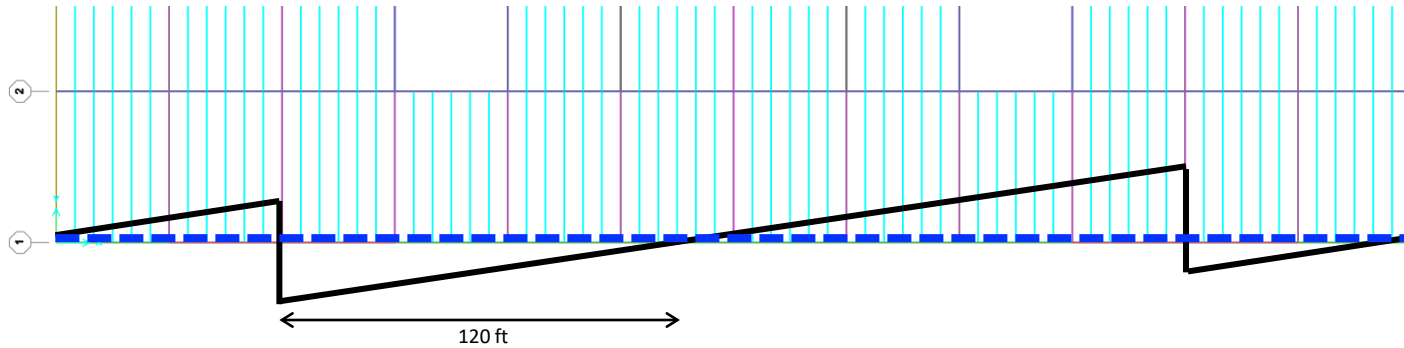
Level	Collector length (ft)	Collector force (kip)
Roof	40	275
4th	40	204
3rd	40	205
2nd	40	210

Collector force in ASCE7-22 (Alternative methods) is as follows (section 12.10.3.3):

Collectors and their connections, including connections to vertical elements shall be designed to resist 1.5 times the diaphragm inertial forces plus 1.5 times the design transfer forces.

- The collector force of 275 kip in W24x94 joist with axial capacity of 1029 kip, increases the DCR by 0.27, while the exiting DCR is less than 0.22 and the axial force does not change the cross-section.
- Although, the beam/girder cross-section is not changed, the collector force needs to be considered in the connection design.

Diaphragm collector demands (ASCE7-22, $R_s=1$) – Short Direction



Collector Design in tension and compression (Short direction) - $R_s=1$

$L_{diaphragm} =$	360.00
$h_{diaphragm} =$	120.00
Lines of LFRS	2.00
Number of bays	4.00

$$(1.2 + 0.2S_{DS})D + 1.5Q_E + 0.5L$$

Level	1.5Fp (kips)	v (lb/ft)
Roof	3302	4587
4th	2443	3393
3rd	2461	3418
2nd	2525	3507

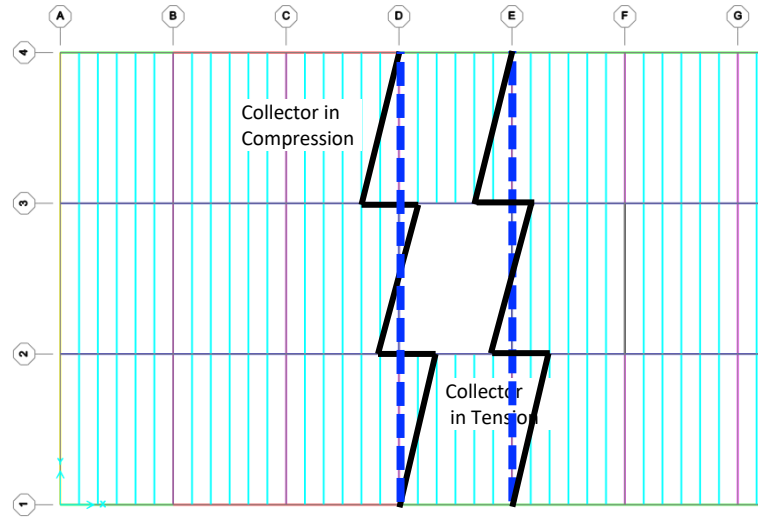
Level	Collector length (ft)	Collector force (kip)
Roof	120	550
4th	120	407
3rd	120	410
2nd	120	421

Collector force in ASCE7-22 (Alternative methods) is as follows (section 12.10.3.3):

Collectors and their connections, including connections to vertical elements shall be designed to resist 1.5 times the diaphragm inertial forces plus 1.5 times the design transfer forces.

- The chord force of 550 kip in W30x132 perimeter girder with axial capacity of 1563 kip, increases the DCR by 0.35, while the exiting DCR is less than 0.30 and the axial force does not change the cross-section.
- Although, the beam/girder cross-section is not changed, the collector force needs to be considered in the connection design.

Diaphragm collector demands (ASCE7-22, Standard) – Long Direction



Collector Design in tension and compression (Long direction)

$L_{diaphragm} =$	360.00
$h_{diaphragm} =$	120.00
Lines of LFRS	4.00
Number of bays	1.00

$$(1.2 + 0.2S_{DS})D + 1.5Q_E + 0.5L$$

Level	F1 (kips)	F2(kips)	F3(kips)	Maximum	v (lb/ft)
Roof	1805	1805	721	1805	3760
4th	1588	1820	834	1820	3792
3rd	1094	1566	834	1566	3263
2nd	618	1339	850	1339	2789

*F3 needs to be check with gravity load

Collector force in ASCE7-22 is maximum of the following (section 12.10.2.1):

(F1) Amplified Seismic forces determined by equivalent lateral force procedure modal response analysis.

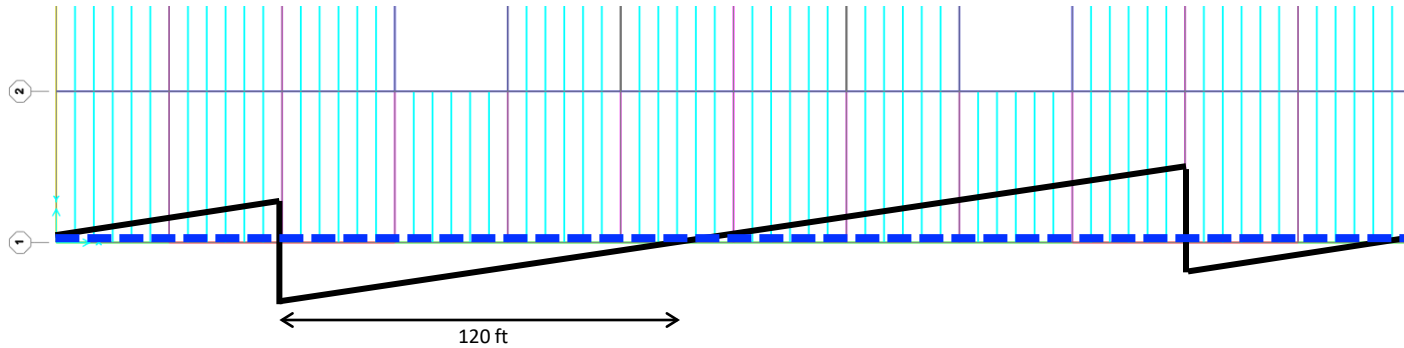
(F2) Amplified diaphragm forces without minimum ($0.2S_{DS}I_e w_{px}$) and maximum diaphragm ($0.4S_{DS}I_e w_{px}$) forces.

(F3) Combination of Gravity load and minimum diaphragm forces $0.2S_{DS}I_e w_{px}$ (without amplification)

Level	Collector length (ft)	Collector force (kip)
Roof	40	150
4th	40	152
3rd	40	131
2nd	40	112

- The collector force of 152 kip in W24x94 joist with axial capacity of 1029 kip, increases the DCR by 0.15, while the exiting DCR is less than 0.22 and the axial force does not change the cross-section.
- Although, the beam/girder cross-section is not changed, the collector force needs to be considered in the connection design.

Diaphragm collector demands (ASCE7-22, Standard) – Short Direction



Collector Design in tension and compression (Short direction) - Standard

$L_{diaphragm}$ =	360.00
$h_{diaphragm}$ =	120.00
Lines of LFRS	2.00
Number of bays	4.00

Level	F1 (kips)	F2(kips)	F3(kips)	Maximum v (lb/ft)
Roof	1805	1805	721	1805
4th	1588	1820	834	1820
3rd	1094	1566	834	1566
2nd	618	1339	850	1339

*F3 needs to be check with gravity load

Level	Collector length (ft)	Collector force (kip)
Roof	120	301
4th	120	303
3rd	120	261
2nd	120	223

Collector force in ASCE7-22 is maximum of the following (section 12.10.2.1):

(F1) Amplified Seismic forces determined by equivalent lateral force procedure modal response analysis.

(F2) Amplified diaphragm forces without minimum ($0.2S_{DS}I_e w_{px}$) and maximum diaphragm ($0.4S_{DS}I_e w_{px}$) forces.

(F3) Combination of Gravity load and minimum diaphragm forces $0.2S_{DS}I_e w_{px}$ (without amplification)

- The chord force of 301 kip in W30x132 perimeter girder with axial capacity of 1563 kip, increases the DCR by 0.19, while the exiting DCR is less than 0.30 and the axial force does not change the cross-section.
- Although, the beam/girder cross-section is not changed, the collector force needs to be considered in the connection design.

