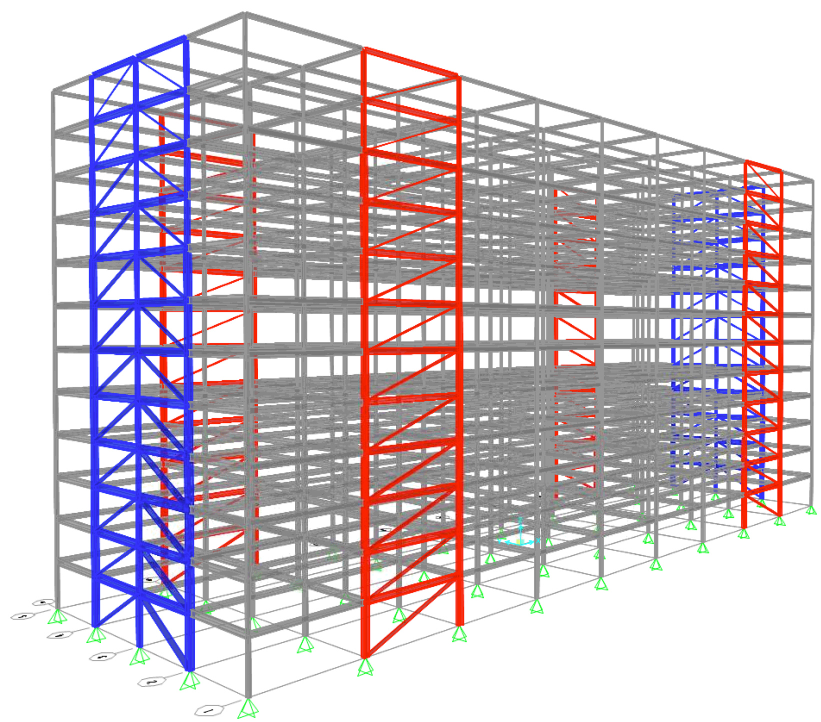
COLD-FORMED STEEL RESEARCH CONSORTIUM

SDII

Steel Diaphragm Innovation Initiative

Archetype Building Design



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24 July 2017

Archetype Buildings

The objectives of developing archetype SDII buildings include:

- Develop a series of 3D steel-framed archetype buildings that explore and document the design of horizontal LFRSs with steel deck-based diaphragms as well as vertical LFRSs and the inter-relationship between the two.
- Provide a series of buildings that form a common basis of comparison for diaphragms in steel-framed buildings much the same way the SAC buildings did for the vertical LFRS.
- Explicitly explore ASCE 7-16 standard, and ASCE 7-16 alternate diaphragm design with R_s=1 and R_s=3 in designs.
- Inform areas for needed experimentation, and create targets for advancing nonlinear analysis within the full SDII effort.

Archetype Buildings

SDII Archetypes: Steel framed buildings with steel deck-centric diaphragms

Shaded are included

GENERAL (VERTICAL) BUILDING DEFINITION

Vertical SFRS		High R, Low Ω	Middle R, Middle Ω	Low R, Middle Ω	
		SMF (R=8, Ω_o =3)	IMF (R=4.5, Ω_o =3)	OMF (R=3.5, Ω_o =3)	
		EBF (R=8, Ω_o =2.5)	SCBF (R=6, Ω_o =2)	OCBF (R=3.25, Ω_o =2)	
	⍰	BRB (R=8, Ω_o =2.5)			
Gravity Demand		High	Low		
		Related Occupancy Cat	Related Occupancy Cat		
Seismic Demand	⍰	High	Moderate		
		SDC D, Sds=1.0	SDC C, Sds=0.5		
Building Stories		1	4	8	⍰ 12
Vertical Demand Calc	⍰	ASCE 7-16 ELF	ASCE 7-16 Nonlinear time H		
Vertical Irregularities		Not considered in SDII initial archetypes (see ASCE 7 for full list)			

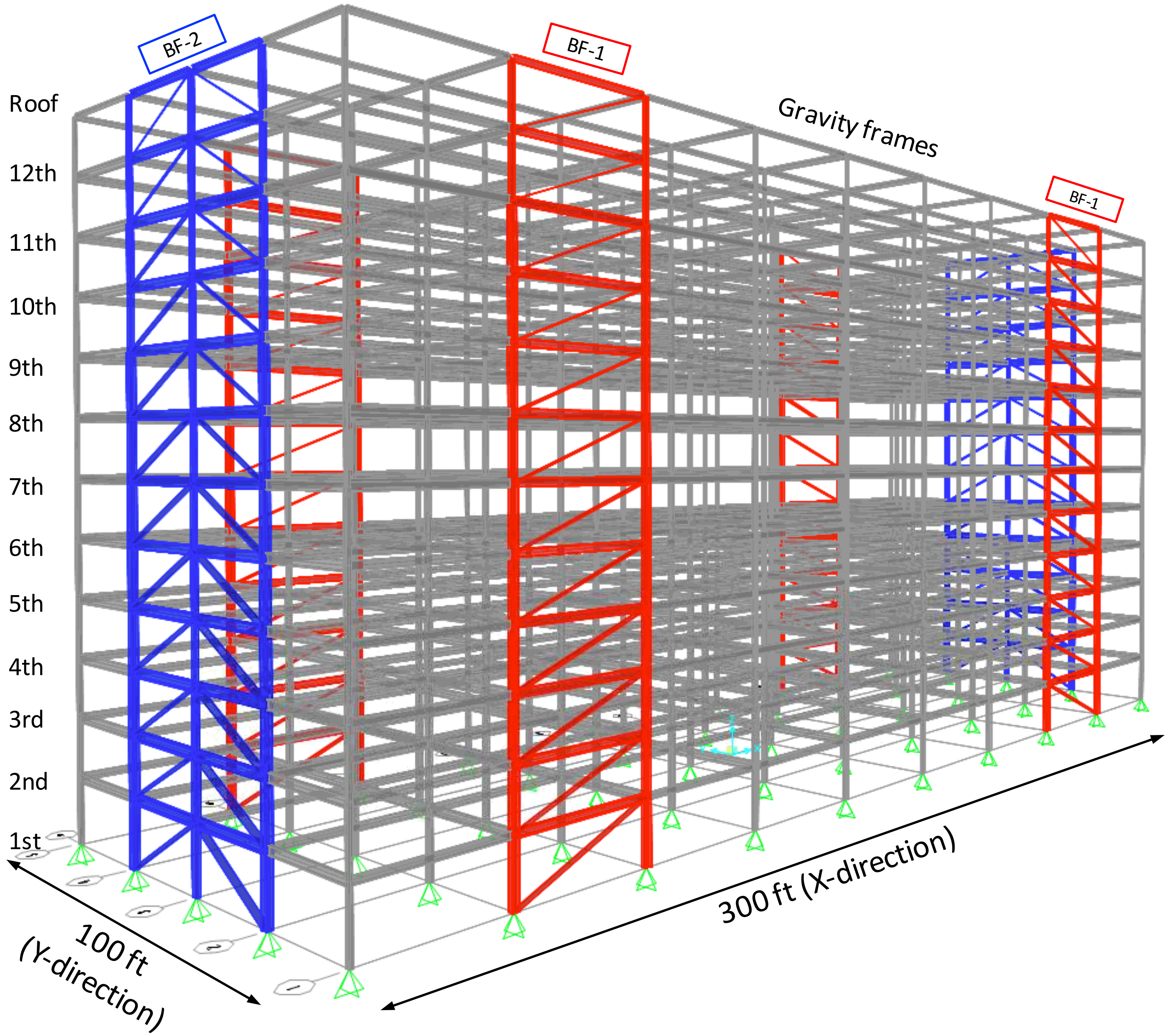
The first selected set of archetypes are buildings with 1, 4, 8, and 12 stories with different structural systems. This document is focusing on the design of the 12 story archetype building with Buckling Restrained Braced Frame (BRBF) for the vertical Seismic Force Resisting System (VSFRS).

Shaded areas in the table above are the parameters that will be included in the first set of archetypes and those indicated by ⍰ are specifically assumed in this document for the 12 story archetype example.

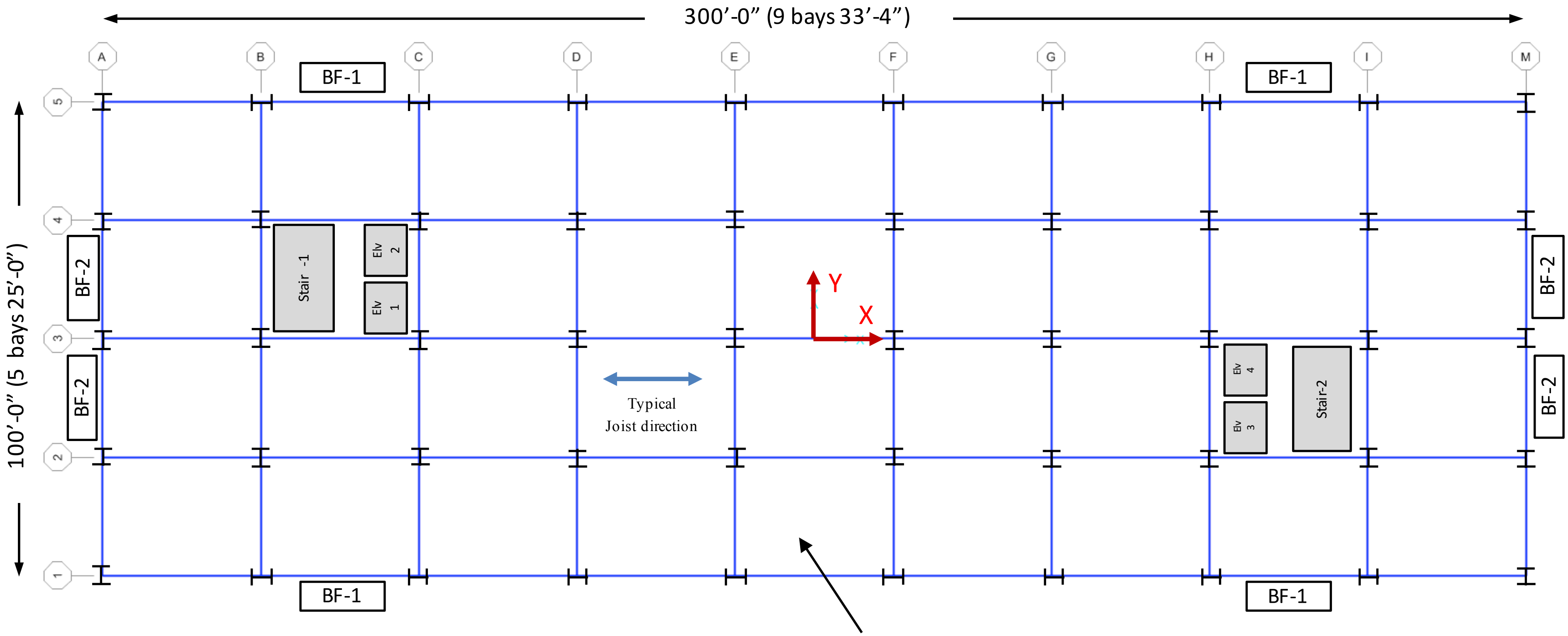
FLOOR/ROOF and DIAPHRAGM DEFINITION

Overall Floor/Roof plate	Rectangular	Complex		
		L-shaped		
		Other shapes		
		(note this also plays out in horizontal irregularities)		
Floor plate size	small	medium	large	extra large
	10,000 sf/floor	30,000 sf/floor	50,000 sf/floor	100,000 sf/floor
Floor plate aspect ratio (a:b)	High	Medium	Square	
	5:1 comp, 3:1 bare	3:1 comp., 2:1 bare	1:1	
Diaphragm spans	single (whole floor)	multi-span		
Diaphragm Demand Calc	ASCE 7-10/7-16	ASCE 7-16 Alt. R_s=1	ASCE 7-16 Alt. R_s=3	Nonlinear time history
Diaphagm Capacity Calc	AISI S310	Others		
Horizontal SFRS*	High R_s, Low Ω_s	Middle R_s, Middle Ω_s	Middle R_s, Middle Ω_s	Low R_s,Middle Ω_s
Floor	Comp. Deck	Comp. Deck	Comp. Deck	Comp. Deck
Roof	Bare Deck	Bare Deck	Comp. Deck	Comp. Deck
(* approximated, not defined in ASCE7 at this time)				
Beam supporting deck	rolled shapes	steel joists		
Horizontal Irregularity	*Out-of-Plane Offset*			
(only consider options	Transfer Floor	No Transfer Floors		
listed, see ASCE 7 for others)	Near bottom			
	Near middle			
	Near top			
	Torsion Irregularity			
	Only Accidental Torsion	1.2 to 1.4 Drift Amp	>1.4 Drift Amp	
	Diaphragm Holes (Discontinuity Irregularity)			
	No Holes	Elevator and Stair Holes	Large hole = discontinuity irr	
Diaphragm utilization (shear)	low	medium	high	
	<50% at peak demand	near 75% at peak demand	near 100% at peak	
Diaphragm stiffness	flexible	semi-rigid	rigid	
(depends on the design)	bare deck?	comp. deck?	comp. deck?	
		bare deck?		
Diaphragm Design	Zones based on demands	Single design for whole floor		
Collector design	Capacity Protected 1	Capacity Protected 2	Unprotected	
(Full length)	Based on VLFRS (ASCE 7-16)	Generic Ω	No Ω (ASCE 7-10)	
ADDITIONAL CONSIDERATIONS				
Input Sensitivities	1D motion input	2D motion input	3D motion input	

Project Information (iso view)

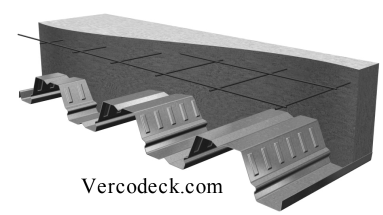


Project Information (typical plan)

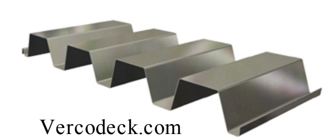


Building Dimensions		
Width (E-W)	100.00	(ft)
Length (N-S)	300.00	(ft)
H (First story)	14.00	(ft)
H (Typical story)	12.50	(ft)
Bay size X	33.33	(ft)
Bay size Y	25.00	(ft)
Parapet	3.00	(ft)
Number of Stories	12	(1, 4, 8,12)
H (total height)	151.5	(ft)

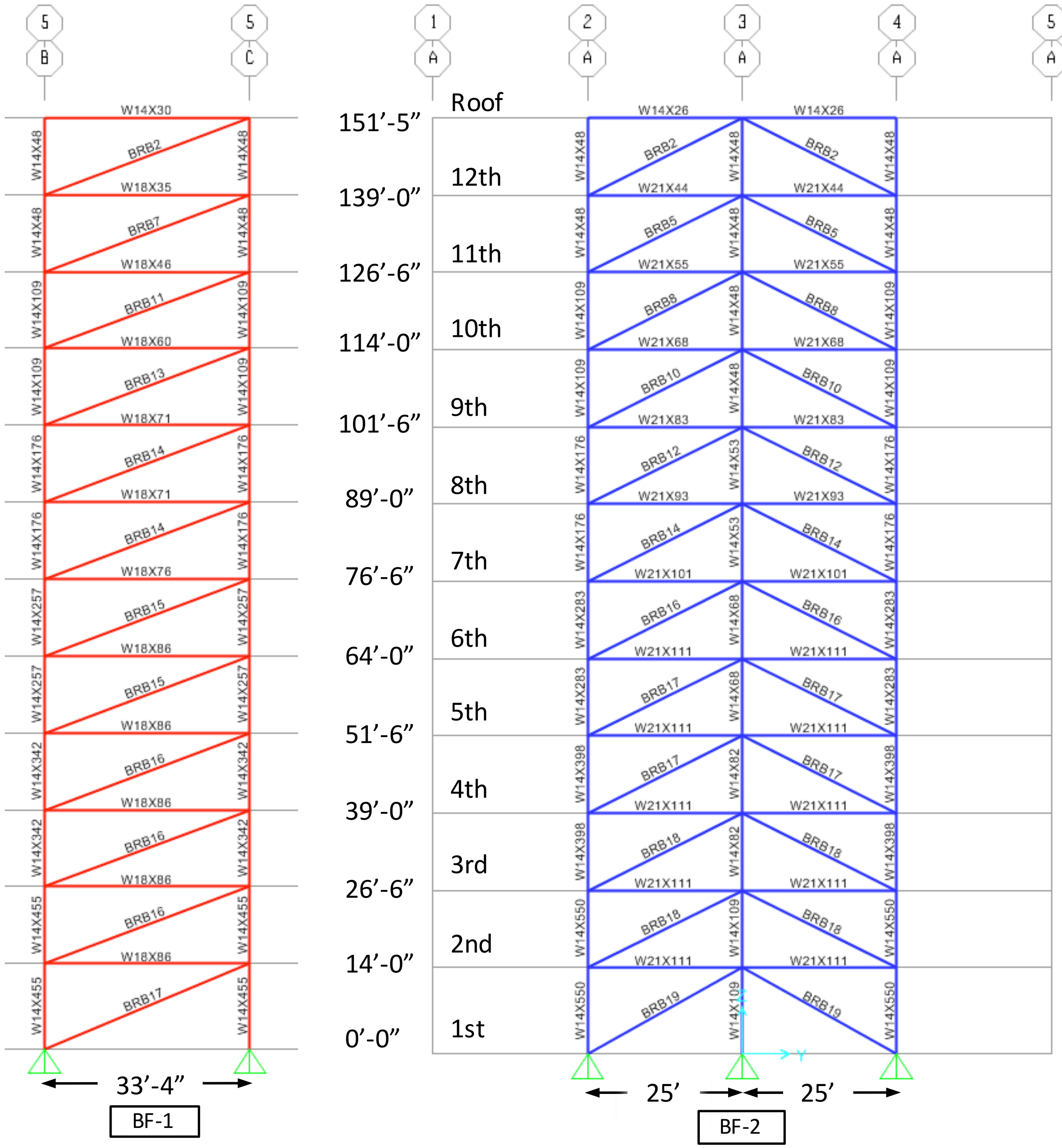
Typical floors:
7.5" total slab deck,
Normal weight concrete,
2 Hours fire rating



Roof:
Bare Steel deck



Project Information (Brace frames)



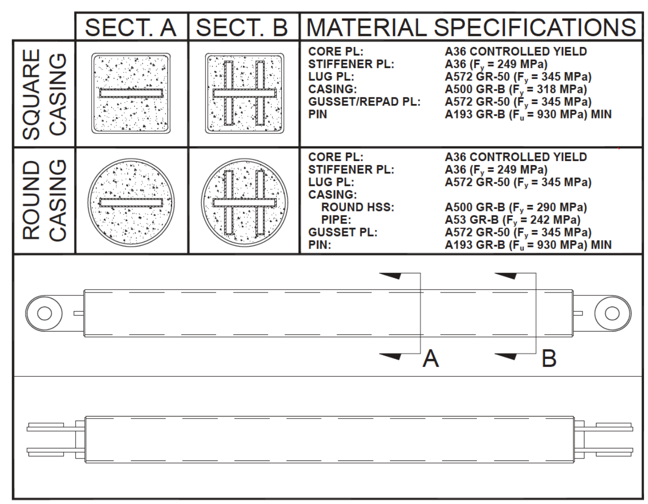
Seismic force-resisting system (SFRS)

BRBF (buckling-restrained braced frame)
Diagonal bracing (Same direction)

BRB x → A_{sc} = x in^2 (Steel core area)

Example: BRB15 → A_{sc} = 15 in^2

Steel buckling-restrained braced frames	
R	8
Ω_0	2.5
C_d	5
Max Height for SDC=D	160 ft



CoerBrace Pinned BRBs

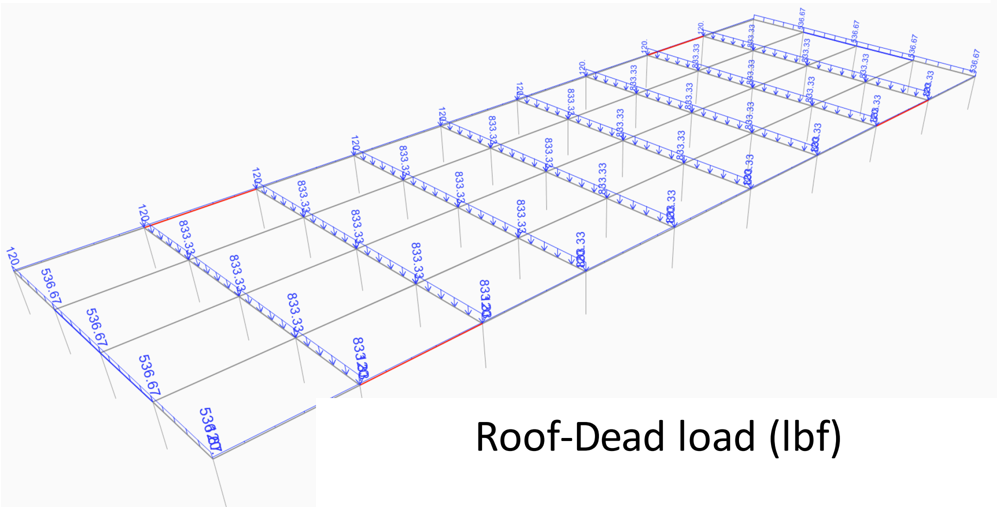
Gravity Load and Mass

Diaphragm/Deck (Typical floors)			
Concrete	4.5	(in)	2HR fire rating
Steel deck	3	(in.)	Floor weight (Verco Catalog) = 72.5 psf
Steel deck weight	3	(psf)	
Total height	7.5	(in.)	
Concrete unit Weight	145	(pcf)	normal weight concrete

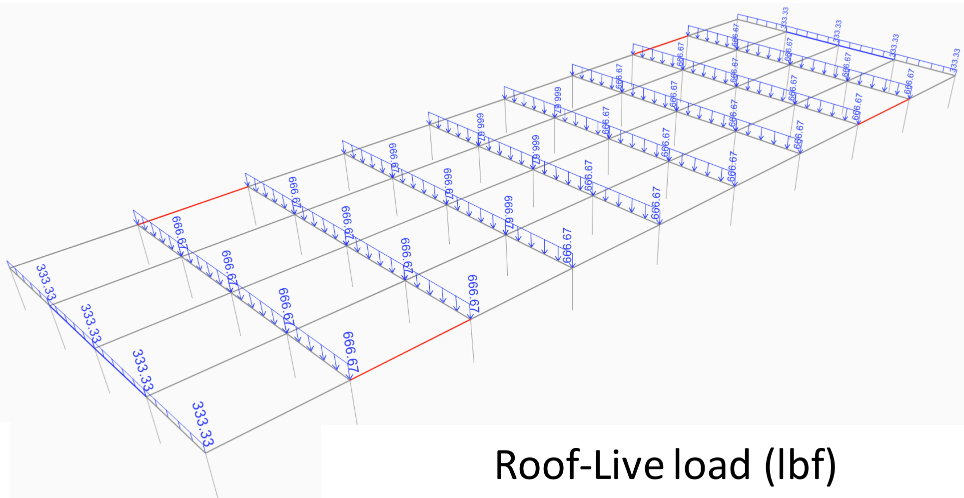
Roof	3	(psf)	Bare steel deck
Roofing, mechanical, beams	22	(psf)	(uniform on the roof)
Roof unit weight	**25**	(psf)	
Roof Live load	**20**	(psf)	
Floor (concrete and deck)	75.5	(psf)	
Super-imposed Dead	10	(psf)	(uniform on the floor)
Floor unit weight	**85.5**	(psf)	
Floor Live load	**50**	(psf)	
External Walls	**40**	(psf)	(external wall unit weight)
Partitions (Int. Walls)	**15**	(psf)	(uniform on the floor)

Level	Assembly	Unit Wt (psf)	Area (ft^2)	Weight (kips)	Story Wt (kips)
	Roof	25	50000	1250.0	
Roof	Ext wall	40	11100	444.0	2069.0
	Int. wall	15	25000	375.0	
	Floor	85.5	50000	4275.0	
Typical floor	Ext wall	40	15000	600.0	5625.0
	Int. wall	15	50000	750.0	

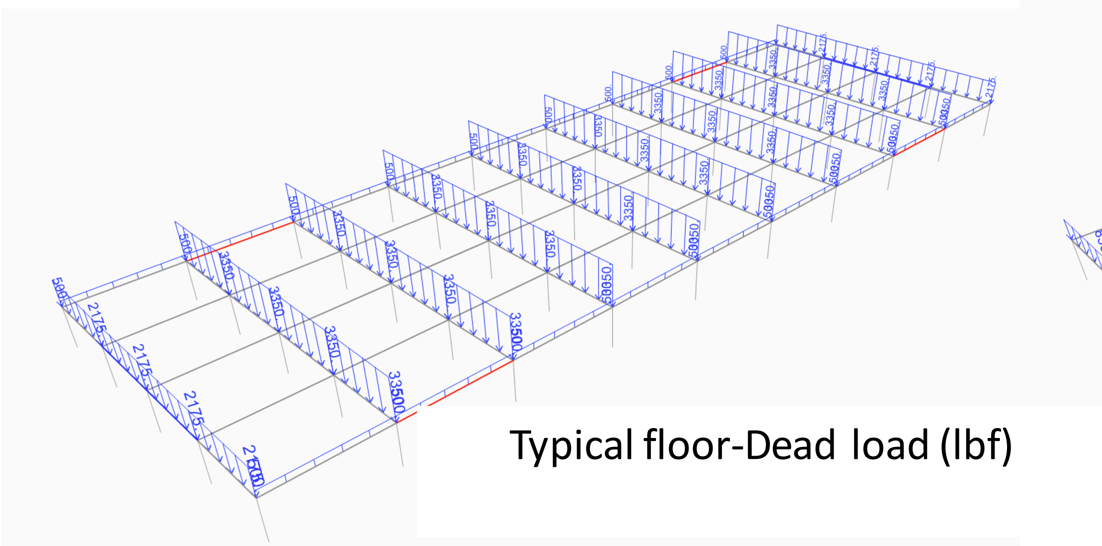
Gravity loading of the beams (line loads in the model)



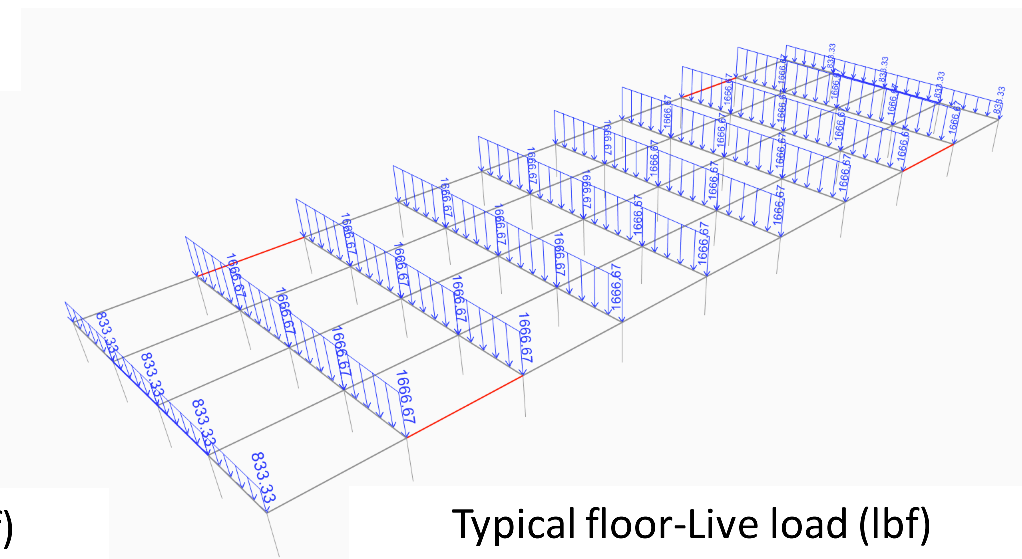
Roof-Dead load (lbf)



Roof-Live load (lbf)



Typical floor-Dead load (lbf)



Typical floor-Live load (lbf)

Material properties

Material	Designation
Columns and beams	ASTM A992 (F_y /F_u = 50/65 ksi, E=29000 ksi)
Diaphragm (Normal Weight Concrete)	f'c=3000 psi
Composite Deck (Floors)	Galvanized deck, ASTM A653 or ASTM 1063 F_y /F_u = 50/60 ksi, E=29500 ksi
Bare Deck (Roof)	Galvanized deck, ASTM A653 or ASTM 1063 F_y /F_u = 33/45 ksi, F_y /F_u = 50/65 ksi, E=29500 ksi

Material (BRBF)	Designation (CoreBrace Specifications)
BRB Core Steel (CoreBrace)	ASTM A36, F_{ysc} =42 ksi (±4ksi). F_{ysc} (min)=38 ksi
BRB Stiffener Plate (CoreBrace)	ASTM A36 (F_y = 36 ksi)
BRB Lug Plate	ASTM A572 GR-50 (F_y = 50 ksi)
BRB Casing	ASTM A500 GR-B (F_y = 46 ksi)
BRB Gusset and Repad plates	ASTM A572 GR-50 (F_y = 50 ksi)
BRB PIN	ASTM A193 GR-B (F_y = 138 ksi)
BRB Weld Electrodes	E70-XX (notch toughness: 20ft-lb at -20 degrees F)

Seismicity and Design Response Spectrum

Spectral parameters		ASCE 7 References
Site Coordinates	33.67068°N, 117.8227°W (Irvine, CA)	
Risk Category	II	Residential
I_e =	1.00	
S_s =	1.545	
S_1 =	0.569	
F_a =	1.0	(Table 11.4-1)
F_v =	1.5	(Table 11.4-2)
Site Class	D	Chapter-20
$S_{MS} = F_a S_S$ =	1.545	(Eq. 11.4-1)
$S_{M1} = F_v S_1$ =	0.8535	(Eq. 11.4-2)
S_{DS} = 2/3 S_{MS} =	1.030	(Eq. 11.4-3)
S_{D1} = 2/3 S_{M1} =	0.569	(Eq. 11.4-4)
T_0=	0.110	
T_s=	0.552	
T_L =	8	(Fig. 22-12)

Design Maps Summary Report

USGS Design Maps Summary Report

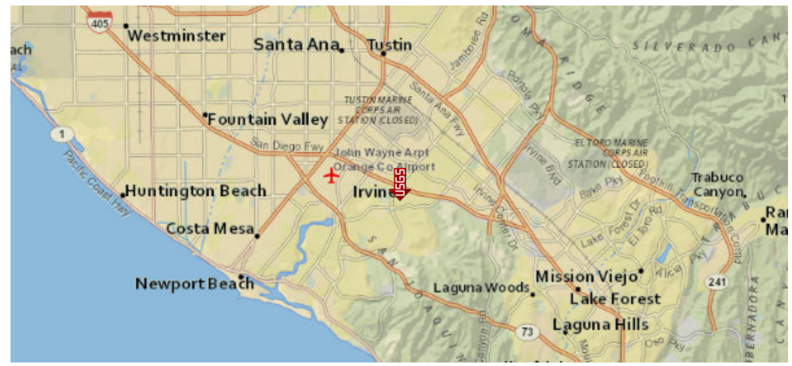
User–Specified Input

Building Code Reference Document 2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)

Site Coordinates 33.6707°N, 117.8227°W

Site Soil Classification Site Class D – "Stiff Soil"

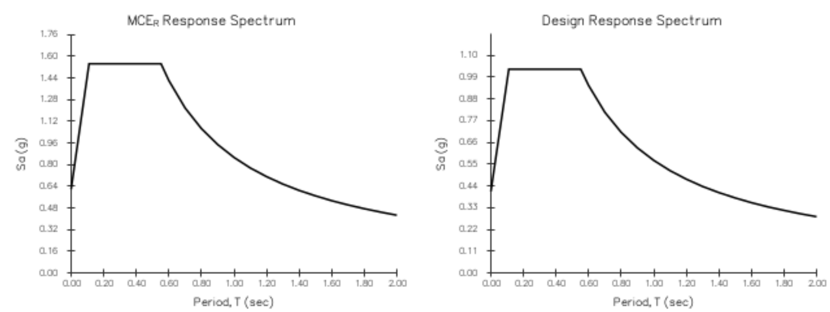
Risk Category I/II/III



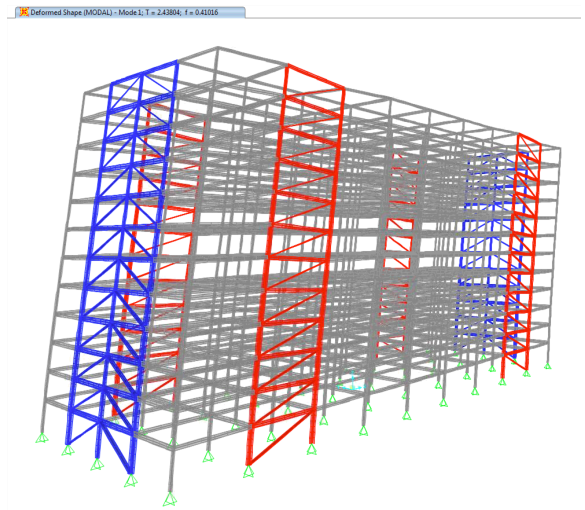
USGS–Provided Output

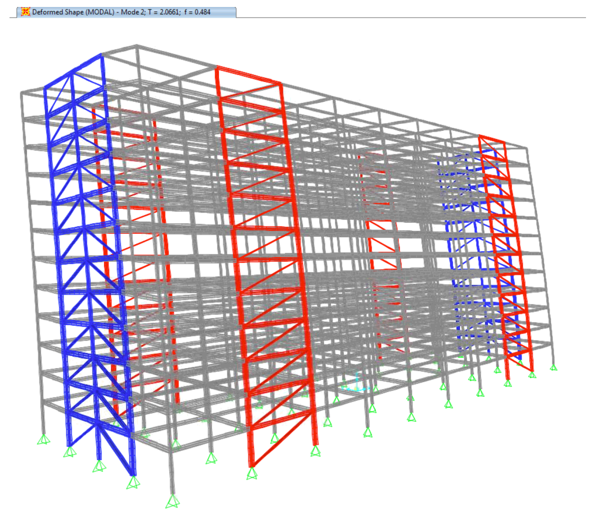
S_S =	1.545 g	S_{MS} =	1.545 g	S_{DS} =	1.030 g
S_1 =	0.569 g	S_{M1} =	0.853 g	S_{D1} =	0.569 g

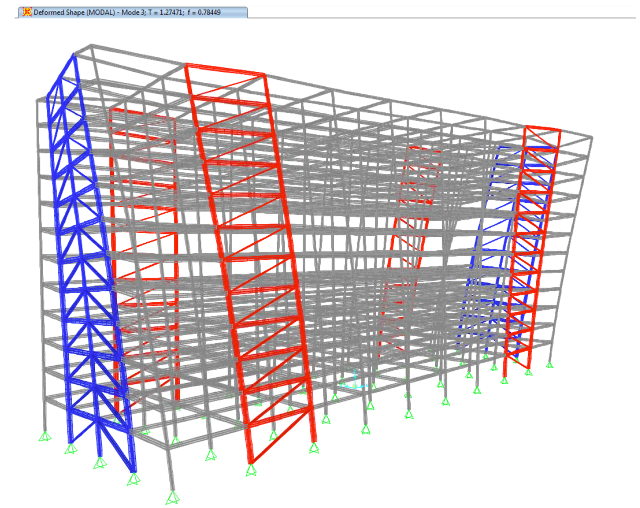
For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



Building Period



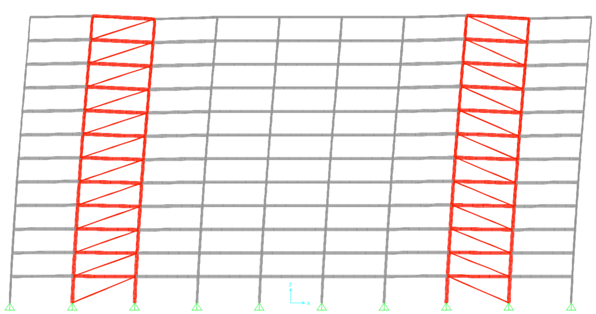


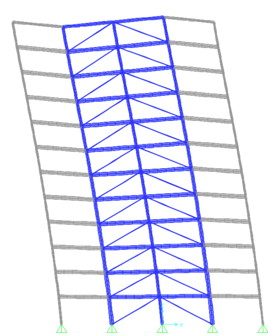


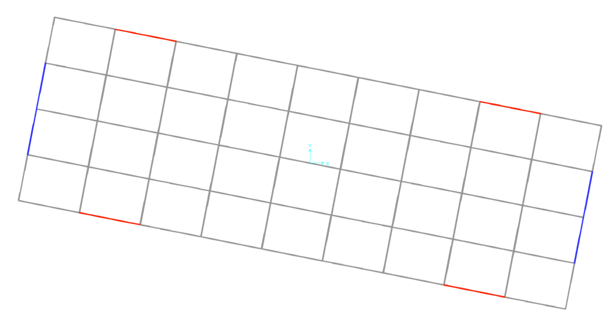
Mode-1:T=2.43 sec.
(longitudinal, X-direction)

Mode-2:T=2.07 sec.
(Transverse, Y-direction)

Mode-3:T=1.28 sec.
(torsional)







Seismic forces

Building Period		References
C_u	1.4	Table 12.8.1
C_t	0.03	Table 12.8.2
x	0.75	Table 12.8.2
T_a	1.295 sec.	Eq. 12.8.7
C_uT_a	1.814 sec.	upper limit
T_{model} (mode 1 –X direction)	2.43 sec.	$>C_uT_a$ *
T_{model} (mode 2 –Y direction)	2.07sec.	$>C_uT_a$ *
T (used for Seismic force in both directions)	1.814	C_uT_a governs

Seismic response coefficients		
C_s	0.12875	Eq. 12.8-2
but should not exceed	0.03922	Eq. 12.8-3 ($T<T_L$=8 sec.)
C_s should not be less than	0.04532	Eq. 12.8-5 (C_s=0.044.I.S_{DS}>0.01)**

Seismic response coefficient and base shear		
$V = C_sW$		Eq. 12.8.1
C_s	**0.04532**	
W	38836 kips	
V	**1760 kips**	

* The computed fundamental period of the structure without the upper limit (C_uT_a) specified in Section 12.8.2 can be used for controlling drift, but the upper limit of C_uT_a is assumed for the period here.
** Eq. 12.8-5 need not be considered for computing drift.