Width-to-thickness ratio of Chords

ASCE7 Standard			Table D1.1 limiting ratios							Member check			
Story	Fp Direction	Cross-section	F_y (ksi)	E (ksi)	R_y	P_u^* (kip)	C_a	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
Roof	X	W24x94	50	29000	1.1	92	0.07	54.91	7.35	41.90	Passed	5.2	Passed
Floors (max)	X	W24x94	50	29000	1.1	70	0.05	55.87	7.35	41.90	Passed	5.2	Passed
Roof	Y	W30x132	50	29000	1.1	206	0.11	52.42	7.35	43.90	Passed	5.3	Passed
Floors (max)	Y	W30x132	50	29000	1.1	158	0.08	53.97	7.35	43.90	Passed	5.3	Passed

Rs=1			Table D1.1 limiting ratios							Member check			
Story	Fp Direction	Cross-section	F_y	E	R_y	P_u^*	C_a	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
Roof	X	W24x94	50	29000	1.1	92	0.07	54.91	7.35	41.90	Passed	5.2	Passed
Floors (max)	X	W24x94	50	29000	1.1	70	0.05	55.87	7.35	41.90	Passed	5.2	Passed
Roof	Y	W30x132	50	29000	1.1	206	0.11	52.42	7.35	43.90	Passed	5.3	Passed
Floors (max)	Y	W30x132	50	29000	1.1	158	0.08	53.97	7.35	43.90	Passed	5.3	Passed

* Chord Demands

** Designed for Highy Ductile Members

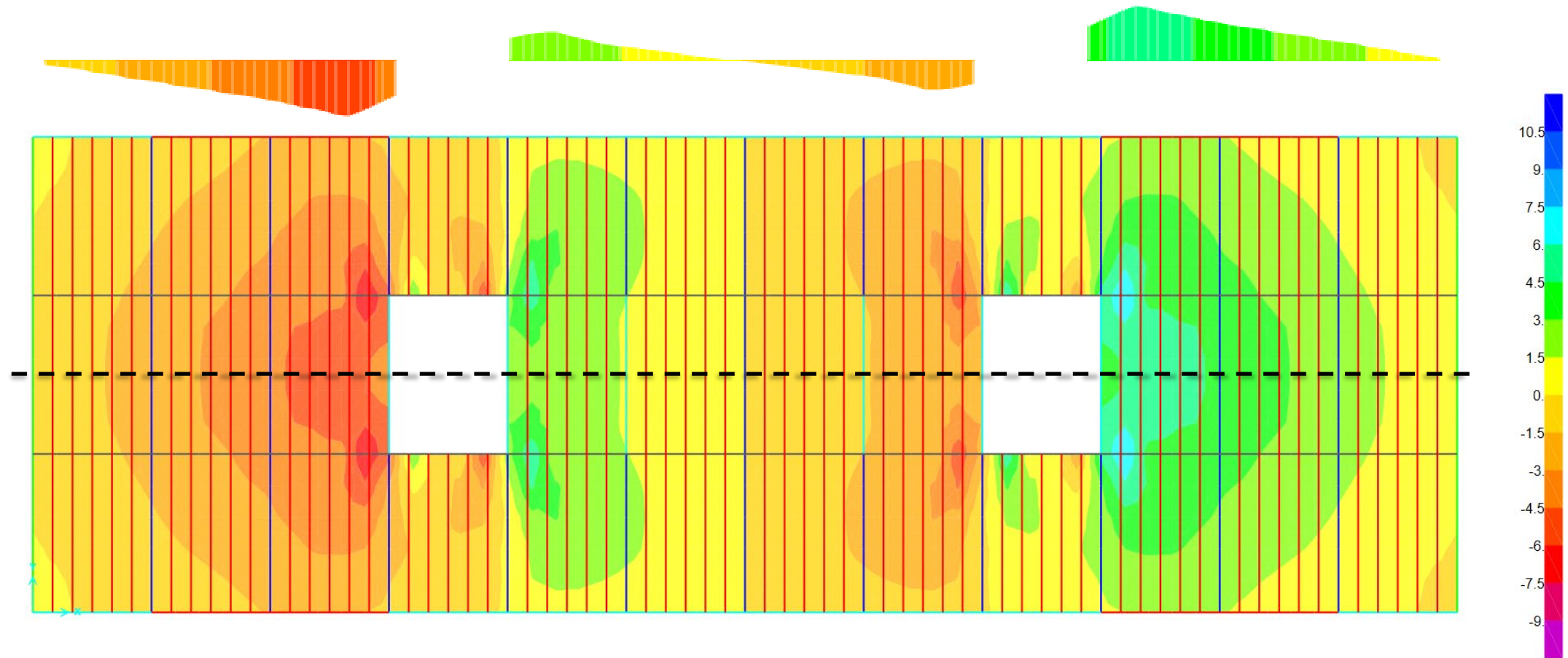
Width-to-thickness ratio of Collectors

ASCE7 Standard			Table D1.1 limiting ratios							Member check			
Story	Braced frame	Cross-section	F_y (ksi)	E (ksi)	R_y	P_u^* (kip)	C_a	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
Roof	X	W30x132	50	29000	1.1	344	0.18	50.54	7.35	43.90	Passed	5.3	Passed
Floors (max)	X	W30x132	50	29000	1.1	263	0.14	51.39	7.35	43.90	Passed	5.3	Passed
Roof	Y	W24x94	50	29000	1.1	94	0.07	54.81	7.35	41.90	Passed	5.2	Passed
Floors (max)	Y	W24x94	50	29000	1.1	158	0.12	51.83	7.35	41.90	Passed	5.2	Passed

Rs=1			Table D1.1 limiting ratios							Member check			
Story	Braced frame	Cross-section	F_y	E	R_y	P_u^*	C_a	h/t_w	$b_f/(2t_f)$	h/t_w	Check	$b_f/(2t_f)$	Check
Roof	X	W30x132	50	29000	1.1	344	0.18	50.54	7.35	43.90	Passed	5.3	Passed
Floors (max)	X	W30x132	50	29000	1.1	263	0.14	51.39	7.35	43.90	Passed	5.3	Passed
Roof	Y	W24x94	50	29000	1.1	275	0.2	50.10	7.35	41.90	Passed	5.2	Passed
Floors (max)	Y	W24x94	50	29000	1.1	210	0.15	51.05	7.35	41.90	Passed	5.2	Passed

* Collector Demands

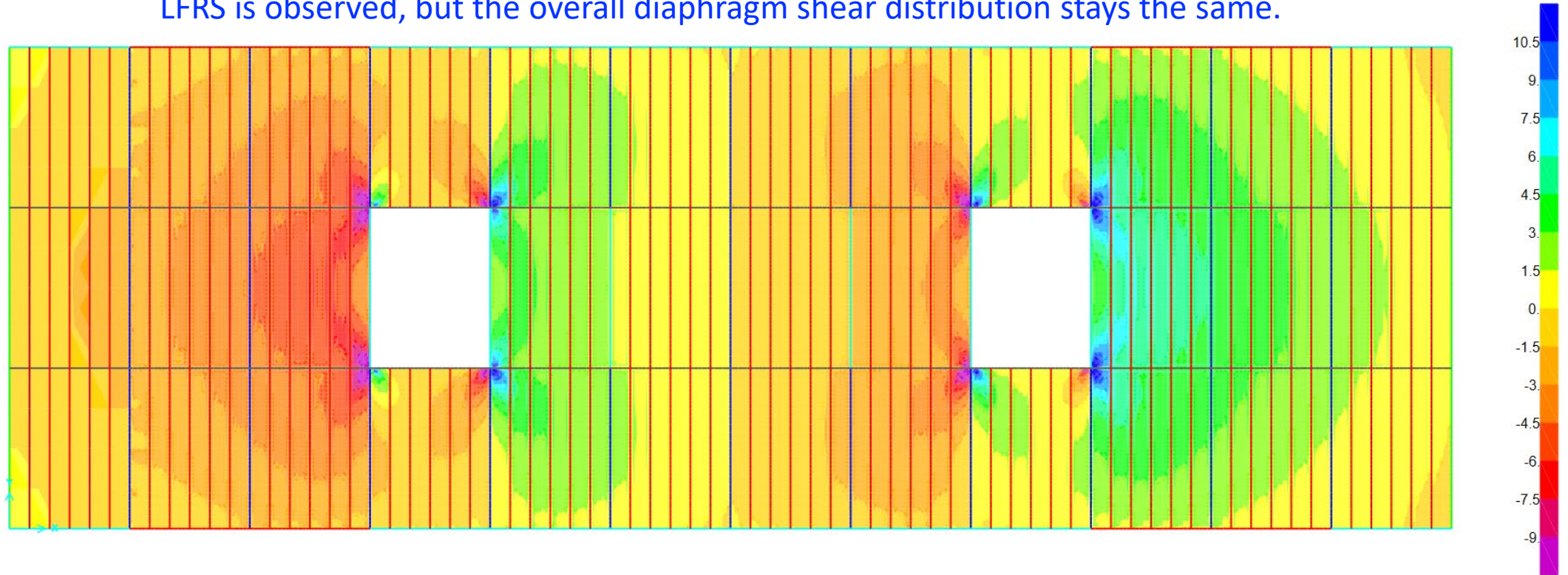
Diaphragm demands (In-plane shear F12 kip/ft) Y (long) - direction ASCE7-22 $R_s=1$



- Maximum shear stress in the diaphragm skin is around 1.7 ksi, which is small by inspection.
- The connections between the steel plate in the FastFloor modules need to be designed considering these demands.