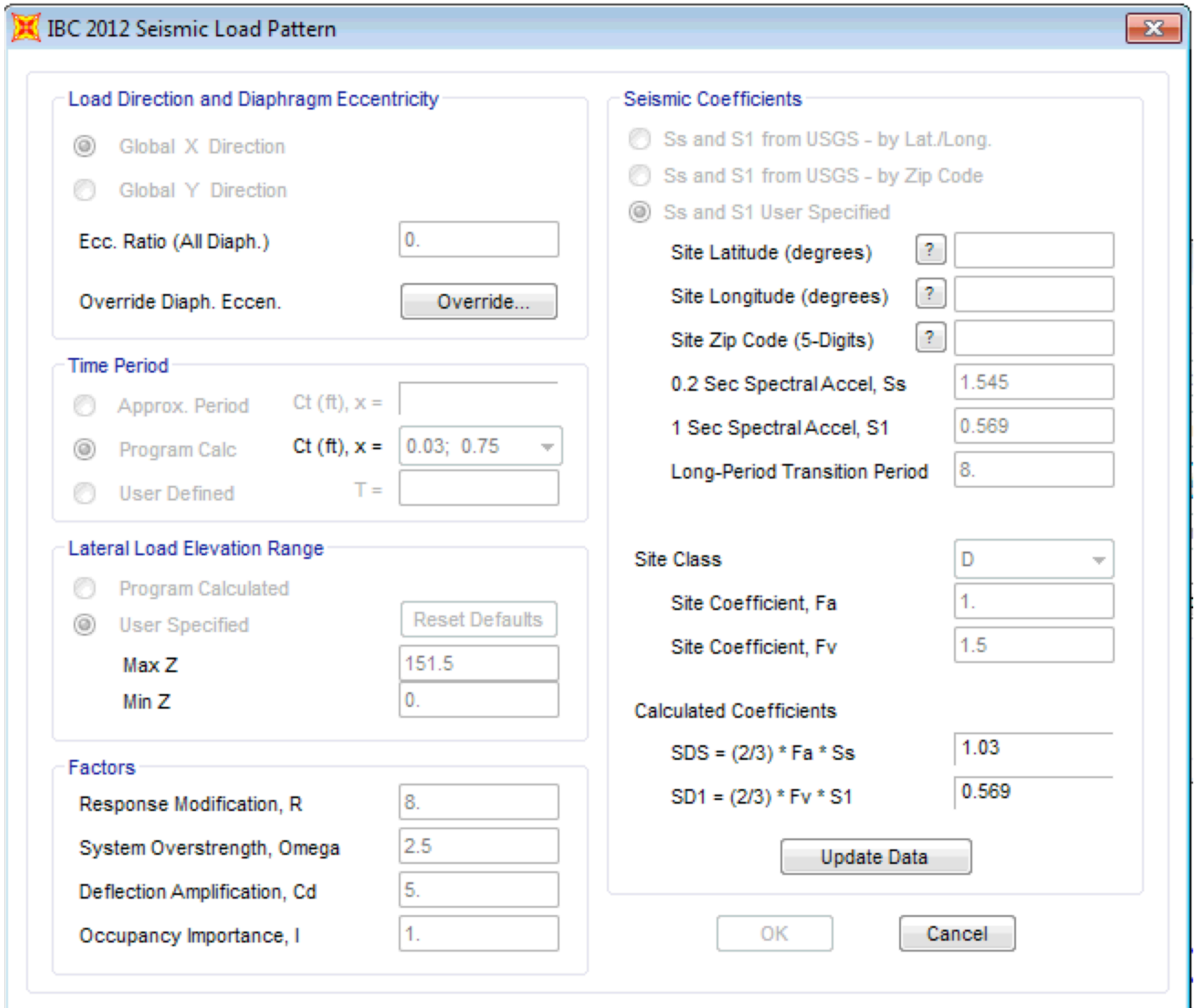
Seismic Forces (vertical distribution)

Level	**w_x (kips)**	**h_x (ft)**	**$w_x h_x^k$**	**C_{vx}**	**F_x (kips)**
Roof	1271	151.50	5208729	0.084	**147.39**
12th	3415	139.00	12134303	0.195	**343.35**
11th	3415	126.50	10380303	0.167	**293.72**
10th	3415	114.00	8736647	0.140	**247.21**
9th	3415	101.50	7207356	0.116	**203.94**
8th	3415	89.00	5797094	0.093	**164.03**
7th	3415	76.50	4511374	0.073	**127.65**
6th	3415	64.00	3356872	0.054	**94.99**
5th	3415	51.50	2341934	0.038	**66.27**
4th	3415	39.00	1477488	0.024	**41.81**
3rd	3415	26.50	778889	0.013	**22.04**
2nd	3415	14.00	270606	0.004	**7.66**
Total	38836		62201595		

Vertical Distribution (12.8.3)

k = 1.657 Sec-12-8-3

Seismic Forces (SAP2000 v19.0.0)



SAP2000 IBC-2012 Seismic Load Pattern Matches
the Seismic forces in the design spreadsheet

Computer Model Assumptions

- Roof and floor live load are modeled as floor live load for simplicity.
- Live Load Reduction Factors (LLRF) are calculated and implemented in the computer program. LLRF is 0.4 for all columns in the 1st to 10th story, 0.43 for 11th story, and 0.51 for the last story. LLRF is calculated 0.51 for all beams.
- 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.
- Beams are modeled as pin-ended frame elements, and columns are continuous from base to the roof. 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

- To estimate the actual stiffness of the BRBs, an axial stiffness factor (KF) is consider in the model as a “Frame Property Modifier”.
- 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.
- SAP applies the seismic force to the center of mass. The accidental eccentricity (5%) is applied by moving the center of mass : e_x=.05x300=15’, and e_y=.05x100=5’.
- The accidental eccentricity (5%) is not applied to the roof.
- Frame columns are modeled as pinned at their bases.
- P-Δ is considered via frame geometric stiffness under gravity loads.

Design Story Drifts

Ex+5%eccentricity

X-Direction	Story height (ft)	Elastic deformation (ft)	Design story 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
12	12.5	0.048	0.238	0.25	0.38	1.90	2.0
11	12.5	0.047	0.234	0.25	0.37	1.87	2.0
10	12.5	0.046	0.231	0.25	0.37	1.85	2.0
9	12.5	0.047	0.235	0.25	0.38	1.88	2.0
8	12.5	0.048	0.238	0.25	0.38	1.91	2.0
7	12.5	0.048	0.242	0.25	0.39	1.94	2.0
6	12.5	0.047	0.234	0.25	0.37	1.87	2.0
5	12.5	0.045	0.226	0.25	0.36	1.81	2.0
4	12.5	0.042	0.209	0.25	0.34	1.68	2.0
3	12.5	0.039	0.196	0.25	0.31	1.57	2.0
2	12.5	0.036	0.178	0.25	0.28	1.42	2.0
1	14	0.033	0.167	0.28	0.24	1.19	2.0

Ey+5%eccentricity

Y-Direction	Story height (ft)	Elastic deformation (ft)	Design story 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
12	12.5	0.036	0.180	0.25	0.29	1.44	2.0
11	12.5	0.041	0.207	0.25	0.33	1.65	2.0
10	12.5	0.041	0.207	0.25	0.33	1.66	2.0
9	12.5	0.041	0.207	0.25	0.33	1.65	2.0
8	12.5	0.040	0.200	0.25	0.32	1.60	2.0
7	12.5	0.037	0.186	0.25	0.30	1.49	2.0
6	12.5	0.034	0.172	0.25	0.27	1.37	2.0
5	12.5	0.032	0.160	0.25	0.26	1.28	2.0
4	12.5	0.031	0.153	0.25	0.24	1.22	2.0
3	12.5	0.028	0.138	0.25	0.22	1.10	2.0
2	12.5	0.025	0.125	0.25	0.20	1.00	2.0
1	14	0.024	0.118	0.28	0.17	0.84	2.0

Amplification of Accidental Torsional Moment

Story	Max. X-displacement (ft) $\delta_{x\text{-}max}$	Avg. X- disp. (ft) $\delta_{x\text{-}avg}$	Torsional amplification factor A_x	Max. Y-displacement (ft) $\delta_{y\text{-}max}$	Avg. Y- disp. (ft) $\delta_{y\text{-}avg}$	Torsional amplification factor A_y
12	0.526	0.523	1.00	0.410	0.384	1.00
11	0.478	0.475	1.00	0.374	0.349	1.00
10	0.431	0.429	1.00	0.333	0.309	1.00
9	0.385	0.383	1.00	0.292	0.270	1.00
8	0.338	0.336	1.00	0.250	0.232	1.00
7	0.290	0.289	1.00	0.210	0.195	1.00
6	0.242	0.241	1.00	0.173	0.160	1.00
5	0.195	0.194	1.00	0.139	0.128	1.00
4	0.150	0.149	1.00	0.107	0.099	1.00
3	0.108	0.108	1.00	0.076	0.070	1.00
2	0.069	0.069	1.00	0.049	0.045	1.00
1	0.033	0.033	1.00	0.024	0.022	1.00

Amplification is not required in X or Y directions

P-Delta Effects

P-Delta effect controls for the X-direction, which is more critical.

C_d= 5

I_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}	$\theta<\theta_{max}$
12	12.5	1271	0.238	147.4	0.033	0.100	Passed
11	12.5	4686	0.234	490.7	0.036	0.100	Passed
10	12.5	8101	0.231	784.5	0.038	0.100	Passed
9	12.5	11516	0.235	1031.7	0.042	0.100	Passed
8	12.5	14931	0.238	1235.6	0.046	0.100	Passed
7	12.5	18346	0.242	1399.6	0.051	0.100	Passed
6	12.5	21761	0.234	1527.3	0.053	0.100	Passed
5	12.5	25176	0.226	1622.3	0.056	0.100	Passed
4	12.5	28591	0.209	1688.5	0.057	0.100	Passed
3	12.5	32006	0.196	1730.4	0.058	0.100	Passed
2	12.5	35421	0.178	1752.4	0.058	0.100	Passed
1	14	38836	0.167	1760.0	0.053	0.100	Passed

Design per AISC 341-10/16 and AISC 360-10

Designed here:

- Gravity frames (Composite and non-composite joists, girders, and columns)
- Buckling Restrained Braces (BRBs)
- Columns of the Buckling Restrained Braced Frame (BRBF)
- Beams of the BRBF
- Diaphragm, chord and collectors

Not designed here:

- 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 deigned per AISC 360-10 and ASIC 341-10/16.
- Size of the members in the model are determined after several iterations 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.
  - *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 braces are assumed to be 1.0 for both minor axis and major axis flexural buckling, $K_{minor}=K_{major}=1.0$.
- All gravity beams and girders of the concrete floors are manually designed as composite beams. Beam cross-sections in the model match the beam sizes in the composite design.
- All gravity beams and girders of the bare deck roof are designed manually as non-composite beams. Beam cross-sections in the model match the beam sizes in the manual design.
- 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

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

(Beams and columns)

UDSTL1: 1.4D
UDSTL2: 1.2D+1.6L
UDSTL3: $1.406D+0.5L+E_x$
UDSTL4: $1.406D+0.5L-E_x$
UDSTL5: $1.406D+0.5L+E_y$
UDSTL6: $1.406D+0.5L-E_y$
UDSTL7: $1.406D+0.5L+E_{x+5\%}$
UDSTL8: $1.406D+0.5L-E_{x+5\%}$
UDSTL9: $1.406D+0.5L+E_{x-5\%}$
UDSTL10: $1.406D+0.5L-E_{x-5\%}$
UDSTL11: $1.406D+0.5L+E_{y+5\%}$
UDSTL12: $1.406D+0.5L-E_{y+5\%}$
UDSTL13: $1.406D+0.5L+E_{y-5\%}$
UDSTL14: $1.406D+0.5L-E_{y-5\%}$
UDSTL15: $0.694D+E_x$
UDSTL16: $0.694D-E_x$
UDSTL17: $0.694D+E_y$
UDSTL18: $0.694D-E_y$
UDSTL19: $0.694D+E_{x+5\%}$
UDSTL20: $0.694D-E_{x+5\%}$
UDSTL21: $0.694D+E_{x-5\%}$
UDSTL22: $0.694D-E_{x-5\%}$
UDSTL23: $0.694D+E_{y+5\%}$
UDSTL24: $0.694D-E_{y+5\%}$
UDSTL25: $0.694D+E_{y-5\%}$
UDSTL26: $0.694D-E_{y-5\%}$

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

(Amplified load combinations for bracing columns and collectors)

AUDSTL3: $1.406D+0.5L+2.5E_x$
AUDSTL4: $1.406D+0.5L-2.5E_x$
AUDSTL5: $1.406D+0.5L+2.5E_y$
AUDSTL6: $1.406D+0.5L-2.5E_y$
AUDSTL7: $1.406D+0.5L+2.5E_{x+5\%}$
AUDSTL8: $1.406D+0.5L-2.5E_{x+5\%}$
AUDSTL9: $1.406D+0.5L+2.5E_{x-5\%}$
AUDSTL10: $1.406D+0.5L-2.5E_{x-5\%}$
AUDSTL11: $1.406D+0.5L+2.5E_{y+5\%}$
AUDSTL12: $1.406D+0.5L-2.5E_{y+5\%}$
AUDSTL13: $1.406D+0.5L+2.5E_{y-5\%}$
AUDSTL14: $1.406D+0.5L-2.5E_{y-5\%}$
AUDSTL15: $0.694D+2.5E_x$
AUDSTL16: $0.694D-2.5E_x$
AUDSTL17: $0.694D+2.5E_y$
AUDSTL18: $0.694D-2.5E_y$
AUDSTL19: $0.694D+2.5E_{x+5\%}$
AUDSTL20: $0.694D-2.5E_{x+5\%}$
AUDSTL21: $0.694D+2.5E_{x-5\%}$
AUDSTL22: $0.694D-2.5E_{x-5\%}$
AUDSTL23: $0.694D+2.5E_{y+5\%}$
AUDSTL24: $0.694D-2.5E_{y+5\%}$
AUDSTL25: $0.694D+2.5E_{y-5\%}$
AUDSTL26: $0.694D-2.5E_{y-5\%}$

(BRB Design-no gravity)

C1: E_x
C2: E_y
C3: $E_{x+5\%}$
C4: $E_{x-5\%}$
C5: $E_{y+5\%}$
C6: $E_{y-5\%}$

AISC 341-10/16 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

- Details shall be provided to transfer loads between the diaphragm and boundary members, collector elements, and elements of the horizontal framing system.
- The nominal in-plane shear strength of composite diaphragms and concrete slab on steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with ACI 318 excluding Chapter 14. Alternatively, the composite diaphragm nominal shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

AISC 341-10/16 requirements for BRBFs (summary of the applicable provisions)

Section F4.2

- BRBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through brace yielding in tension and compression.
- Expected deformations are those corresponding to a story drift of at least 2% of the story height or two times the design story drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading.

Section F4.2a

- Brace connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength. The adjusted brace strength in compression shall be $\beta\omega R_y P_{ysc}$. The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$. β and ω shall be determined per F4.2b.

Section F4.3

- The required strength of columns, beams, struts and connections in BRBF shall be determined using the capacity-limited seismic load effect including the adjusted strength in compression or in tension.
- Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.

AISC 341-10/16 requirements for BRBFs (summary of the applicable provisions)

Section F4.3

- The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Section K3.

Section F4.4c

- Where the compression strength adjustment factor, β, as determined in Section F4.2b exceeds 1.3, at least 30%, but no more than 70%, of the total horizontal force along that line is resisted by braces in tension.