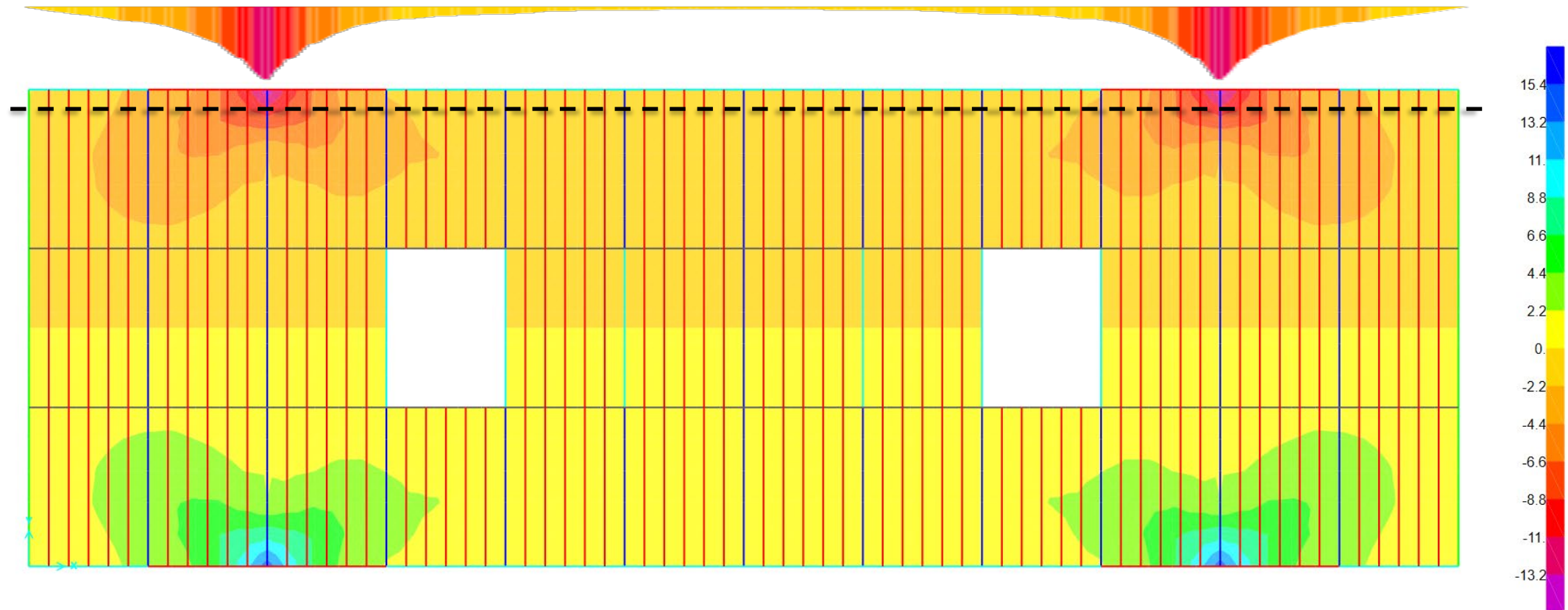
Diaphragm demands (In-plane shear F_{12} kip/ft) Y (long) - direction ASCE7-22 $R_s=1$ Mesh sensitivity

When the mesh size reduced from 5 ft to a maximum of 1 ft, a higher stress resolution at the connections to LFRS is observed, but the overall diaphragm shear distribution stays the same.



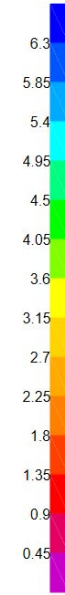
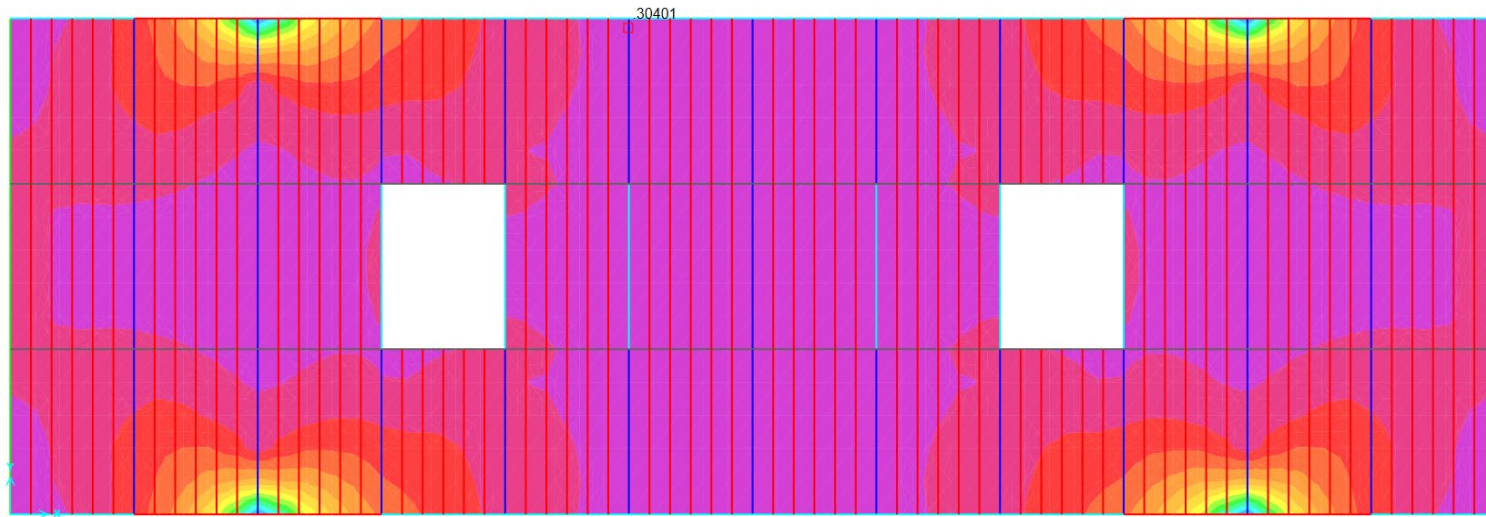
This analysis does not consider the cut-outs needed around the column, discrete connection between the steel plate and the frames, and effect of different members framing into column in the connection area. The actual load path at these corners need to be consider in the design.

Diaphragm demands (In-plane shear F12 kip/ft) X (short) - direction ASCE7-22 $R_s=1$

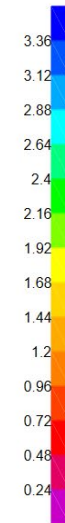
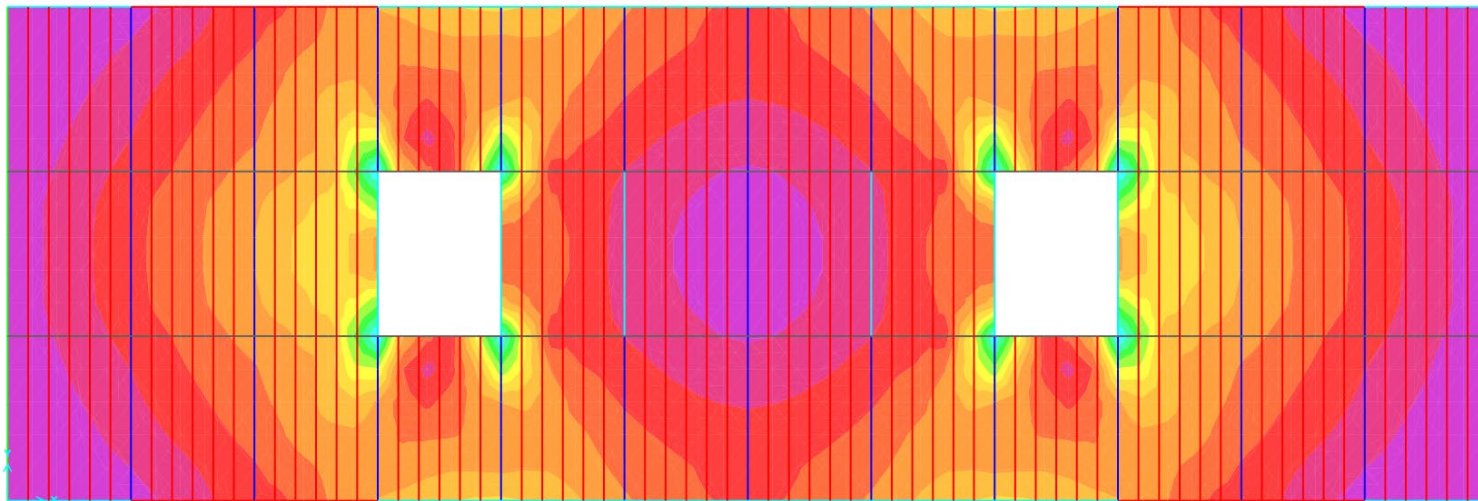


- Maximum shear stress in the diaphragm skin is around 2.6 ksi, which is small by inspection.
- The connections between the steel plate in the FastFloor modules need to be designed considering these demands.