Section F4.5a

- Beams and columns shall satisfy the requirements of Section D1.1 for <u>moderately ductile</u> members.

Section F4.5b2

- The brace design axial strength, ϕP_{ysc} (LRFD) in tension and compression, in accordance with the limit state of yielding, shall be determined as $P_{ysc} = F_{ysc}A_{sc}$ (ϕ=0.9)

*Section F4.*5c.

- The protected zone shall include the steel core of braces and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section D1.3.

Width-to-thickness ratio per Table D1.1

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

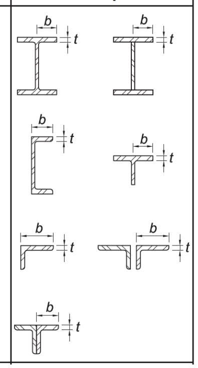
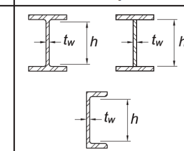
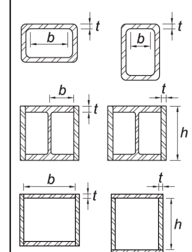
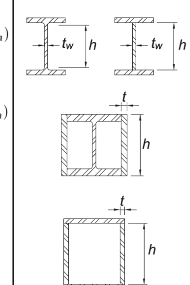
	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double-angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.32\sqrt{\frac{E}{R_yF_y}}$	$0.40\sqrt{\frac{E}{R_yF_y}}$	

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

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Stiffened Elements	Webs of rolled or built-up I shaped sections and channels used as diagonal braces	h/t_w	$1.57\sqrt{\frac{E}{R_yF_y}}$	$1.57\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams or columns as flanges in uniform compression due to axial, flexure, or combined axial and flexure: 1) Walls of rectangular HSS 2) Flanges and side plates of boxed I-shaped sections, webs and flanges of built-up box shapes	b/t h/t	$0.65\sqrt{\frac{E}{R_yF_y}}$	$1.18\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: 1) Webs of rolled or built-up I-shaped sections or channels [b] 2) Side plates of boxed I-shaped sections 3) Webs of built-up box sections	h/t_w h/t h/t	For $C_a \le 0.114$ $2.57\sqrt{\frac{E}{R_yF_y}}(1-1.04C_a)$ For $C_a > 0.114$ $0.88\sqrt{\frac{E}{R_yF_y}}(2.68-C_a)$ $\ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a=\frac{P_u}{\phi_c P_y}$ (LRFD) $C_a=\frac{\Omega_c P_a}{P_y}$ (ASD) $P_y=R_yF_yA_g$	For $C_a \le 0.114$ $3.96\sqrt{\frac{E}{R_yF_y}}(1-3.04C_a)$ For $C_a > 0.114$ $1.29\sqrt{\frac{E}{R_yF_y}}(2.12-C_a)$ $\ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a=\frac{P_u}{\phi_c P_y}$ (LRFD) $C_a=\frac{\Omega_c P_a}{P_y}$ (ASD) $P_y=R_yF_yA_g$	

Width-to-thickness ratio of all Beams and columns shall satisfy the requirements of Section D1.1 for moderately 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 $C_a=P_u/(\phi_c R_y F_y A_g)$

Width-to-thickness ratio of the columns

			Tabe 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**
11,12	BF-1	W14X48*	50	29000	1.1	36.05	9.18	33.6	Passed	6.75	Passed
9,10	BF-1	W14X109*	50	29000	1.1	36.05	9.18	21.7	Passed	8.49	Passed
7,8	BF-1	W14X176	50	29000	1.1	36.05	9.18	13.7	Passed	5.97	Passed
5,6	BF-1	W14x257	50	29000	1.1	36.05	9.18	9.7	Passed	4.23	Passed
3,4	BF-1	W14x342	50	29000	1.1	36.05	9.18	7.4	Passed	3.31	Passed
1,2	BF-1	W14x455	50	29000	1.1	36.05	9.18	5.7	Passed	2.62	Passed
11,12	BF-2	W14X48*	50	29000	1.1	36.05	9.18	33.6	Passed	6.75	Passed
9,10	BF-2	W14X109*	50	29000	1.1	36.05	9.18	21.7	Passed	8.49	Passed
7,8	BF-2	W14X176	50	29000	1.1	36.05	9.18	13.7	Passed	5.97	Passed
5,6	BF-2	W14x283	50	29000	1.1	36.05	9.18	8.8	Passed	3.89	Passed
3,4	BF-2	W14x398	50	29000	1.1	36.05	9.18	6.4	Passed	2.92	Passed
1,2	BF-2	W14x550	50	29000	1.1	36.05	9.18	4.8	Passed	2.25	Passed
11,12	BF-2	W14X48*	50	29000	1.1	36.05	9.18	33.6	Passed	6.75	Passed
9,10	BF-2	W14X48*	50	29000	1.1	36.05	9.18	33.6	Passed	6.75	Passed
7,8	BF-2	W14X53	50	29000	1.1	36.05	9.18	30.9	Passed	6.11	Passed
5,6	BF-2	W14x68	50	29000	1.1	36.05	9.18	27.5	Passed	6.97	Passed
3,4	BF-2	W14x82	50	29000	1.1	36.05	9.18	22.4	Passed	5.92	Passed
1,2	BF-2	W14x109	50	29000	1.1	36.05	9.18	21.7	Passed	8.49	Passed

* column cross-section is changed to satisfy width-to-thickness requirements for moderately ductile members.

Width-to-thickness ratio of the beams

								Tabe 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**
12	BF-1	W14X30	50	29000	1.1	32	0.07	70.74	9.18	45.4	Passed	8.74	Passed
11	BF-1	W18X35	50	29000	1.1	151	0.3	54.02	9.18	53.5	Passed	7.06	Passed
10	BF-1	W18X46	50	29000	1.1	340	0.51	47.73	9.18	44.6	Passed	5.01	Passed
9	BF-1	W18X60	50	29000	1.1	489	0.56	46.17	9.18	38.7	Passed	5.44	Passed
8	BF-1	W18X71	50	29000	1.1	566	0.55	46.59	9.18	32.4	Passed	4.71	Passed
7	BF-1	W18X71	50	29000	1.1	606	0.59	45.45	9.18	32.4	Passed	4.71	Passed
6	BF-1	W18X76	50	29000	1.1	624	0.57	46.05	9.18	37.8	Passed	8.11	Passed
5	BF-1	W18X86	50	29000	1.1	668	0.53	47.00	9.18	33.4	Passed	7.20	Passed
4	BF-1	W18X86	50	29000	1.1	689	0.55	46.50	9.18	33.4	Passed	7.20	Passed
3	BF-1	W18X86	50	29000	1.1	708	0.57	46.05	9.18	33.4	Passed	7.20	Passed
2	BF-1	W18X86	50	29000	1.1	743	0.59	45.22	9.18	33.4	Passed	7.20	Passed
1	BF-1	W18X86	50	29000	1.1	717	0.57	45.84	9.18	33.4	Passed	7.20	Passed
12	BF-2	W14X26	50	29000	1.1	33	0.09	66.97	9.18	48.1	Passed	5.98	Passed
11	BF-2	W21X44	50	29000	1.1	147	0.23	56.03	9.18	53.6	Passed	7.22	Passed
10	BF-2	W21X55	50	29000	1.1	294	0.37	51.94	9.18	50.0	Passed	7.87	Passed
9	BF-2	W21X68	50	29000	1.1	422	0.43	50.17	9.18	43.6	Passed	6.04	Passed
8	BF-2	W21X83	50	29000	1.1	528	0.44	49.85	9.18	36.4	Passed	5.00	Passed
7	BF-2	W21X93	50	29000	1.1	650	0.48	48.55	9.18	32.3	Passed	4.53	Passed
6	BF-2	W21X101	50	29000	1.1	768	0.52	47.38	9.18	37.5	Passed	7.68	Passed
5	BF-2	W21X111	50	29000	1.1	844	0.52	47.30	9.18	34.1	Passed	7.05	Passed
4	BF-2	W21X111	50	29000	1.1	883	0.55	46.59	9.18	34.1	Passed	7.05	Passed
3	BF-2	W21X111	50	29000	1.1	914	0.57	46.02	9.18	34.1	Passed	7.05	Passed
2	BF-2	W21X111	50	29000	1.1	939	0.58	45.56	9.18	34.1	Passed	7.05	Passed
1	BF-2	W21X111	50	29000	1.1	940	0.58	45.54	9.18	34.1	Passed	7.05	Passed

CoreBrzce Design Aids for KF and YLR

APPROXIMATE STIFFNESS MODIFICATION FACTORS, KF{1,2}

F_{ysc} = 38 ksi (262 MPa)			Lwp, ft (m)				
A_{sc} in² (cm²)	P_{y_axial} kip (kN)	Conn. Type[3]	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	40 (12.2)
5.0 (32)	171 (760)	W	1.35	1.26	1.22	1.19	1.16
		B	1.42	1.31	1.25	1.20	1.17
		P	1.45	1.33	1.26	1.22	1.18
10.0 (65)	342 (1520)	W	1.47	1.34	1.26	1.22	1.19
		B	1.62	1.44	1.34	1.28	1.24
		P	1.47	1.34	1.27	1.22	1.19
15.0 (97)	513 (2280)	W	1.56	1.38	1.31	1.25	1.21
		B	1.65	1.46	1.34	1.28	1.24
		P	1.58	1.41	1.32	1.26	1.22
20.0 (129)	684 (3040)	W	1.76	1.47	1.37	1.29	1.25
		B	1.76	1.51	1.39	1.32	1.27
		P	1.61	1.43	1.34	1.28	1.23
25.0 (161)	855 (3800)	W	-	-	-	-	-
		B	1.84	1.57	1.43	1.34	1.29
		P	1.66	1.47	1.36	1.29	1.26
30.0 (194)	1026 (4560)	W	-	-	-	-	-
		B	2.05	1.60	1.46	1.37	1.31
		P	1.64	1.46	1.35	1.29	1.25

APPROXIMATE YIELD-TO-LENGTH RATIOS, YLR{1,2}

F_{ysc} = 38 ksi (262 MPa)			Lwp, ft (m)				
A_{sc} in² (cm²)	P_{y_axial} kip (kN)	Conn. Type[3]	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	40 (12.2)
5.0 (32)	171 (760)	W	0.64	0.71	0.75	0.79	0.81
		B	0.62	0.70	0.75	0.78	0.81
		P	0.56	0.65	0.70	0.75	0.78
10.0 (65)	342 (1520)	W	0.56	0.65	0.71	0.75	0.78
		B	0.52	0.62	0.68	0.73	0.76
		P	0.58	0.67	0.72	0.76	0.79
15.0 (97)	513 (2280)	W	0.51	0.61	0.67	0.72	0.75
		B	0.51	0.60	0.67	0.72	0.75
		P	0.54	0.63	0.69	0.74	0.77
20.0 (129)	684 (3040)	W	0.43	0.57	0.62	0.68	0.72
		B	0.46	0.57	0.64	0.69	0.73
		P	0.51	0.61	0.67	0.72	0.75
25.0 (161)	855 (3800)	W	-	-	-	-	-
		B	0.44	0.55	0.63	0.68	0.72
		P	0.49	0.59	0.66	0.71	0.74
30.0 (194)	1026 (4560)	W	-	-	-	-	-
		B	0.37	0.53	0.61	0.67	0.71
		P	0.50	0.60	0.66	0.71	0.74

Approximate Stiffness Modification Factor and Yield-to Length Ratio

A_{sc}	Stort height	Bay size	H/W	Adj.	Lwp	KF	YLR	**Adjusted KF**	**Adjusted YLR**
in^2	ft	ft			ft.				
5	14	33.3	0.42	0.89	36.2	1.21	0.76	**1.36**	**0.85**
5	12.5	33.3	0.38	0.88	35.6	1.22	0.75	**1.37**	**0.85**
5	14	25	0.56	0.91	28.7	1.28	0.69	**1.44**	**0.77**
5	12.5	25	0.50	0.90	28.0	1.29	0.68	**1.45**	**0.77**
10	14	33.3	0.4	0.89	36.2	1.21	0.77	**1.37**	**0.86**
10	12.5	33.3	0.4	0.88	35.6	1.22	0.76	**1.37**	**0.86**
10	14	25	0.6	0.91	28.7	1.29	0.71	**1.45**	**0.80**
10	12.5	25	0.5	0.90	28.0	1.30	0.70	**1.46**	**0.79**
15	14	33.3	0.4	0.89	36.2	1.25	0.75	**1.41**	**0.84**
15	12.5	33.3	0.4	0.88	35.6	1.26	0.74	**1.41**	**0.84**
15	14	25	0.6	0.91	28.7	1.34	0.67	**1.51**	**0.76**
15	12.5	25	0.5	0.90	28.0	1.36	0.67	**1.53**	**0.75**
20	14	33.3	0.4	0.89	36.2	1.27	0.73	**1.43**	**0.82**
20	12.5	33.3	0.4	0.88	35.6	1.27	0.72	**1.43**	**0.81**
20	14	25	0.6	0.91	28.7	1.36	0.65	**1.54**	**0.74**
20	12.5	25	0.5	0.90	28.0	1.38	0.65	**1.55**	**0.73**

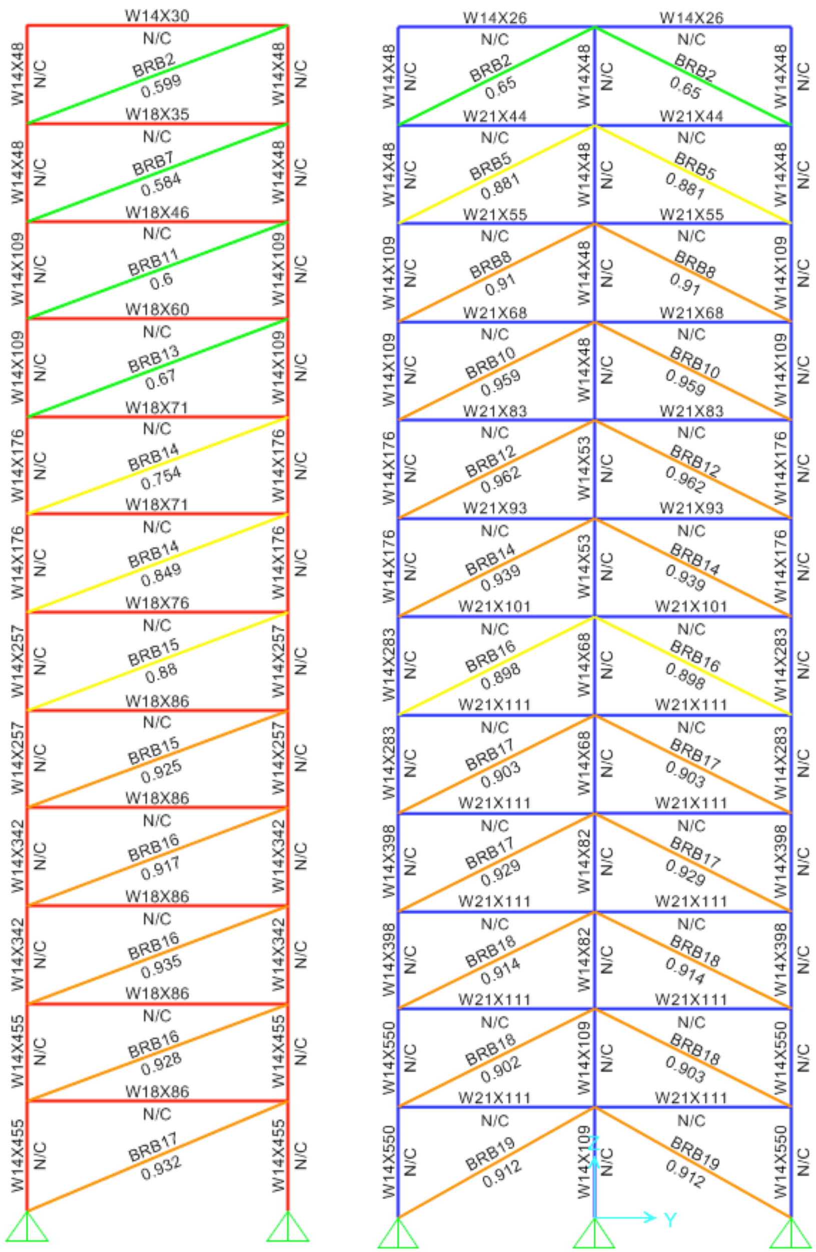
Table values based on 14” (356 mm) deep columns and 18” (457 mm) deep beam

Approximate Stiffness Modification Factor and Yield-to Length Ratio

Story	Brace -X A_{sc} (in^2)	KF_X	YLR_X	Brace-Y A_{sc} (in^2)	KF_Y	YLR_Y
12th	2	1.37	0.85	2	1.45	0.77
11th	7	1.37	0.85	5	1.45	0.77
10th	11	1.37	0.86	8	1.46	0.79
9th	13	1.37	0.86	10	1.46	0.79
8th	14	1.37	0.86	12	1.53	0.75
7th	14	1.37	0.86	14	1.53	0.75
6th	15	1.41	0.84	16	1.53	0.75
5th	15	1.41	0.84	17	1.53	0.75
4th	16	1.41	0.84	17	1.53	0.75
3rd	16	1.41	0.84	18	1.53	0.75
2nd	16	1.41	0.84	18	1.53	0.75
1st*	17	1.41	0.84	19	1.51	0.76
Approximate factors		1.39	0.85		1.50	0.76

* Story Height =14 ft

BRB Design (DCR ratios)



BF-1

BF-2

Story	Brace -X A_{sc} (in^2)	Brace-Y A_{sc} (in^2)
12th	2.0	2.0
11th	7.0	5.0
10th	11.0	8.0
9th	13.0	10.0
8th	14.0	12.0
7th	14.0	14.0
6th	15.0	16.0
5th	15.0	17.0
4th	16.0	17.0
3rd	16.0	18.0
2nd	16.0	18.0
1st	17.0	19.0

(BRB Design load combination)