Diaphragm demands (von Mises Stress ksi) - ASCE7-22 $R_s=1$



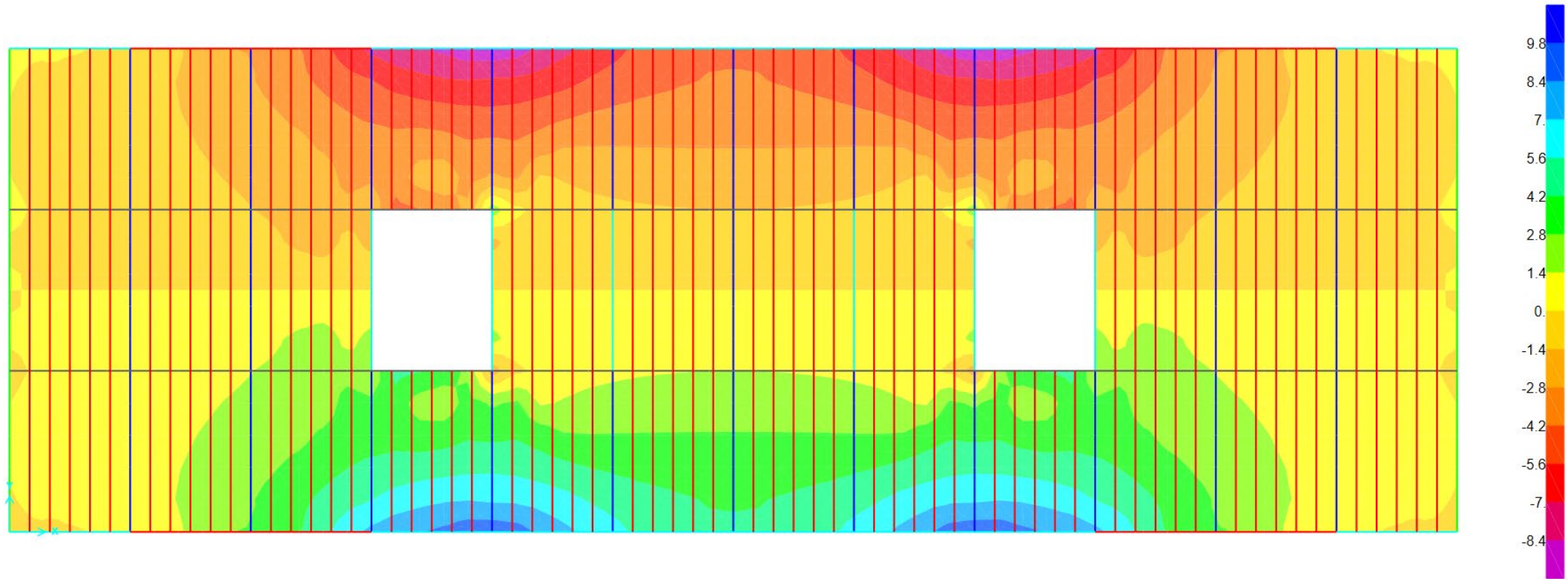
X (short) -direction forces



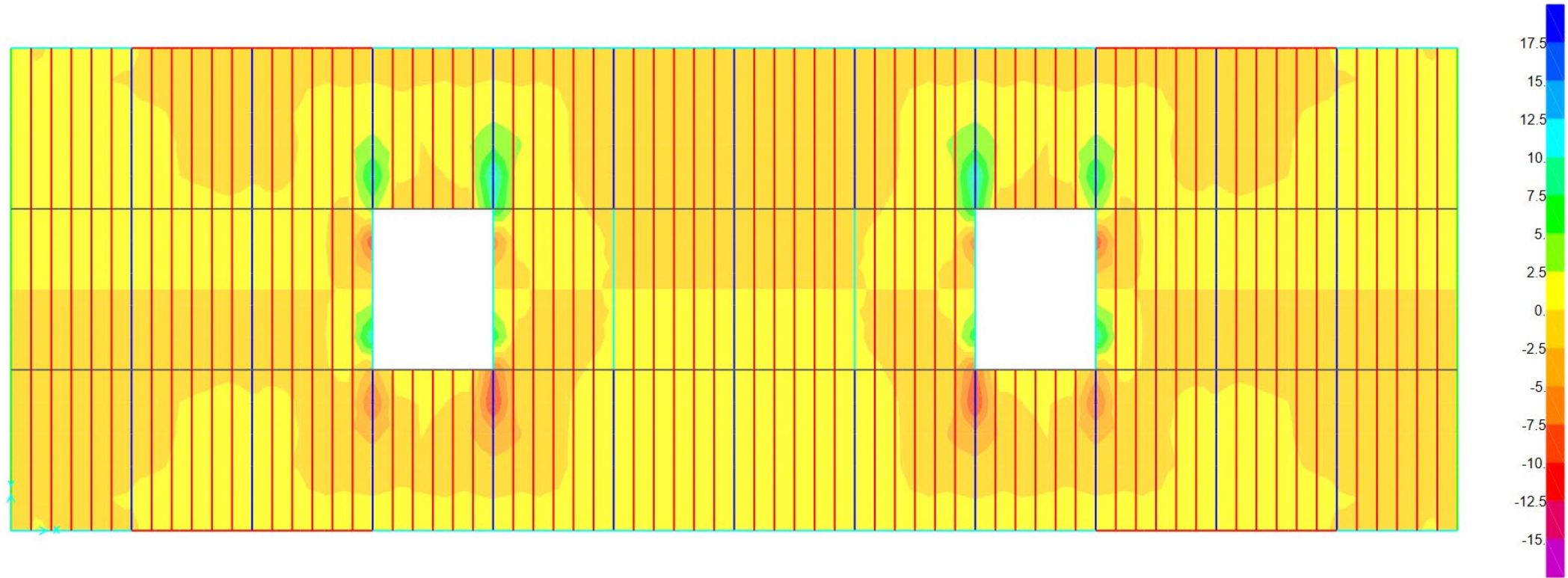
Y (long) -direction forces

Maximum von Mises stress in the diaphragm skin is around 6.3 ksi at brace points.

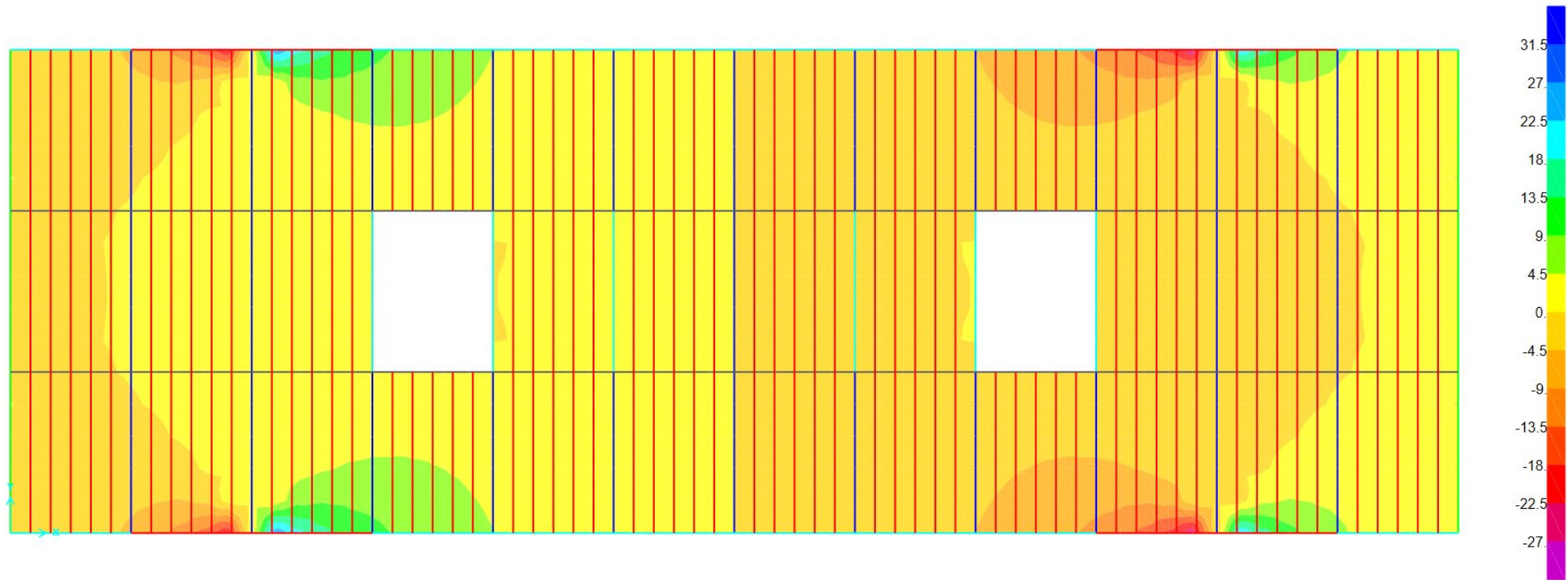
Diaphragm demands (In-plane shear F11 kip/ft) Y (long) - direction ASCE7-22 $R_s=1$



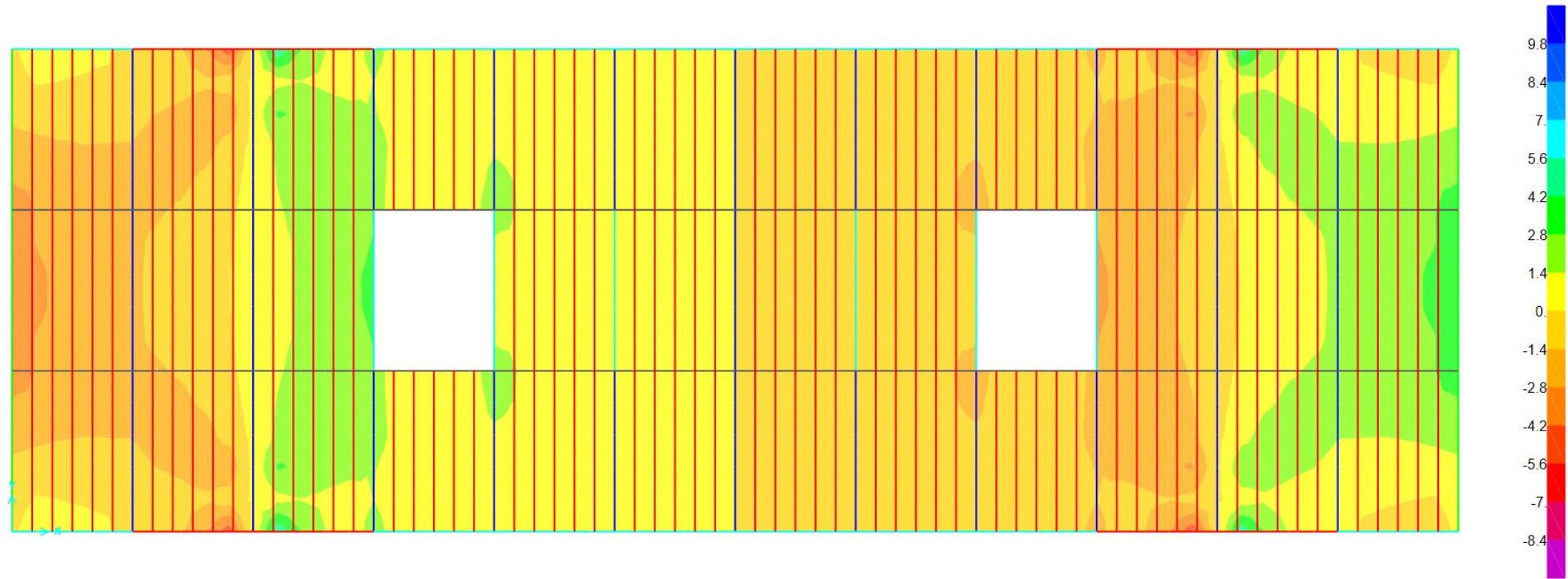
Diaphragm demands (In-plane shear F22 kip/ft) Y (long) - direction ASCE7-22 $R_s=1$



Diaphragm demands (In-plane shear F11 kip/ft) X (short) - direction ASCE7-22 $R_s=1$