C1: E_x
C2: E_y
C3: $E_{x+5\%}$
C4: $E_{x-5\%}$
C5: $E_{y+5\%}$
C6: $E_{y-5\%}$

Average Brace Strain

$$\theta_x = \frac{\Delta_x}{h_{sx}}$$

$$h_{sx} = L_{wp}\sin\alpha \qquad L_y = YLR\ L_{wp}$$

$$\Delta_{bx} = \Delta_x \cos\alpha$$

$$\theta_x = \frac{\Delta_{bx}/\cos\alpha}{L_{wp}\sin\alpha} = \frac{2\Delta_{bx}}{L_{wp}\sin 2\alpha}$$

$$\theta_x = \frac{2\Delta_{bx}}{L_y}\frac{L_y}{L_{wp}\sin 2\alpha} = 2\varepsilon_{sc}\frac{YLR}{\sin 2\alpha}$$

$$\boxed{\varepsilon_{sc} = \frac{\theta_x \sin 2\alpha}{2YLR}}$$

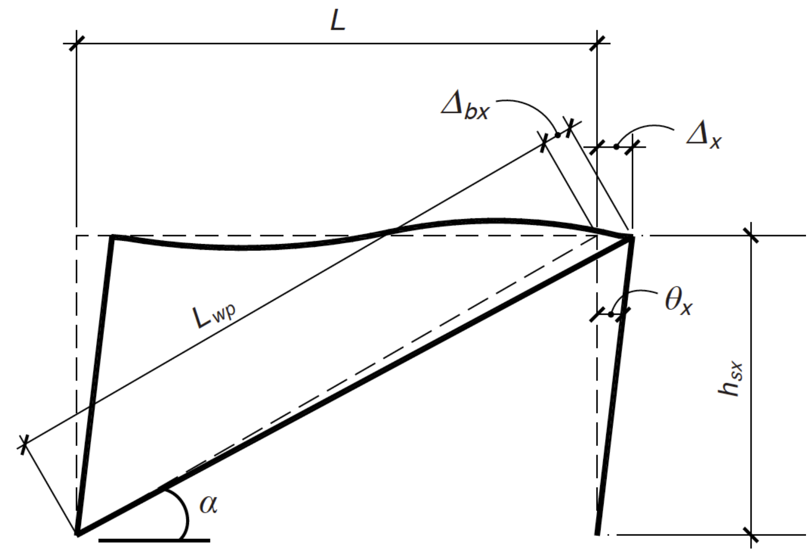
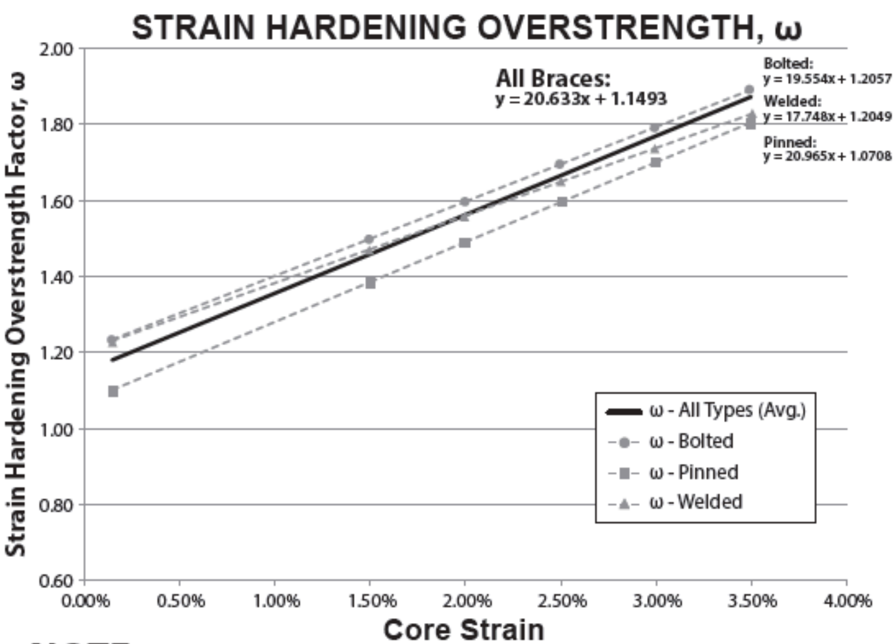
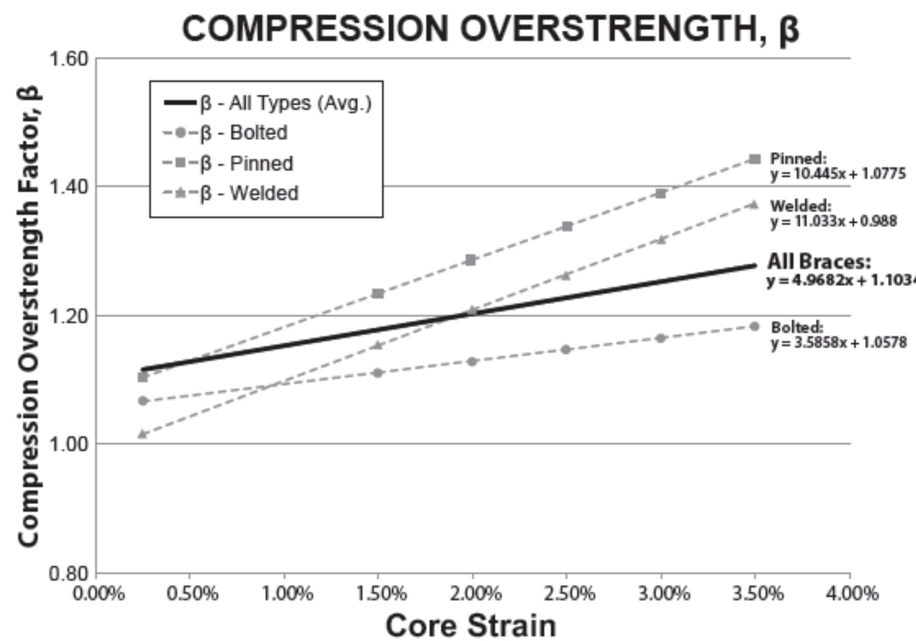


Figure ref.:
Seismic Design of Steel Buckling-Restrained Braced Frames: A Guide for Practicing Engineers, NIST GCR 15-917-34 (2015)

CoreBrace Adjusted Strength Factors (β, ω)





NOTE:

The graphs above are based on subassemblage testing data from multiple BRB manufacturers and provide results at approximately a standard deviation above the mean. They may be conservative. Used by permission of the authors: B. Saxey and M. Daniels.

Average Brace Strain

X-Direction	Story height (ft)	Bay size (ft)	*Expected Drift Ratio (%)	Angle (deg.)	Yield Length Ratio	Average Brace Strain (%)	Strain Hardening	Compression Overstrength	
Story	h_{sx}	B	θ_M	α	YLR	ε_{sc}	ω	β	$\omega\beta$
12	12.5	33.33	3.80	20.6	0.85	1.47	1.38	1.23	1.70
11	12.5	33.33	3.74	20.6	0.85	1.45	1.37	1.23	1.69
10	12.5	33.33	3.70	20.6	0.86	1.41	1.37	1.23	1.67
9	12.5	33.33	3.76	20.6	0.86	1.44	1.37	1.23	1.68
8	12.5	33.33	3.81	20.6	0.86	1.46	1.38	1.23	1.69
7	12.5	33.33	3.87	20.6	0.86	1.48	1.38	1.23	1.70
6	12.5	33.33	3.74	20.6	0.84	1.46	1.38	1.23	1.69
5	12.5	33.33	3.62	20.6	0.84	1.42	1.37	1.23	1.68
4	12.5	33.33	3.35	20.6	0.84	1.31	1.35	1.21	1.63
3	12.5	33.33	3.13	20.6	0.84	1.23	1.33	1.21	1.60
2	12.5	33.33	2.85	20.6	0.84	1.11	1.30	1.19	1.56
1	14	33.33	2.39	22.8	0.84	1.01	1.28	1.18	1.52

Y-Direction	Story height (ft)	Bay size (ft)	*Expected Drift Ratio (%)	Angle (deg.)	Yield Length Ratio	Average Brace Strain (%)	Strain Hardening	Compression Overstrength	
Story	h_{sx}	B	θ_M	α	YLR	ε_{sc}	ω	β	$\omega\beta$
12	12.5	25	2.88	26.6	0.77	1.50	1.38	1.23	1.71
11	12.5	25	3.31	26.6	0.77	1.72	1.43	1.26	1.80
10	12.5	25	3.32	26.6	0.79	1.68	1.42	1.25	1.78
9	12.5	25	3.31	26.6	0.79	1.67	1.42	1.25	1.78
8	12.5	25	3.20	26.6	0.75	1.71	1.43	1.26	1.79
7	12.5	25	2.98	26.6	0.75	1.59	1.40	1.24	1.75
6	12.5	25	2.75	26.6	0.75	1.47	1.38	1.23	1.70
5	12.5	25	2.56	26.6	0.75	1.37	1.36	1.22	1.66
4	12.5	25	2.44	26.6	0.75	1.30	1.34	1.21	1.63
3	12.5	25	2.20	26.6	0.75	1.17	1.32	1.20	1.58
2	12.5	25	2.00	26.6	0.75	1.07	1.29	1.19	1.54
1	14	25	2.00	29.2	0.76	1.12	1.31	1.19	1.56

*Expected drift ratio is 2% of the story height or two times the design story drift, whichever is larger.

Adjusted Brace Strength

Based on the CoreBrace design Aids, β and ω factors are estimated to be as follows:

β=1.25
ω=1.41
$\beta\omega$=1.7625

F_{ysc} =42 ksi (±4ksi)

$F_{ysc\ (min)}$=38 ksi
$F_{ysc\ (max)}$=46 ksi

R_y (estimated)=46/38=1.21

Adjusted brace strength in compression: $R_y\beta\omega$=2.134
Adjusted brace strength in tension: $R_y\omega$=1.707

Adjusted Brace Strength

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

$E=1$ ksi
$\alpha=1/^{o}F$
KF=1.0

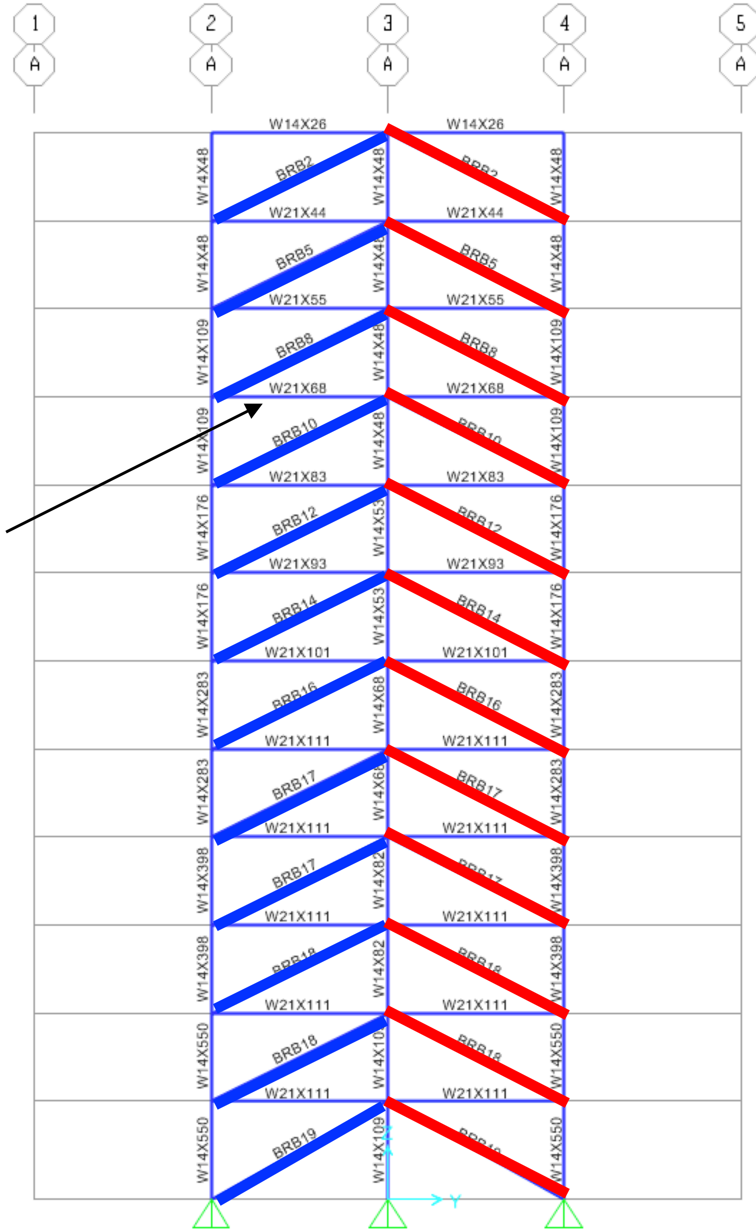
$\Delta T= R_y\beta\omega\, F_{ysc\ (min)}=81.08$
(Compression)

$\Delta T= R_y\omega\, F_{ysc\ (min)}=-64.87$ ksi
(Tension)

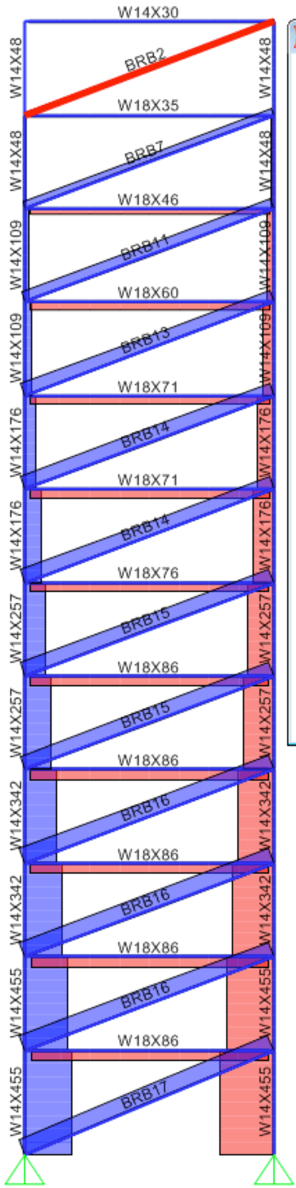
The force will be combined with dead and live loads as follows:

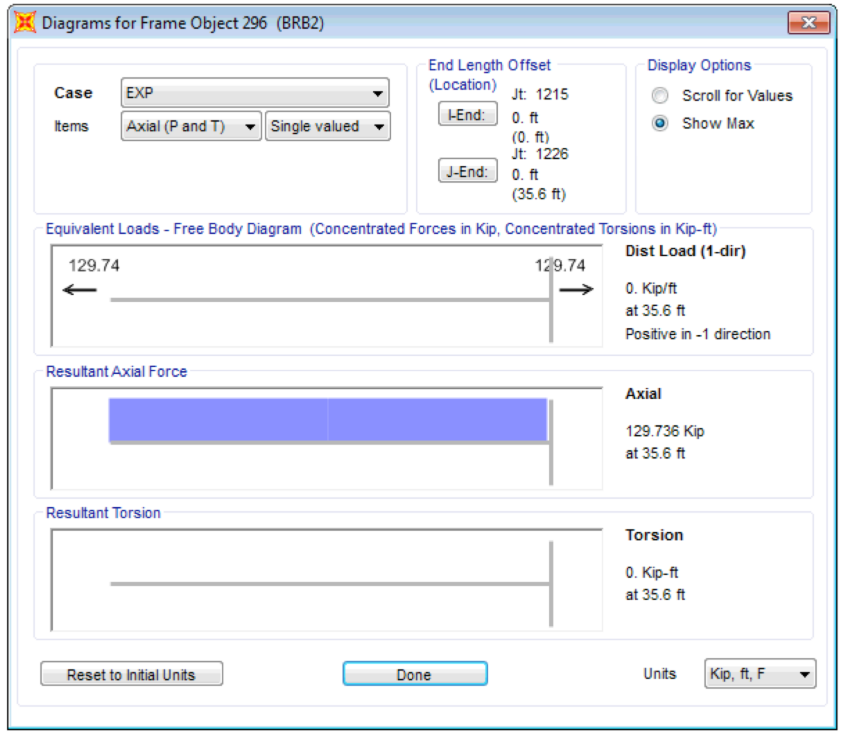
$(1.2D+0.2S_{ds})+0.5L+\text{brace\_force}$

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



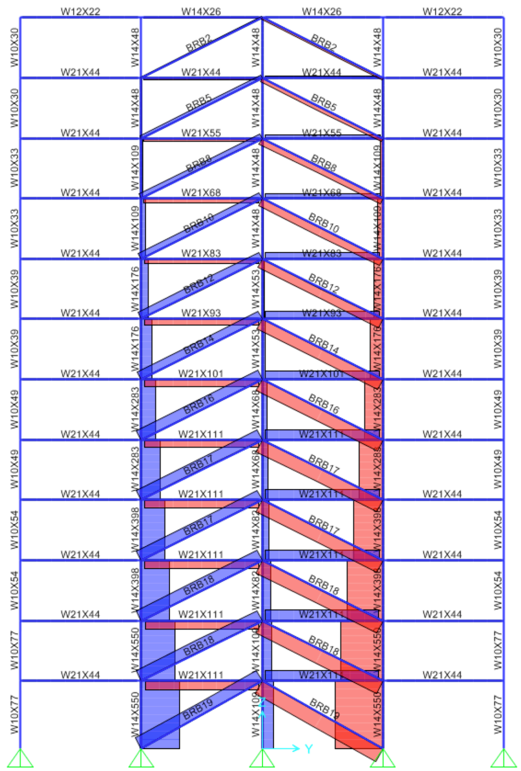
Adjusted Brace Strength (Verification)



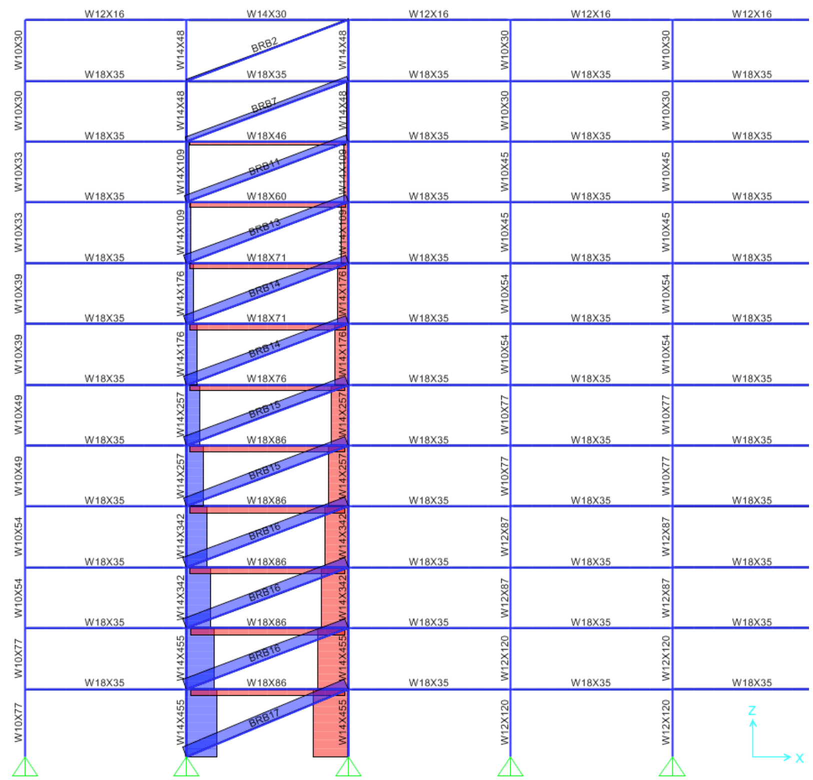


$R_y \omega F_{ysc\,(min)} A_{sc}$=2x64.87 ksi=129.74 kips (Tension)

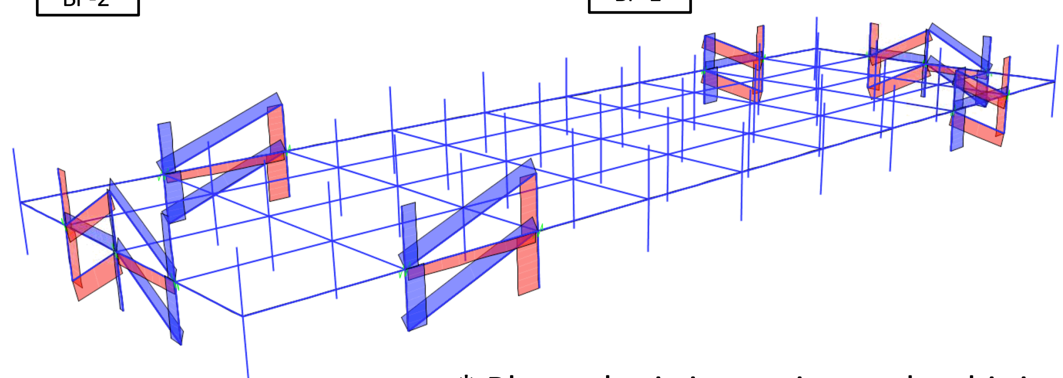
Axial forces in the beams and columns
under capacity-limited seismic load effect



BF-2



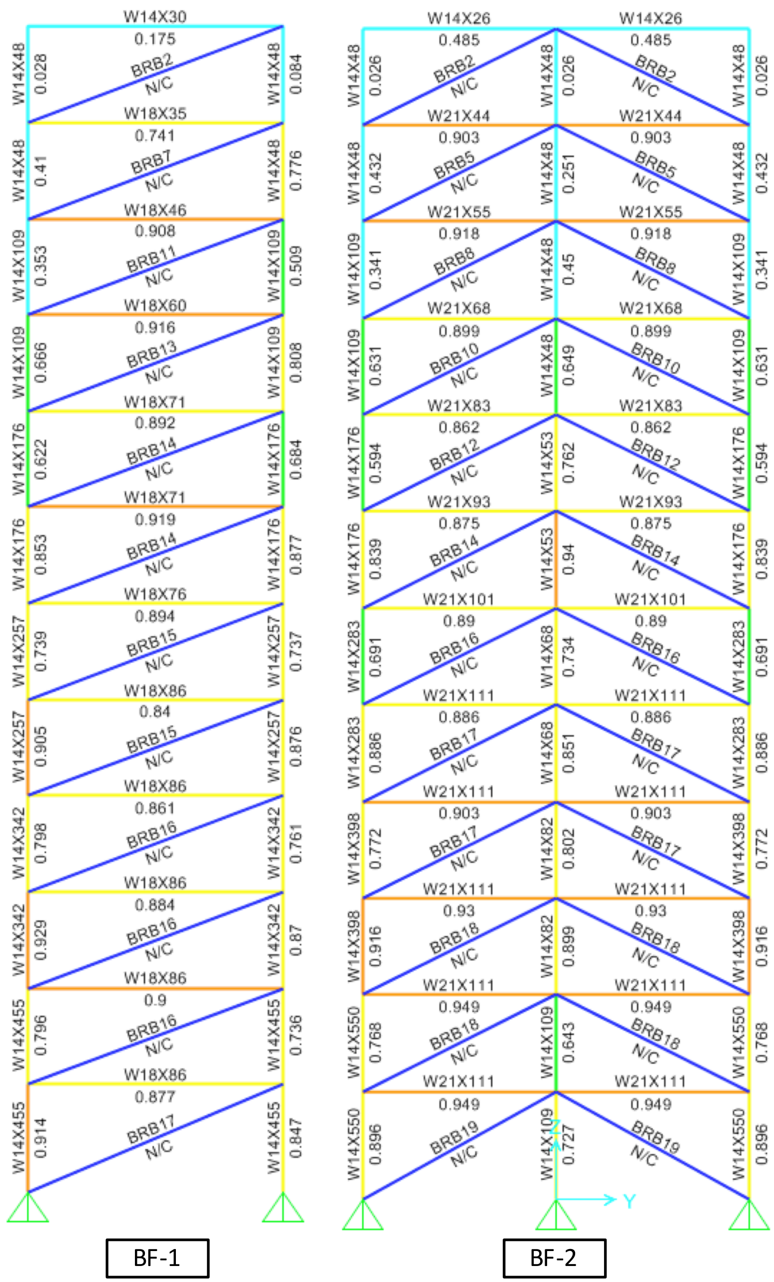
BF-1



* Blue color is in tension and red is in compression.

Bracing Column and Beam Design (DCR ratios)

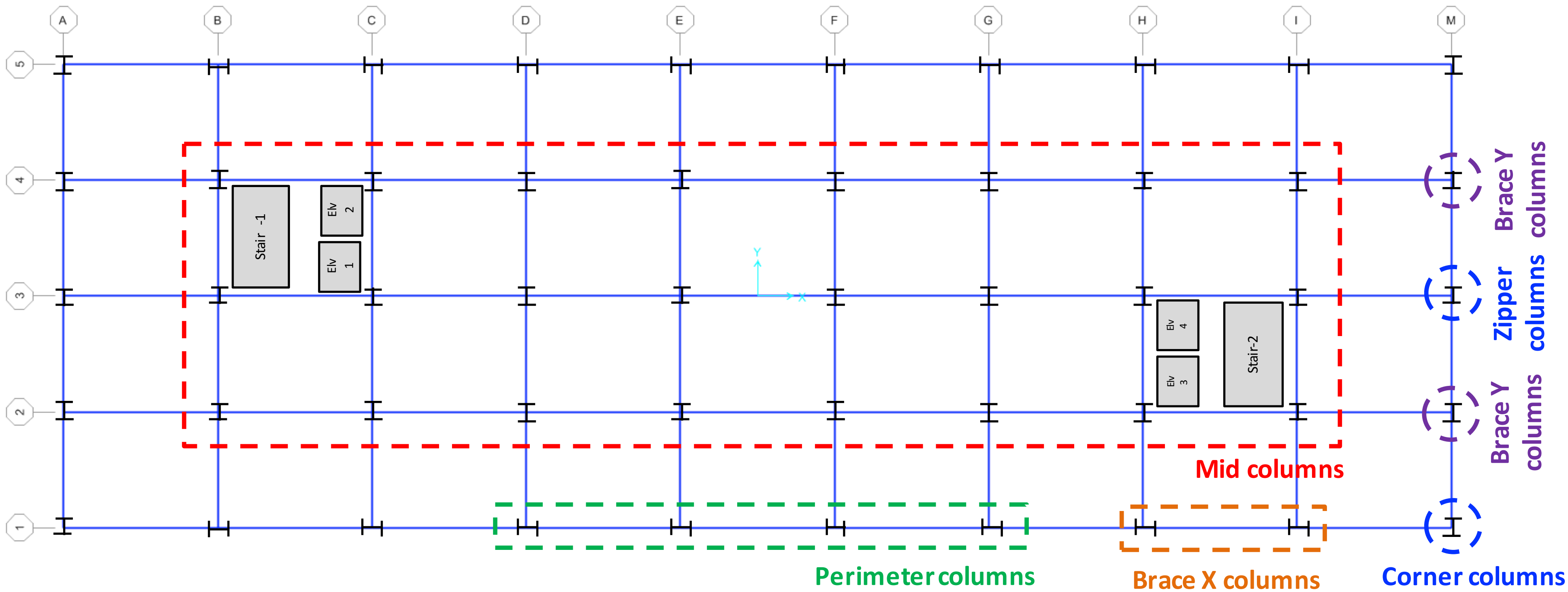
Bracing frame beams columns are designed for the maximum of the adjusted strength of the BRBs and the amplified load combinations. (AISC 341-16: Chapter D.4a)



adjusted strength of the BRBs

amplified load combinations

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-Y, and Brace-Y-Zipper 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.

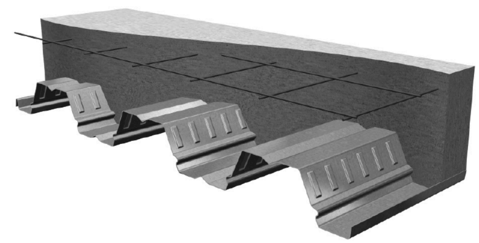
Column Design Summary

	Gravity Columns			Brace frame columns		
Story	**Perimeter columns**	**Mid columns**	**Corner Columns**	**Brace-X columns**	**Brace-Y columns**	**Brace-Y-Zipper-columns**
12 11	W10x30	W10x33	W10x30	W14x48	W14x48	W14x48
10 9	W10x45	W10x49	W10x33	W14x109	W14x109	W14x48
8 7	W10x54	W10x77	W10x39	W14x176	W14x176	W14x53
6 5	W10x77	W10x100	W10x49	W14x257	W14x283	W14x68
4 3	W12x87	W12x120	W10x54	W14x342	W14x398	W14x82
2 1	W12x120	W12x152	W10x77	W14x455	W14x550	W14x109

Composite deck gravity design (typical floors)

PLW3™ or W3 FORMLOK™

- 7½ in. TOTAL SLAB DEPTH
- Normal Weight Concrete
- 2 Hour Fire Rating



Maximum Unshored Clear Span (ft-in.)

Deck Gage	Number of Deck Spans 1	2	3
22	8'-3"	7'-4"	7'-4"
21	8'-11"	9'-2"	9'-2"
20	9'-7"	10'-4"	10'-8"
19	10'-6"	11'-5"	11'-10"
18	11'-0"	12'-5"	12'-10"
16	11'-8"	13'-10"	13'-8"

Shoring is required for spans greater than those shown above. See Footnote 1 on page 69 for required bearing.

Concrete Properties

Density (pcf)	Uniform Weight (psf)	Uniform Volume ($yd^3/100\ ft^2$)	Compressive Strength, f'_c (psi)
145	72.5	1.852	3000

Notes:
1. Volumes and weights do not include allowance for deflection.
2. Weights are for concrete only and do not include weight of steel deck.
3. Total slab depth is nominal depth from top of concrete to bottom of steel deck.

Allowable Superimposed Loads (psf)

Deck Gage	Number of Deck Spans	Span (ft-in.) 8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13-0"	13'-6"	14'-0"	15'-0"	16'-0"
22	1	388	283	251	224	200	179	161	144	130	117	105	95	85	69	55
	2	320	283	251	224	200	179	161	144	130	117	105	95	85	69	55
	3	320	283	251	224	200	179	161	144	130	117	105	95	85	69	55
21	1	400	378	276	247	221	198	179	161	145	131	119	107	97	79	64
	2	400	378	344	247	221	198	179	161	145	131	119	107	97	79	64
	3	400	378	344	247	221	198	179	161	145	131	119	107	97	79	64
20	1	400	400	367	335	240	216	195	176	160	145	131	119	108	89	73
	2	400	400	367	335	308	216	195	176	160	145	131	119	108	89	73
	3	400	400	367	335	308	284	195	176	160	145	131	119	108	89	73
19	1	400	400	400	378	347	320	228	207	188	171	156	143	130	109	91
	2	400	400	400	378	347	320	296	207	188	171	156	143	130	109	91
	3	400	400	400	378	347	320	296	275	188	171	156	143	130	109	91
18	1	400	400	400	400	384	354	327	236	215	196	180	164	151	127	107
	2	400	400	400	400	384	354	327	304	283	196	180	164	151	127	107
	3	400	400	400	400	384	354	327	304	283	264	180	164	151	127	107
16	1	400	400	400	400	400	400	389	361	267	245	225	208	191	163	139
	2	400	400	400	400	400	400	389	361	336	314	294	276	191	163	139
	3	400	400	400	400	400	400	389	361	336	314	294	276	191	163	139

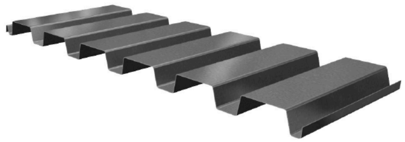
See footnotes on page 69.

Shoring required in shaded areas to right of heavy line.

- Super imposed load for typical floors is 100.5 (dead) + 50 (live)=150.5psf
- The joists are along x-direction and joist spacing is assumed to be 12.5 ft, which is one joist in the middle of each bay.
- Minimum required deck thickness for PLW3 or W3 FORMLOK decks is 16 gage for unshored and 19 gage for shored construction, based on 2 span design.
- The immediate deflection of the composite slab due to allowable superimposed load is limited to L/360.
- Deck thickness may change in diaphragm design.

Bare deck gravity design (Roof)

Type PLB™-36 or HSB®-36



Allowable Uniform Loads (psf)

SPAN	DECK GAGE	CRITERIA	2'-0"	3'-0"	4'-0"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	11'-0"	12'-0"
SINGLE	22	Stress	300	300	220	141	116	98	83	72	63	55	49	43	39	35	29	24
		L/360	♦♦♦	287	121	62	47	36	28	23	18	15	13	11	9	8	6	4
		L/240	♦♦♦	♦♦♦	182	93	70	54	42	34	28	23	19	16	14	12	9	7
		L/180	♦♦♦	♦♦♦	♦♦♦	124	93	72	56	45	37	30	25	21	18	15	12	9
	20	Stress	300	300	288	184	152	128	109	94	82	72	64	57	51	46	38	32
		L/360	♦♦♦	♦♦♦	150	77	58	44	35	28	23	19	16	13	11	10	7	6
		L/240	♦♦♦	♦♦♦	225	115	86	67	52	42	34	28	23	20	17	14	11	8
		L/180	♦♦♦	♦♦♦	♦♦♦	153	115	89	70	56	45	37	31	26	22	19	14	11
	18	Stress	300	300	300	251	208	174	149	128	112	98	87	78	70	63	52	44
		L/360	♦♦♦	♦♦♦	207	106	79	61	48	39	31	26	22	18	15	13	10	8
		L/240	♦♦♦	♦♦♦	♦♦♦	159	119	92	72	58	47	39	32	27	23	20	15	11
		L/180	♦♦♦	♦♦♦	♦♦♦	212	159	122	96	77	63	52	43	36	31	26	20	15
	16	Stress	300	300	300	300	264	222	189	163	142	125	110	99	88	80	66	55
		L/360	♦♦♦	♦♦♦	261	133	100	77	61	49	40	33	27	23	19	17	13	10
		L/240	♦♦♦	♦♦♦	♦♦♦	200	150	116	91	73	59	49	41	34	29	25	19	14
		L/180	♦♦♦	♦♦♦	♦♦♦	267	200	154	121	97	79	65	54	46	39	33	25	19
DOUBLE	22	Stress	300	300	235	150	124	104	89	77	67	59	52	46	42	38	31	26
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	122	94	74	59	48	40	33	28	24	20	15	12
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	49	42	35	30	23	18
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	30	23
	20	Stress	300	300	296	190	157	132	112	97	84	74	66	59	53	47	39	33
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	146	113	89	71	58	48	40	33	28	24	18	14
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	71	59	50	43	37	27	21
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	37	28
	18	Stress	300	300	300	265	219	184	157	135	118	103	92	82	73	66	55	46
		L/360	♦♦♦	♦♦♦	♦♦♦	258	194	149	117	94	76	63	53	44	38	32	24	19
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	115	94	79	66	56	48	36	28
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	64	48	37
	16	Stress	300	300	300	300	271	228	194	167	146	128	113	101	91	82	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	241	186	146	117	95	78	65	55	47	40	30	23
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	118	98	83	70	60	45	35
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	80	60	46
TRIPLE	22	Stress	300	300	294	188	155	131	111	96	84	73	65	58	52	47	39	33
		L/360	♦♦♦	♦♦♦	247	127	95	73	58	46	38	31	26	22	18	16	12	9
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	143	110	86	69	56	46	39	33	28	24	18	14
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	92	75	62	52	43	37	32	24	18
	20	Stress	300	300	300	237	196	165	140	121	105	93	82	73	66	59	49	41
		L/360	♦♦♦	♦♦♦	298	152	115	88	69	56	45	37	31	26	22	19	14	11
		L/240	♦♦♦	♦♦♦	♦♦♦	229	172	132	104	83	68	56	47	39	33	29	21	17
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	139	111	90	74	62	52	44	38	29	22
	18	Stress	300	300	300	300	274	230	196	169	147	129	115	102	92	83	68	57
		L/360	♦♦♦	♦♦♦	♦♦♦	202	152	117	92	74	60	49	41	35	29	25	19	15
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	228	175	138	110	90	74	62	52	44	38	28	22
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	184	147	120	99	82	69	59	50	38	29
	16	Stress	300	300	300	300	300	285	243	209	182	160	142	127	114	103	85	71
		L/360	♦♦♦	♦♦♦	♦♦♦	251	189	145	114	92	74	61	51	43	37	31	24	18
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	283	218	172	137	112	92	77	65	55	47	35	27
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	229	183	149	123	102	86	73	63	47	36

See footnotes on page 27.