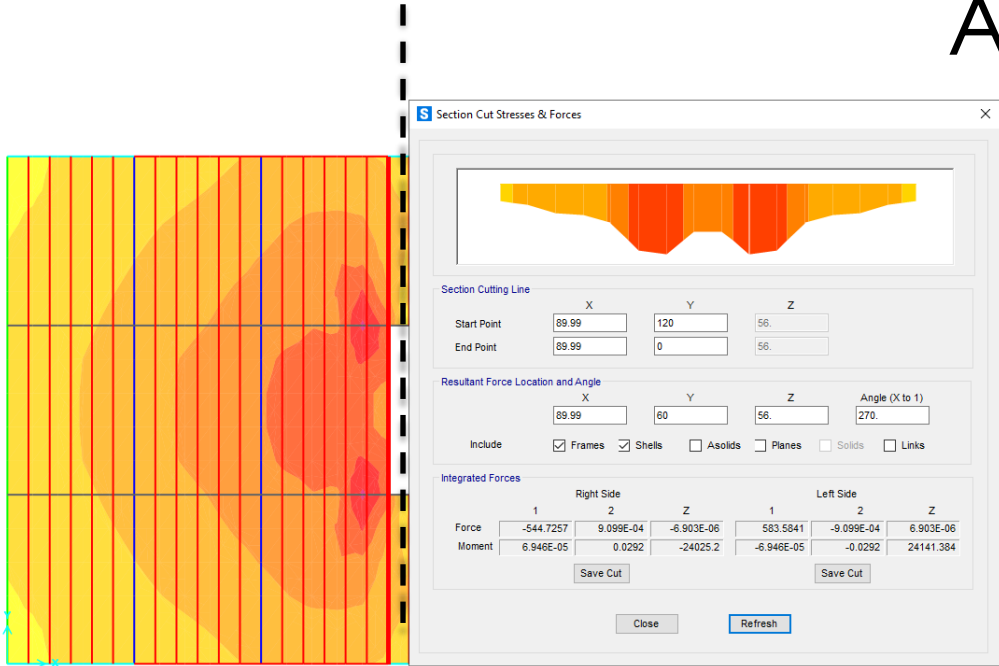


Diaphragm demands (In-plane shear F11 kip/ft) X (short) - direction ASCE7-22 $R_s=1$



In-plane demands in frames and shell, Y (long) -direction forces

ASCE7-22 $R_s=1$

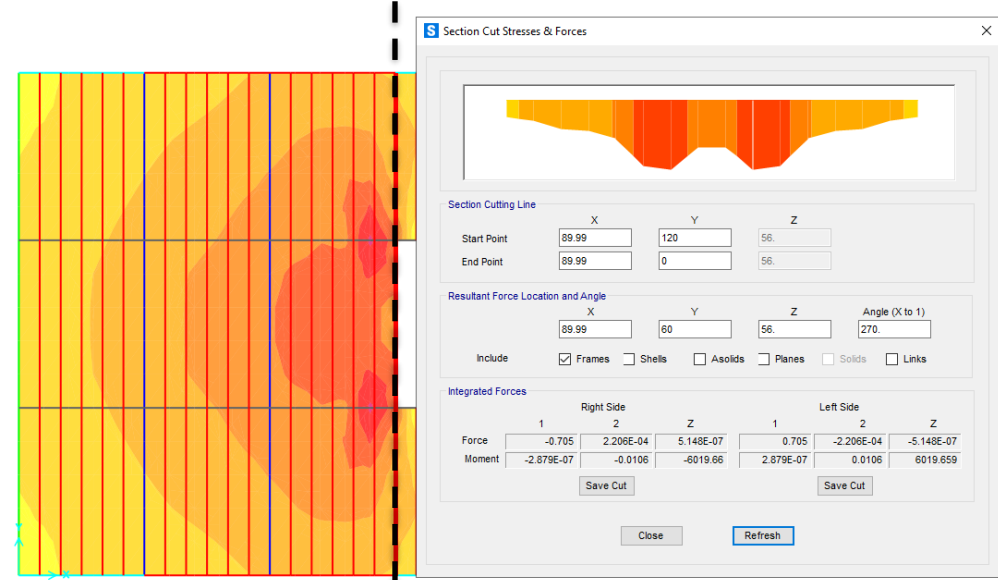
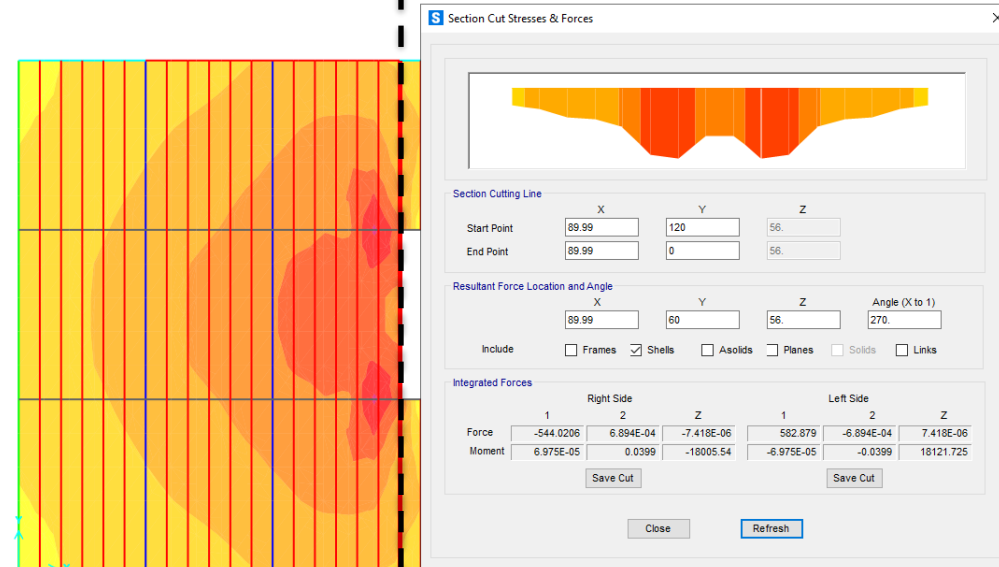


Total in-plane moment = 24141 kip-ft

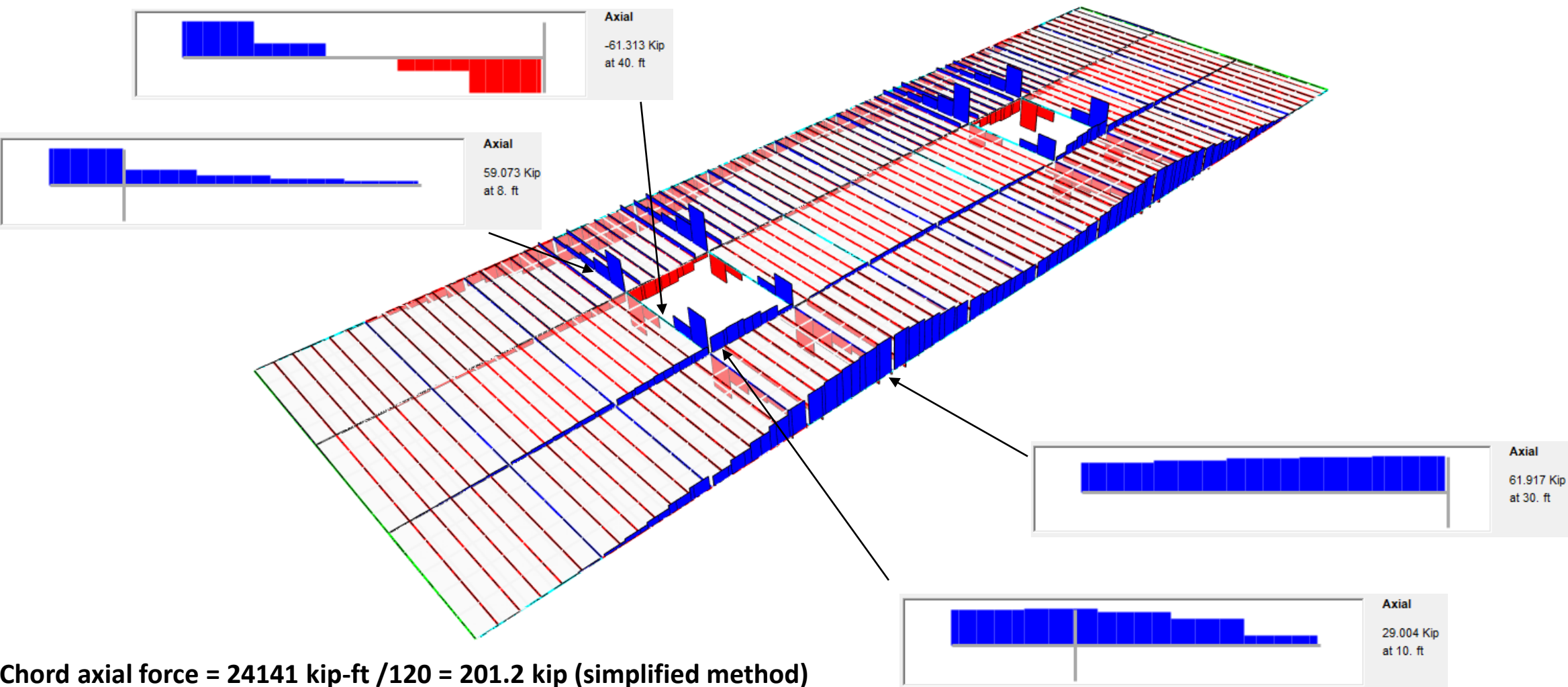
In-place moment in the plate (shell)= 18121 kip-ft **75% contribution**

In-plane moment due to girders (beam)= 6019 kip-ft **25% contribution**

Dividing the total in-plane demand by the depth of the depth of diaphragm could be conservative. 75% of the moment stays in the steel plate.