- Super imposed load for the roof is 25 (dead)+20(live)=55psf
- The joist are along x-direction and joist spacing is assumed to be 6.25ft, which is three joist in each bay of 25ft long..
- Minimum required deck thickness for HSB-36 interlocking deck is 22 gage.
- The immediate deflection of the composite slab due to allowable superimposed load is limited to L/360.
- Deck thickness may change in diaphragm design.

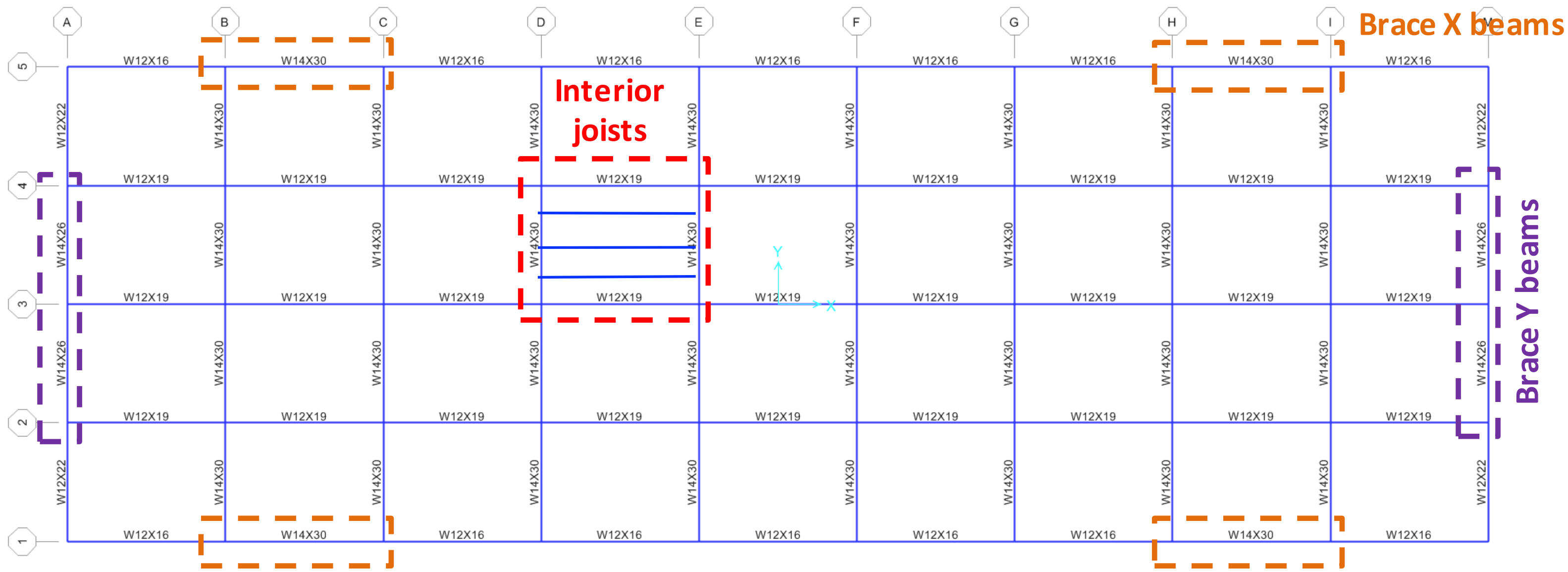
Beam (joist) and Girder design Summary

	Element	Location	Cross-section	Length (ft)	Spacing (ft)	No. Studs	S (in)	S_selected	Camber (in)
Floor	Beams	Interior	W18x40	33.33	12.50	22	18.2	18	1.50
		Exteior	W18x35	33.33	12.50	14	28.6	28	0.00
	Girders	Interior	W21x55	25	33.33	20	15.0	15	0.00
		Exteior	W21x44	25	33.33	16	18.8	18	0.00
Roof	Beams	Interior	W12x19	33.33	6.25	-	-	-	-
		Exteior	W12x16	33.33	6.25	-	-	-	-
	Girders	Interior	W14x30	25	33.33	-	-	-	-
		Exteior	W12x22	25	33.33	-	-	-	-

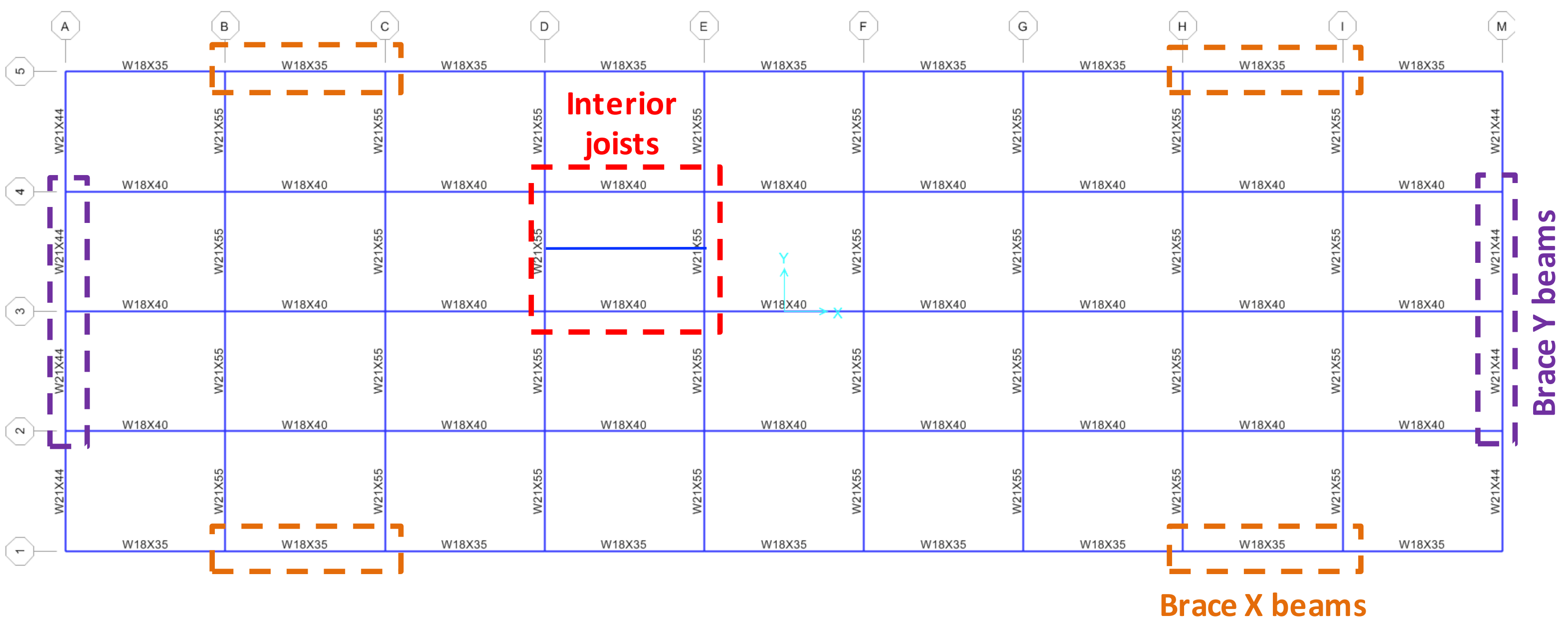
Notes:

- Floor beams (joists) and girders are designed as partially composite beams with ¾” steel headed stud anchors. All beams and girders are braced in lateral torsional buckling (LTB).
- Roof beams (joists) and girders are designed as non-composite beams. Joists are assumed to be braced in LTB by the steel deck, but the girders are just braced at the location of the joists (3 points along the length)
- All beams and girders are design in a separate spreadsheet attached to this document.
- *Exterior beams (joists) and girders may subject to change after designing chords and collectors.

Beam (joists) and Girder design Summary



Roof beam and girders



Typical floor beam and girders

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

Diaphragm Design

Diaphragms are design for the following demands:

1) ASCE 7-16
2) ASCE 7-16 Alternative design, R_s=3.0
3) ASCE 7-16 Alternative design, R_s=1.0

Following parts of the diaphragms are designed:

- Shear design of the composite floors
- Shear design of the bare deck roof
- Collector design of the bare deck roof and the composite floors
- Chord design of the bare deck roof and the composite floors

Chord and collector design (Composite floor)

- Chord is provided by two methods: (1) addition of supplemental slab reinforcement (ASTM A615, Grade 60 deformed reinforcing bars) and compressive strength of concrete; (2) perimeter steel beams designed as beam-columns.

- Collector is provided by two methods: (1) supplemental slab reinforcement for the collector in tension and compressive slab strength in compression. Collectors are designed for the amplified seismic demands (2) perimeter steel beams designed as beam-columns.

- Diaphragm to LFRS connection is provided by the steel headed shear studs designed for the amplified seismic demand.

- 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.

Chord and collector design (Roof deck)

- Chord is provided by perimeter beams designed as beam-columns. Chord are on grids 1 and 5 for the long direction of the diaphragm (Y direction seismic demands) and on grids A and M for the short direction of the diaphragm (X direction seismic demands).

- Collector is provided by by perimeter beams designed as beam-columns. Collectors are on grids A and M for the long direction of the diaphragm (Y direction seismic demands) and on grids 1 and 5 for the short direction of the diaphragm (X direction seismic demands).

- Diaphragm to LFRS connection is provided by the steel headed shear studs designed for the amplified seismic demand.

- The beams 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 (H and I) on grids 1 and 5 are designed as non-composite beams connected to the diaphragms vis shear studs.

Diaphragm demands (ASCE7-16 and ASCE7-16 Alt. R_s=1, 3)

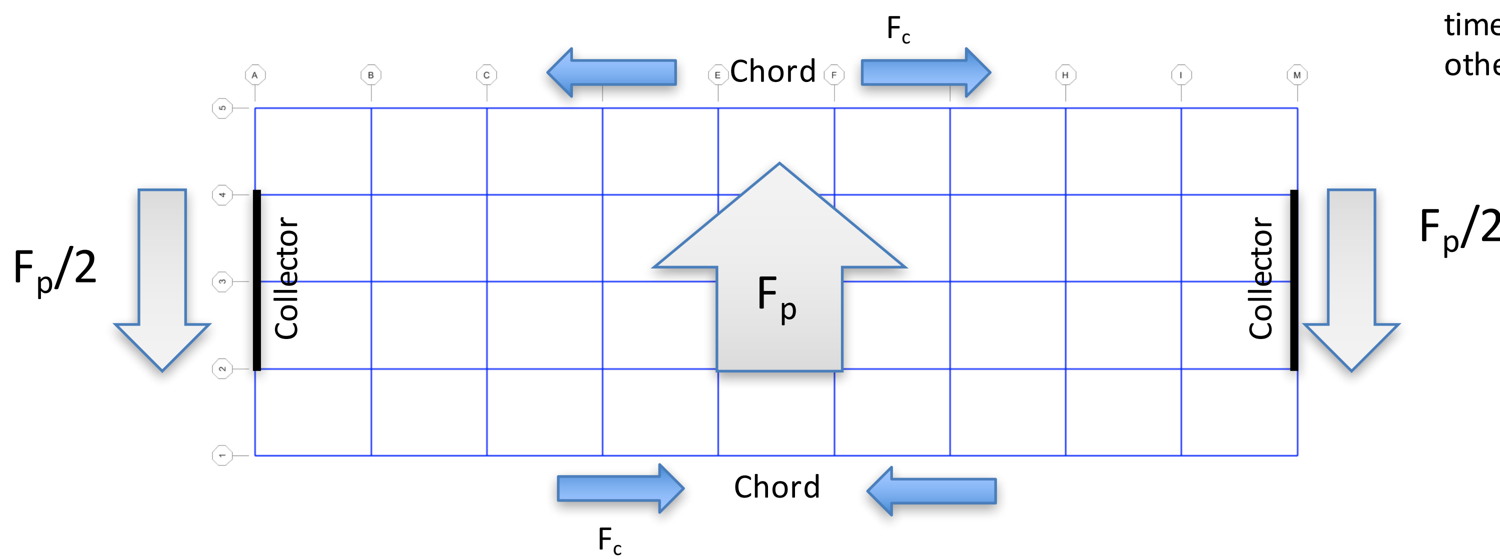
ASCE7-16 and ASCE7-16 Alt. R_s=3

Level	F_i(k)	W_i (k)	F_P (k)	F_{P-min} (k)	F_{P-max} (k)	F_P (k) design
Roof	147	1271	147	262	524	**262**
12th	343	3415	358	703	1407	**703**
11th	294	3415	331	703	1407	**703**
10th	247	3415	306	703	1407	**703**
9th	204	3415	283	703	1407	**703**
8th	164	3415	261	703	1407	**703**
7th	128	3415	240	703	1407	**703**
6th	95	3415	220	703	1407	**703**
5th	66	3415	202	703	1407	**703**
4th	42	3415	185	703	1407	**703**
3rd	22	3415	169	703	1407	**703**
2nd	8	3415	155	703	1407	**703**

ASCE7 Alt. R_s=1

F_P (k) design
419
1126
1126
1142
1171
1200
1229
1258
1287
1316
1345
1374

Note: About 2 times of the other cases



Diaphragm Shear and Chord demand (Composite floor, roof deck-Transverse direction)

ASCE7-16 and ASCE7-16 Alt. R_s=3

Level	v_r (lb/ft)	F_c (kip)
Roof	1309	98
12th	3517	264
11th	3517	264
10th	3517	264
9th	3517	264
8th	3517	264
7th	3517	264
6th	3517	264
5th	3517	264
4th	3517	264
3rd	3517	264
2nd	3517	264

ASCE7-16 Alt. R_s=1

Level	v_r (lb/ft)	F_c (kip)
Roof	2095	157
12th	5628	422
11th	5628	422
10th	5712	428
9th	5857	439
8th	6002	450
7th	6147	461
6th	6292	472
5th	6437	483
4th	6582	494
3rd	6727	505
2nd	6872	515

Diaphragm shear demand, uniform distribution along the depth of diaphragm on gridlines A and M:

$v_r=F_p/(2h_{diaphragm})$

Maximum tension and compression force in the chords along gridlines 1 and 5:

$F_c=F_pL_{diaphragm}/(8h_{diaphragm})$

Diaphragm Design (Composite floor)

ASCE7-16 and ASCE7-16 Alt. R_s=3

- Required shear capacity v_r is 3517 lb/ft, to use allowable strength we need to divide it by 1.5 (as recommended by Verco), v_{ra}=2344 lb/ft < v_a=2950 lb/ft
- Minimum concrete reinforcement 0.0405 in^2 per foot width of the slab in each direction.

DIAPHRAGMS WITH STEEL HEADED STUD ANCHORS

Table 5: Allowable Diaphragm Shear Strengths (plf) and Flexibility Factors (in./lbx10^6) for Decks with Concrete Fill and ¾" Diameter Steel Headed Stud Anchors [1-8, 11, 16, 17, 19]

Concrete Type[9]	Concrete Thickness[10]	Spacing of Steel Headed Stud Anchors[14]							F[12]
		12"	16"	18"	24"	30"	32"	36"	
Minimum Concrete Reinforcement of 0.00075 Times the Area of Fill Above the Deck[13]									
NW	2"[18]	1310	1310	1310	1310	1310	1310	1310	0.40
	2½"	1640	1640	1640	1640	1640	1640	1640	0.32
	3"	1970	1970	1970	1970	1970	1970	1970	0.26
	3½"	2300	2300	2300	2300	2300	2300	2300	0.23
	4½"	2950	2950	2950	2950	2950	2790	2480	0.18
	6"	3940	3940	3940	3720	2980	2790	2480	0.13

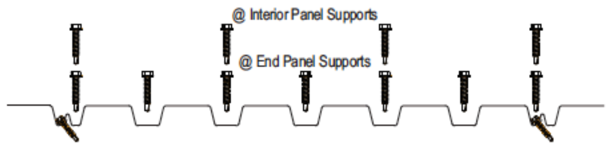
Diaphragm Design (Roof deck)

ASCE7-16 and ASCE7-16 Alt. R_s=3

- Required shear capacity v_r is 1309 lb/ft, to use allowable strength we need to divide it by 1.5 (as recommended by Verco), v_{ra}=872 lb/ft < v_a=1037 lb/ft

Type HSB®-36-SS

- **36/7/4 Screw Pattern at Supports**
  #12 or #14 SDI Recognized Screws
  at Supports 0.0385" and thicker
- **Sidelaps Connected with #10 Screws**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^6)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
18	#10 @ 24"	q	792	729	594	568	475	472	411	416	373
		F	4.7+17R	5.4+11R	6.8+7R	6.9+6R	8+4R	7.8+3R	8.7+3R	8.5+2R	9.2+2R
	#10 @ 18"	q	969	729	696	651	551	538	529	469	468
		F	3.8+18R	5.4+11R	5.8+8R	6.1+6R	7+5R	7+4R	7+3R	7.7+3R	7.6+2R
	#10 @ 12"	q	969	854	793	731	688	658	634	616	601
		F	3.8+18R	4.7+11R	5.2+8R	5.5+7R	5.8+5R	5.9+4R	6.1+4R	6.2+3R	6.3+3R
	#10 @ 8"	q	1122	1074	969	949	877	875	829	833	799
		F	3.3+18R	3.8+12R	4.4+9R	4.5+7R	4.8+6R	4.8+5R	5+4R	4.9+4R	5.1+3R
	#10 @ 6"	q	1254	1169	1123	1076	1043	1019	1001	986	975
		F	3+18R	3.6+12R	3.9+9R	4.1+7R	4.2+6R	4.3+5R	4.4+4R	4.4+4R	4.5+4R
	#10 @ 4"	q	1461	1400	1367	1333	1309	1291	1278	1267	1259
		F	2.6+19R	3.1+12R	3.3+9R	3.5+7R	3.6+6R	3.6+5R	3.7+5R	3.7+4R	3.8+4R
16	#10 @ 24"	q	1019	950	774	748	631	633	552	562	504
		F	4.3+9R	4.9+6R	6.1+3R	6.1+3R	7+2R	6.9+1R	7.6+1R	7.4+1R	8+0R
	#10 @ 18"	q	1260	950	914	862	731	719	709	635	636
		F	3.6+10R	4.9+6R	5.2+4R	5.4+3R	6.1+2R	6.1+2R	6.1+2R	6.7+1R	6.6+1R
	#10 @ 12"	q	1260	1121	1046	971	920	882	853	831	813
		F	3.6+10R	4.2+6R	4.6+4R	4.8+3R	5+3R	5.2+2R	5.3+2R	5.4+2R	5.4+1R
	#10 @ 8"	q	1466	1416	1284	1265	1175	1176	1117	1124	1081
		F	3.1+10R	3.4+7R	3.8+5R	3.9+4R	4.1+3R	4.1+3R	4.3+2R	4.3+2R	4.4+2R
	#10 @ 6"	q	1639	1540	1486	1431	1393	1366	1344	1327	1314
		F	2.8+10R	3.2+7R	3.4+5R	3.5+4R	3.6+3R	3.7+3R	3.7+2R	3.8+2R	3.8+2R
	#10 @ 4"	q	1901	1833	1796	1758	1732	1712	1697	1686	1676
		F	2.5+11R	2.8+7R	2.9+5R	3+4R	3.1+3R	3.1+3R	3.2+3R	3.2+2R	3.2+2R

See footnotes on page 28.