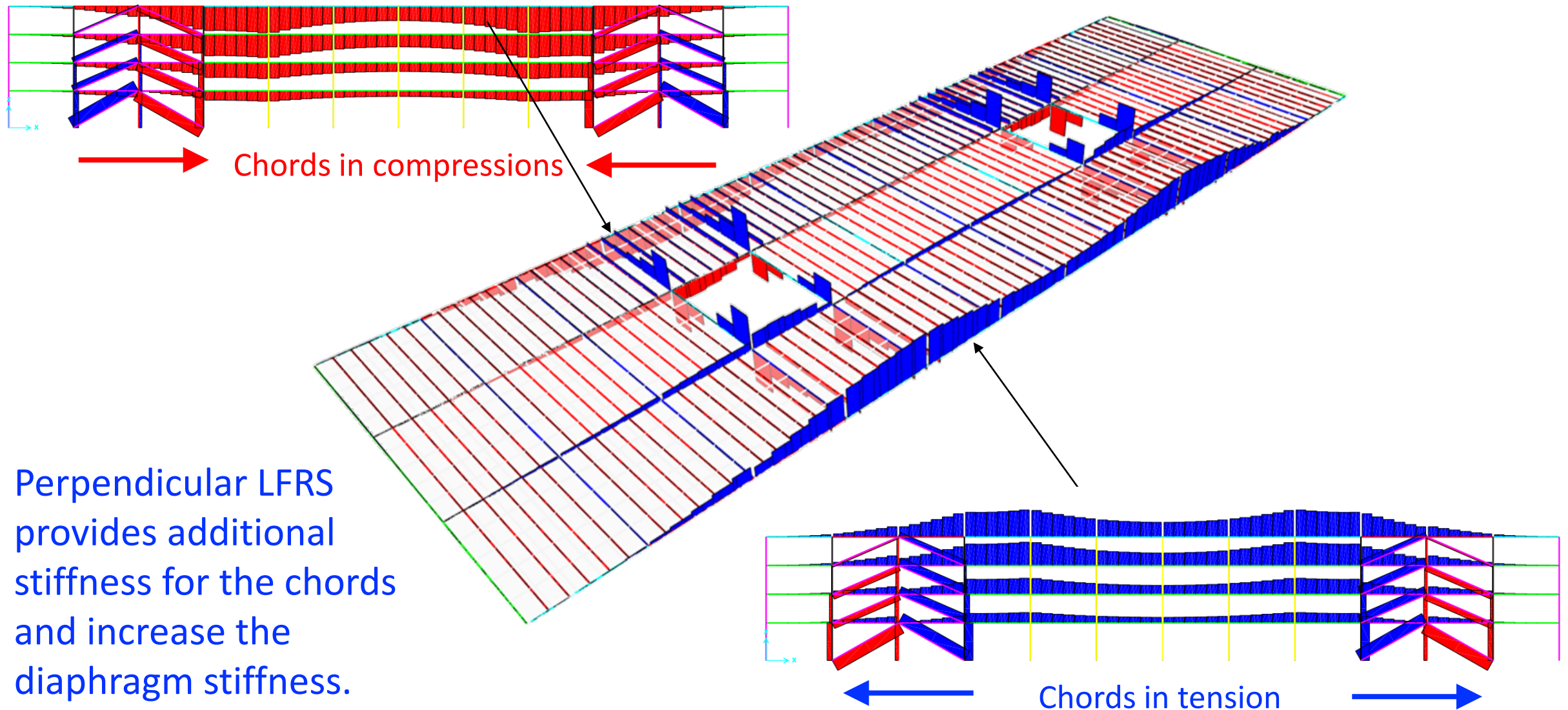


Chord and collector demands, Y (long) -direction forces ASCE7-22 $R_s=1$

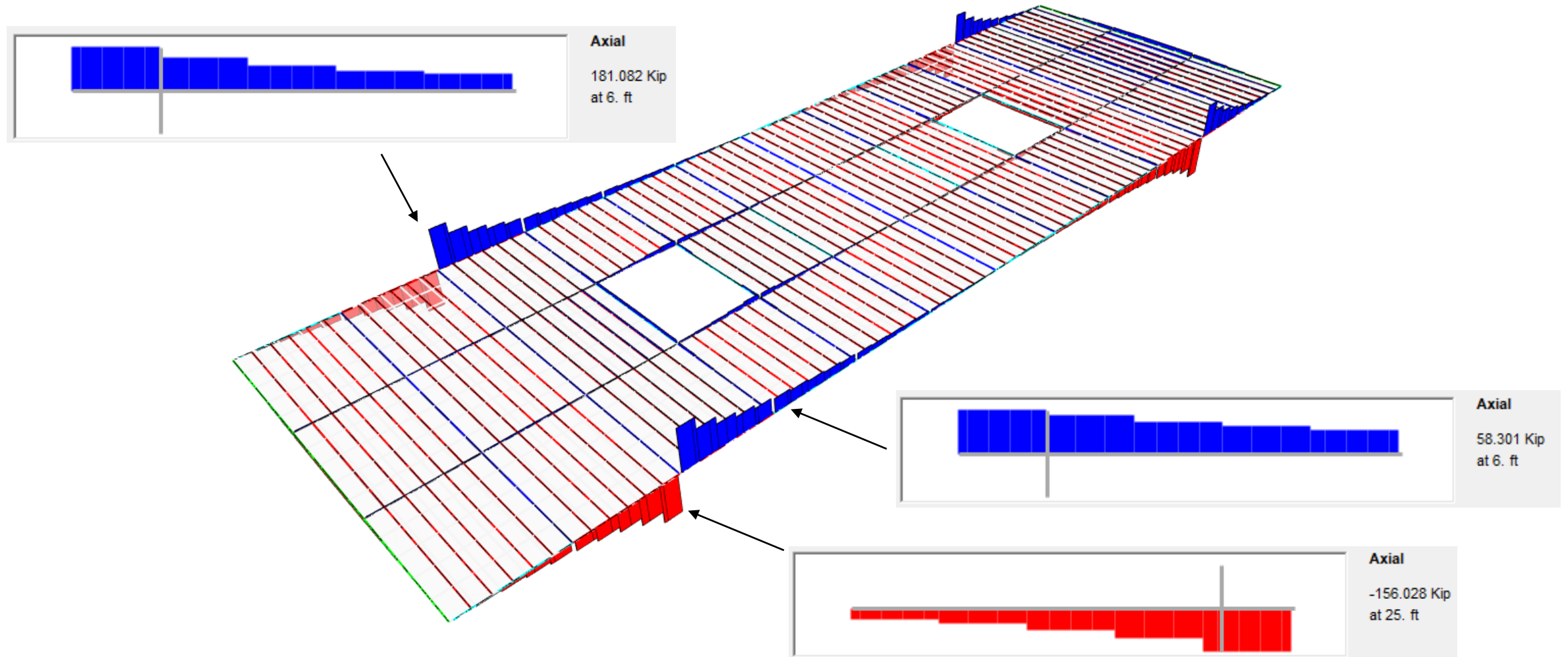


Chord axial force = $24141 \text{ kip-ft} / 120 = 201.2 \text{ kip}$ (simplified method)
Chord force in the SAP2000 model = 61.9 kip (30% of simplified method)
Collector axial force 183 kip (simplified method, without 1.5 factor)
Chord force in the SAP2000 model = 59kip (32% of simplified method)

Effect of Perpendicular LFRS on Diaphragm Stiffness



Chord and collector demands, X (short) -direction forces ASCE7-22 $R_s=1$

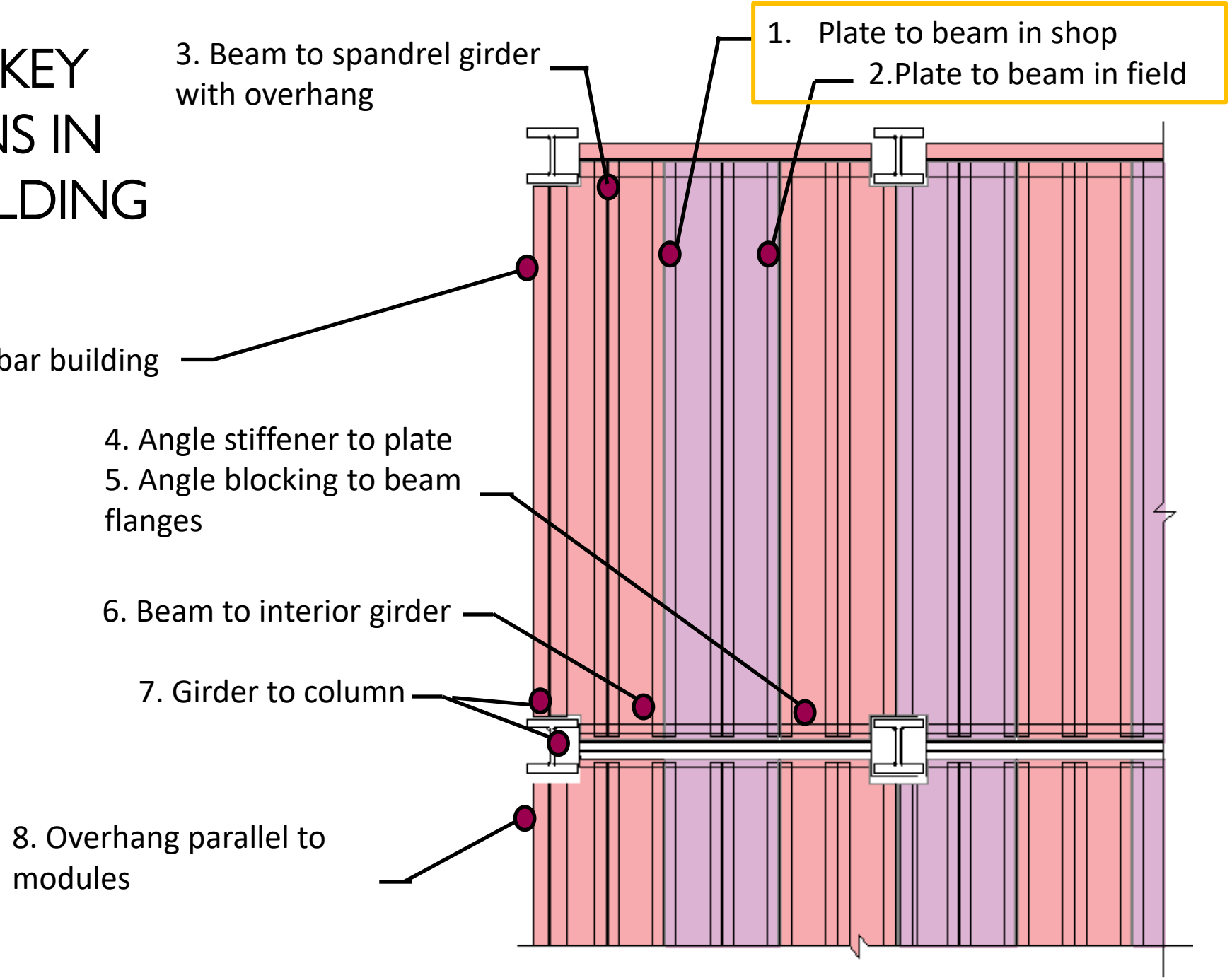


Collector axial force 366 kip (simplified method, without 1.5 factor)
Collector force in the SAP2000 model = 181 kip (49% of simplified method)

IDENTIFYING KEY CONNECTIONS IN ARCHETYPE BUILDING

Portion of the 4-story bar building

- Beam to beam, beam to column shear tab connection has been analyzed.
- Today's focus will be on seam connection.



SEAM CONNECTION – MODULE TO BEAM

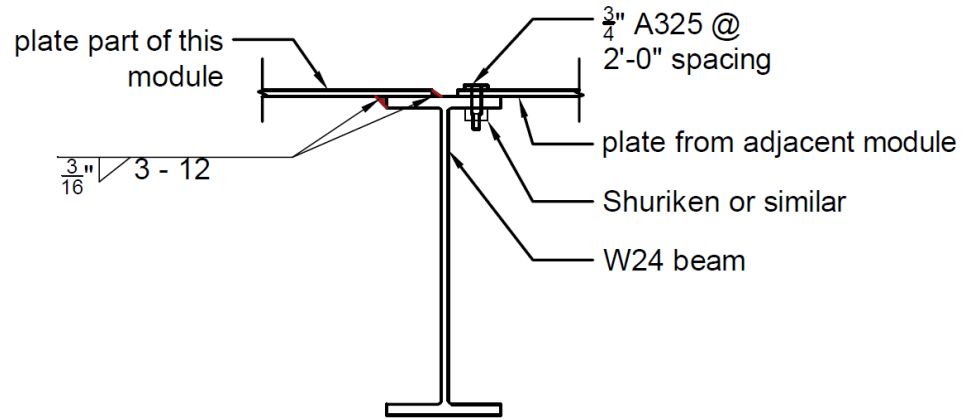
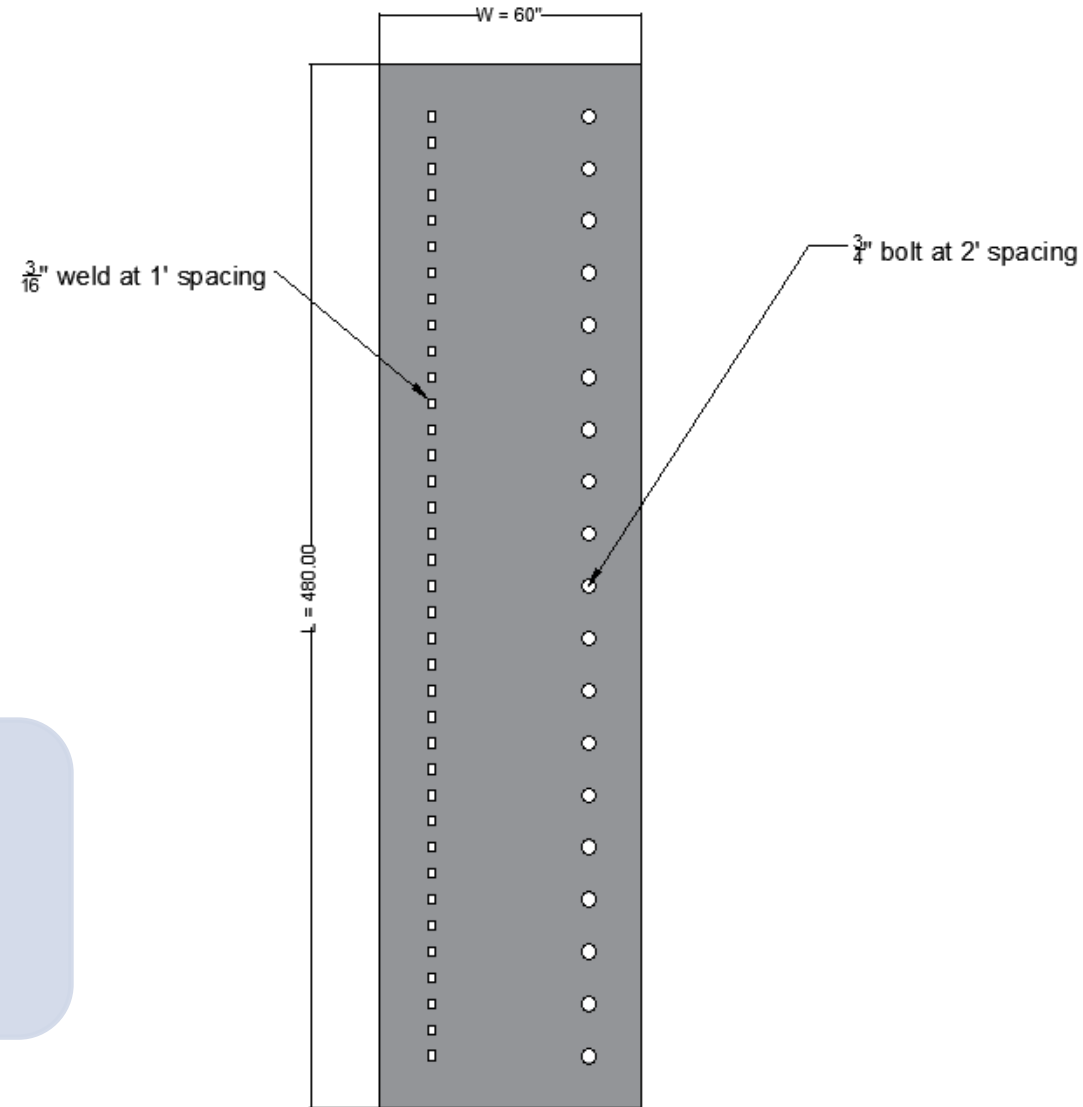


Plate to Beam at Module Edges - Bolts

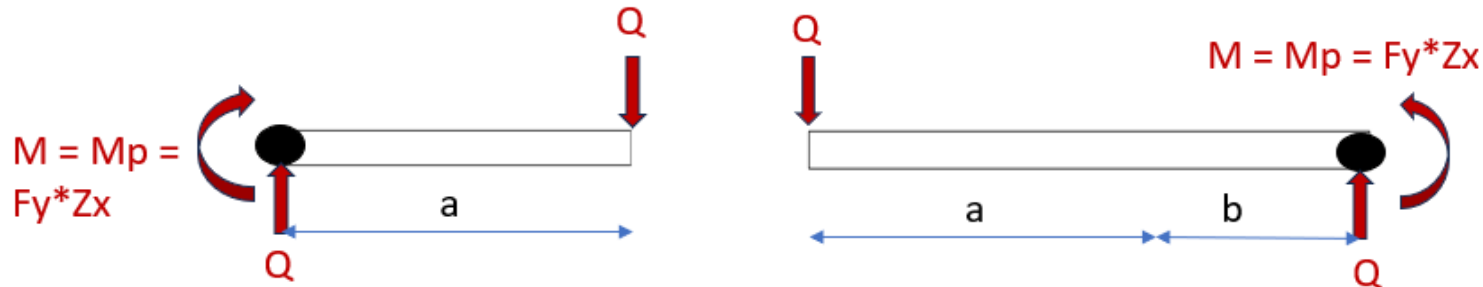
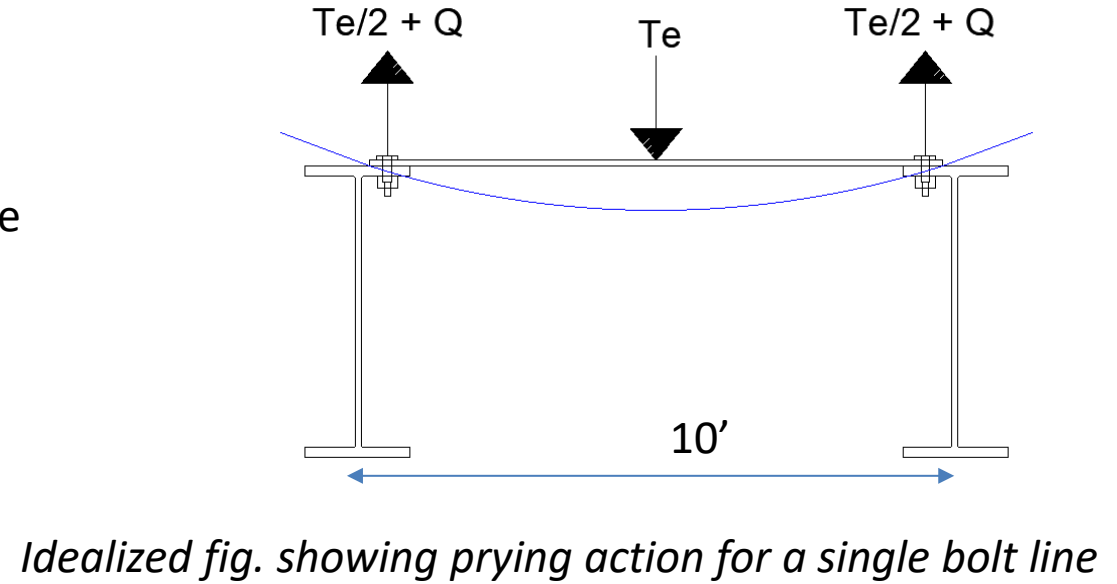
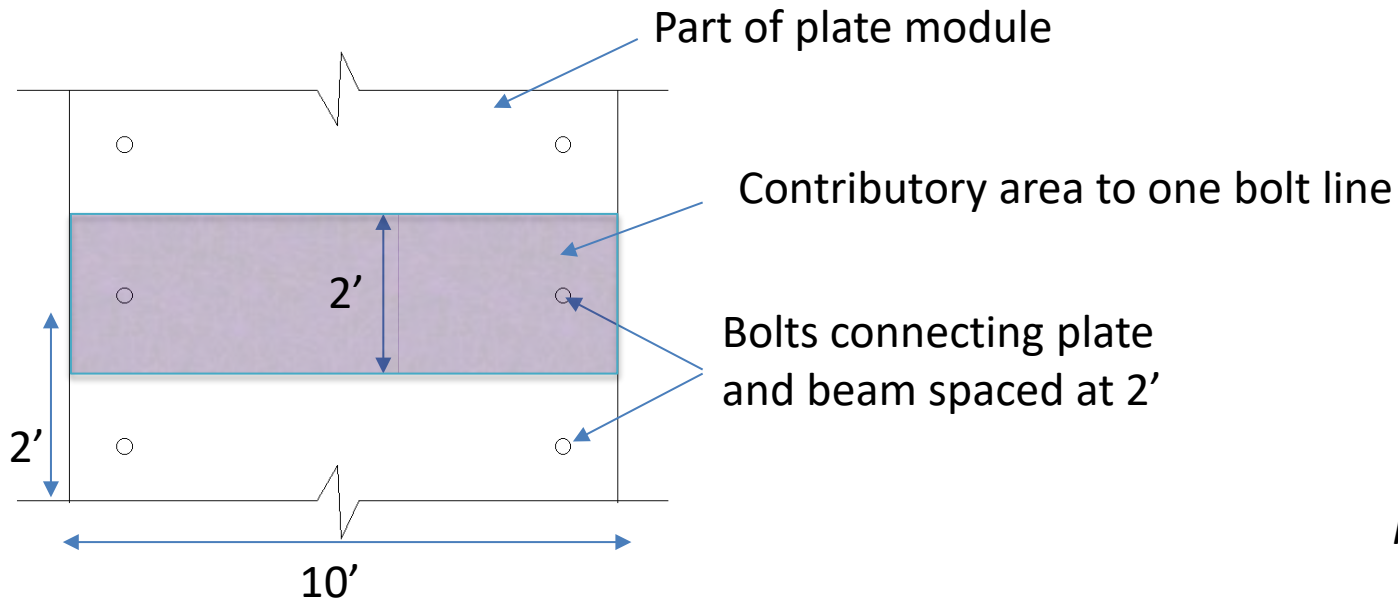
Limit states

- Load effects due to Diaphragm Shear
- Prying Action Force
- Plate Catenary Action
- Shear Buckling of Plate (design is performed)



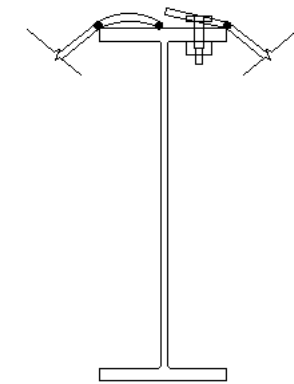
Plan view of connection

SEAM CONNECTION – PRYING ACTION FORCE

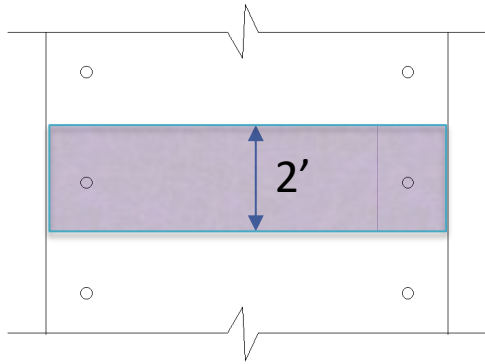


Force resolution assuming moment hinge for bolted and welded side

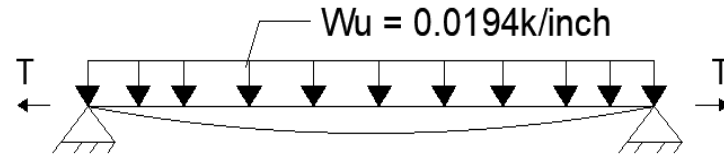
$Q_{weld} = 1.16 \text{ kips}$ $Q_{bolt} = 4.73 \text{ kips}$



SEAM CONNECTION – CATENARY ACTION

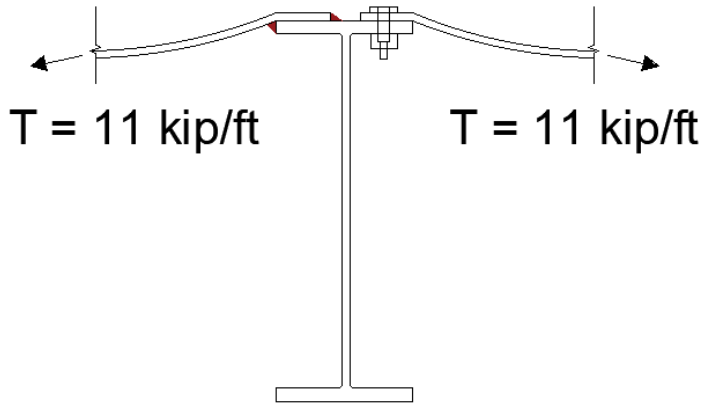


Part of the plate module

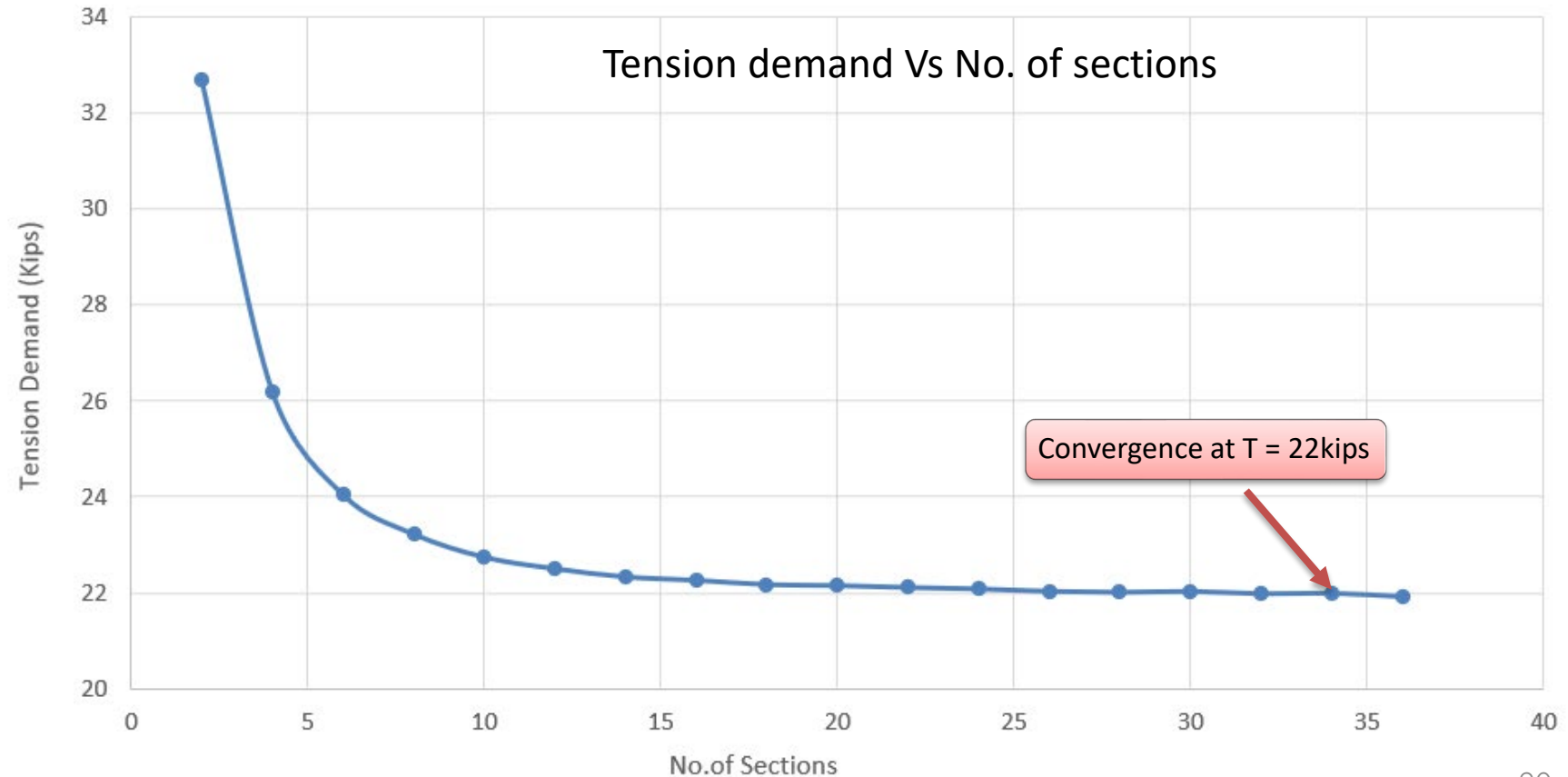


Idealized as a beam with contributory span of 2'

- Calculating Tension in the plate using Mastan



Catenary Force Demand



Attached files

- SAP2000 v25.0.0
 - 01_SCBF_4_Story_FFC_Main_Model.sdb
 - 02_SCBF_4_Story_FFC_Brace_Capcity_Model.sdb
 - 03_Diaphragm_Beam_model_Long_direction.sdb
 - 04_SCBF_4_Story_FFC_Diaphragm_Analysis.sdb
- Spreadsheet
 - FFC_All_Tables_4-Story.xlsx