Span=6.25 ft
q=1037 lb/ft
F=4.25+5.75/2=7.125 (in/lb)x10^6

Detailing:

- 18 gage deck
- 36/7/4 #12 screw Pattern at supports
- #10 @6" sidelap attachments

Diaphragm Design (Composite floor)

ASCE7-16 Alt. R_s=1

- Maximum Required shear capacity v_r is 6872 lb/ft, to use allowable strength we need to divide it by 1.5 (as recommended by Verco), v_{ra}=4581 lb/ft < v_a=4970 lb/ft
- Spacing of ¾" diameter Steel Headed Stud Anchors=18" (each stud 7.46 kip allowable shear strength). Stud length is 4.5".
- Sidelap No.10 screw 36" o.c.
- Minimum concrete reinforcement 0.135 in^2 per foot width of the slab in each direction.

DIAPHRAGMS WITH STEEL HEADED STUD ANCHORS

Table 5: Allowable Diaphragm Shear Strengths (plf) and Flexibility Factors (in./lbx10^6) for Decks with Concrete Fill and ¾" Diameter Steel Headed Stud Anchors [1-8, 11, 16, 17, 19]

Concrete Type[9]	Concrete Thickness[10]	Spacing of Steel Headed Stud Anchors[14]							F[12]
		12"	16"	18"	24"	30"	32"	36"	
Minimum Concrete Reinforcement of 0.0025 Times the Area of Fill Above the Deck									
NW	2"[18]	3110	3110	3110	3110	2980	2790	2480	0.40
	2½"	3890	3890	3890	3720	2980	2790	2480	0.32
	3"	4670	4670	4670	3720	2980	2790	2480	0.26
	3½"	5450	5450	4970	3720	2980	2790	2480	0.23
	4½"	7000	5590	4970	3720	2980	2790	2480	0.18
	6"	7450	5590	4970	3720	2980	2790	2480	0.13

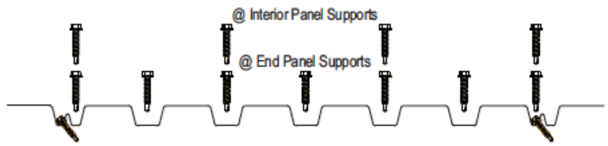
Diaphragm Design (Roof deck)

ASCE7-16 Alt. R_s=1

- Required shear capacity v_r is 2095 lb/ft, to use allowable strength we need to divide it by 1.5 (as recommended by Verco), v_{ra}=1397 lb/ft < v_a=1727 lb/ft

Type HSB®-36-SS

- **36/7/4 Screw Pattern at Supports**
  #12 or #14 SDI Recognized Screws at Supports 0.0385" and thicker
- **Sidelaps Connected with #10 Screws**



Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in./lb)x10^6)

DECK GAGE	SIDELAP ATTACHMENT		SPAN (ft-in.) 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
18	#10 @ 24"	q	792	729	594	568	475	472	411	416	373
		F	4.7+17R	5.4+11R	6.8+7R	6.9+6R	8+4R	7.8+3R	8.7+3R	8.5+2R	9.2+2R
	#10 @ 18"	q	969	729	696	651	551	538	529	469	468
		F	3.8+18R	5.4+11R	5.8+8R	6.1+6R	7+5R	7+4R	7+3R	7.7+3R	7.6+2R
	#10 @ 12"	q	969	854	793	731	688	658	634	616	601
		F	3.8+18R	4.7+11R	5.2+8R	5.5+7R	5.8+5R	5.9+4R	6.1+4R	6.2+3R	6.3+3R
	#10 @ 8"	q	1122	1074	969	949	877	875	829	833	799
		F	3.3+18R	3.8+12R	4.4+9R	4.5+7R	4.8+6R	4.8+5R	5+4R	4.9+4R	5.1+3R
	#10 @ 6"	q	1254	1169	1123	1076	1043	1019	1001	986	975
		F	3+18R	3.6+12R	3.9+9R	4.1+7R	4.2+6R	4.3+5R	4.4+4R	4.4+4R	4.5+4R
	#10 @ 4"	q	1461	1400	1367	1333	1309	1291	1278	1267	1259
		F	2.6+19R	3.1+12R	3.3+9R	3.5+7R	3.6+6R	3.6+5R	3.7+5R	3.7+4R	3.8+4R
16	#10 @ 24"	q	1019	950	774	748	631	633	552	562	504
		F	4.3+9R	4.9+6R	6.1+3R	6.1+3R	7+2R	6.9+1R	7.6+1R	7.4+1R	8+0R
	#10 @ 18"	q	1260	950	914	862	731	719	709	635	636
		F	3.6+10R	4.9+6R	5.2+4R	5.4+3R	6.1+2R	6.1+2R	6.1+2R	6.7+1R	6.6+1R
	#10 @ 12"	q	1260	1121	1046	971	920	882	853	831	813
		F	3.6+10R	4.2+6R	4.6+4R	4.8+3R	5+3R	5.2+2R	5.3+2R	5.4+2R	5.4+1R
	#10 @ 8"	q	1466	1416	1284	1265	1175	1176	1117	1124	1081
		F	3.1+10R	3.4+7R	3.8+5R	3.9+4R	4.1+3R	4.1+3R	4.3+2R	4.3+2R	4.4+2R
	#10 @ 6"	q	1639	1540	1486	1431	1393	1366	1344	1327	1314
		F	2.8+10R	3.2+7R	3.4+5R	3.5+4R	3.6+3R	3.7+3R	3.7+2R	3.8+2R	3.8+2R
	#10 @ 4"	q	1901	1833	1796	1758	1732	1712	1697	1686	1676
		F	2.5+11R	2.8+7R	2.9+5R	3+4R	3.1+3R	3.1+3R	3.2+3R	3.2+2R	3.2+2R

See footnotes on page 28.

Span=6.25 ft
q=1727 lb/ft
F=3.1+3/2=4.6 (in/lb)x10^6

Detailing:

- 16 gage deck
- 36/7/4 #12 screw Pattern at supports
- #10 @4” sidelap attachments

Diaphragm Stiffness

- Diaphragm stiffness per ASCE7-16

$$\Delta = \Delta_s + \Delta_b = \frac{qL^2}{8bG'} + \frac{5qL^4}{384EI} \qquad \text{(DDM04)}$$

$$\Delta = \Delta_s + \Delta_b = \frac{F_x L_d}{8h_d}\frac{F}{10^6} + \frac{5F_x L_d^3}{384EI}$$

$$I_j = 2A_j\left(\frac{h_d}{2}\right)^2,\ I_c = \frac{1}{12}t_c h_d^3$$

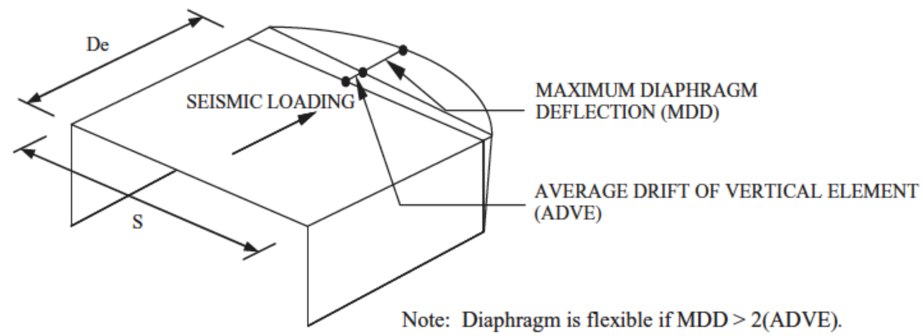


FIGURE 12.3-1 Flexible Diaphragm

F_roof	7.125	in./lbx10^6	Aj	4.7	in^2
F_floors	0.18	in./lbx10^6	Ij	23500	in^4
L_d=	100.00	ft	I_c	6.48E+08	in^4
h_d=	300.00	ft	E_c	3024	ksi

Level	**F_x (kips)**	**δMDD (in)**	**ΔADVE (in)**	**δMDD/ΔADVE**		
Roof	147	0.470	0.42	1.114	**<2**	**not Flexible**
12th	343	0.023	0.47	0.049	**<2**	**not Flexible**
11th	294	0.020	0.47	0.042	**<2**	**not Flexible**
10th	247	0.017	0.46	0.036	**<2**	**not Flexible**
9th	204	0.014	0.45	0.031	**<2**	**not Flexible**
8th	164	0.011	0.41	0.027	**<2**	**not Flexible**
7th	128	0.009	0.38	0.023	**<2**	**not Flexible**
6th	95	0.006	0.36	0.018	**<2**	**not Flexible**
5th	66	0.004	0.34	0.013	**<2**	**not Flexible**
4th	42	0.003	0.31	0.009	**<2**	**not Flexible**
3rd	22	0.001	0.28	0.005	**<2**	**not Flexible**
2nd	8	0.001	0.26	0.002	**<2**	**not Flexible**

* Δ_b is negligible for the composite decks due to high in plane stiffness of the concrete deck.

Diaphragm Chord Design as deep concrete beam (Typical concrete floors)

ASCE7-16 and ASCE7-16 Alt. R_s=3

Chord Design in tension side (Long direction)

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L$ F_y= 60 ksi

Level	T_u (k)	$A_{s\,req}$ (in.^{2})	A_{rebar} (in.^{2})	#8
Roof	98	-	-	-
12th	264	4.885	0.790	7.0
11th	264	4.885	0.790	7.0
10th	264	4.885	0.790	7.0
9th	264	4.885	0.790	7.0
8th	264	4.885	0.790	7.0
7th	264	4.885	0.790	7.0
6th	264	4.885	0.790	7.0
5th	264	4.885	0.790	7.0
4th	264	4.885	0.790	7.0
3rd	264	4.885	0.790	7.0
2nd	264	4.885	0.790	7.0

Chord Design in compression side (Long direction)

$(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L$ f'c= 3000 psi

Level	Z (ft^3)	f_c	0.2f'$_c$	
Roof	-	-	-	-
12th	938	195	600	transverse reinforcement is not required
11th	938	195	600	transverse reinforcement is not required
10th	938	195	600	transverse reinforcement is not required
9th	938	195	600	transverse reinforcement is not required
8th	938	195	600	transverse reinforcement is not required
7th	938	195	600	transverse reinforcement is not required
6th	938	195	600	transverse reinforcement is not required
5th	938	195	600	transverse reinforcement is not required
4th	938	195	600	transverse reinforcement is not required
3rd	938	195	600	transverse reinforcement is not required
2nd	938	195	600	transverse reinforcement is not required

ASCE7-16 Alt. R_s=1

Level	T_u (k)	$A_{s\,req}$ (in.^{2})	A_{rebar} (in.^{2})	#8
Roof	157	-	-	-
12th	422	7.817	0.790	10.0
11th	422	7.817	0.790	10.0
10th	428	7.933	0.790	11.0
9th	439	8.134	0.790	11.0
8th	450	8.336	0.790	11.0
7th	461	8.537	0.790	11.0
6th	472	8.739	0.790	12.0
5th	483	8.940	0.790	12.0
4th	494	9.142	0.790	12.0
3rd	505	9.343	0.790	12.0
2nd	515	9.545	0.790	13.0

Level	Z (ft^3)	f_c	0.2f'$_c$	
Roof	-	-	-	-
12th	938	313	600	transverse reinforcement is not required
11th	938	313	600	transverse reinforcement is not required
10th	938	317	600	transverse reinforcement is not required
9th	938	325	600	transverse reinforcement is not required
8th	938	333	600	transverse reinforcement is not required
7th	938	341	600	transverse reinforcement is not required
6th	938	350	600	transverse reinforcement is not required
5th	938	358	600	transverse reinforcement is not required
4th	938	366	600	transverse reinforcement is not required
3rd	938	374	600	transverse reinforcement is not required
2nd	938	382	600	transverse reinforcement is not required

#8 rebars are provided for the tensile side and since compressive stress in under 0.2f'$_c$ there is no need for transverse reinforcement.

Diaphragm Collector Design as Additional Reinforcement (Typical concrete floors)

ASCE7-16 and ASCE7-16 Alt. R_s=3

Collector Design in tension and compression (Long direction)

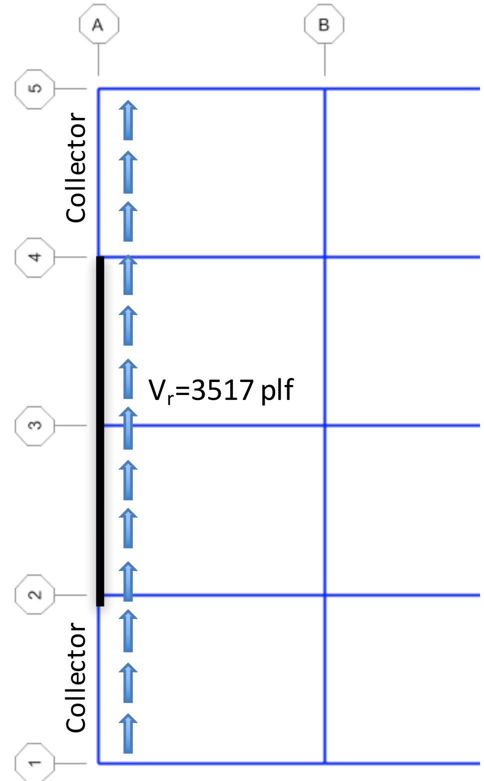
$(1.2 + 0.2S_{DS})D + \Omega_0 Q_E + 0.5L$ Fy= 60 ksi t_c=4.5 in.

B2= 1.065 f'c= 3000 psi

Level	P_u/T_u (k)	$A_{s\,req}$ (in.^{2})	A_{rebar} (in.^{2})	#8	b_{min} (in)
Roof	82	-	-	-	-
12th	220	4.336	0.790	6.0	17
11th	220	4.336	0.790	6.0	17
10th	220	4.336	0.790	6.0	17
9th	220	4.336	0.790	6.0	17
8th	220	4.336	0.790	6.0	17
7th	220	4.336	0.790	6.0	17
6th	220	4.336	0.790	6.0	17
5th	220	4.336	0.790	6.0	17
4th	220	4.336	0.790	6.0	17
3rd	220	4.336	0.790	6.0	17
2nd	220	4.336	0.790	6.0	17

ASCE7-16 Alt. R_s=1

Level	P_u/T_u (k)	$A_{s\,req}$ (in.^{2})	A_{rebar} (in.^{2})	#8	b_{min} (in)
Roof	131	-	-	-	-
12th	352	6.937	0.790	9.0	28
11th	352	6.937	0.790	9.0	28
10th	357	7.040	0.790	9.0	28
9th	366	7.219	0.790	10.0	29
8th	375	7.398	0.790	10.0	30
7th	384	7.577	0.790	10.0	30
6th	393	7.756	0.790	10.0	31
5th	402	7.935	0.790	11.0	32
4th	411	8.113	0.790	11.0	32
3rd	420	8.292	0.790	11.0	33
2nd	430	8.471	0.790	11.0	34



Sample calculation:

Axial force= $\Omega_0 L_{beam} V_r$

Axial force= 2.5 x 25 ft x 3517=220 kips

The collector width, b_{min}, can easily accommodated along line A and M.

$P_r = P_{nt} + B_2 P_{lt} = B_2 P_u$

$b_{min} = P_r/(0.2\, f'_c t_c)$

Diaphragm Collector Design (Shear Studs)

ASCE7-16 and ASCE7-16 Alt. R_s=3

- The maximum required shear capacity (floor 11) $V_r = F_{pi}/2$ = 703/2=351.5 kip. We also need to use <u>amplified seismic demands</u>: $V_{r\text{-}ampl}$=351.5*2.5=878.75 kip
- Allowable capacity of each ¾" diameter stud 7.46 kip. The factored LRFD capacity is 7.46 x1.5=11.19 kip (DDM04: Nominal 21 kip, ϕ=0.55→ Factored 11.55 kip)
- Required number of studs= 878.75/11.19=79
- Stud spacing = 100'/79x12= 15.18"<18" (gravity design)→ Stud Spacing for the exterior girders should be 15" o.c.

Short direction (X- Direction lateral load)

- Stud spacing = 300'/79x12= 70.65">18" (gravity design)→ gravity design stud spacing is adequate.

Diaphragm Collector Design (Shear Studs)

ASCE7-16 Alt. R_s=1

Long direction (Y- Direction lateral load)

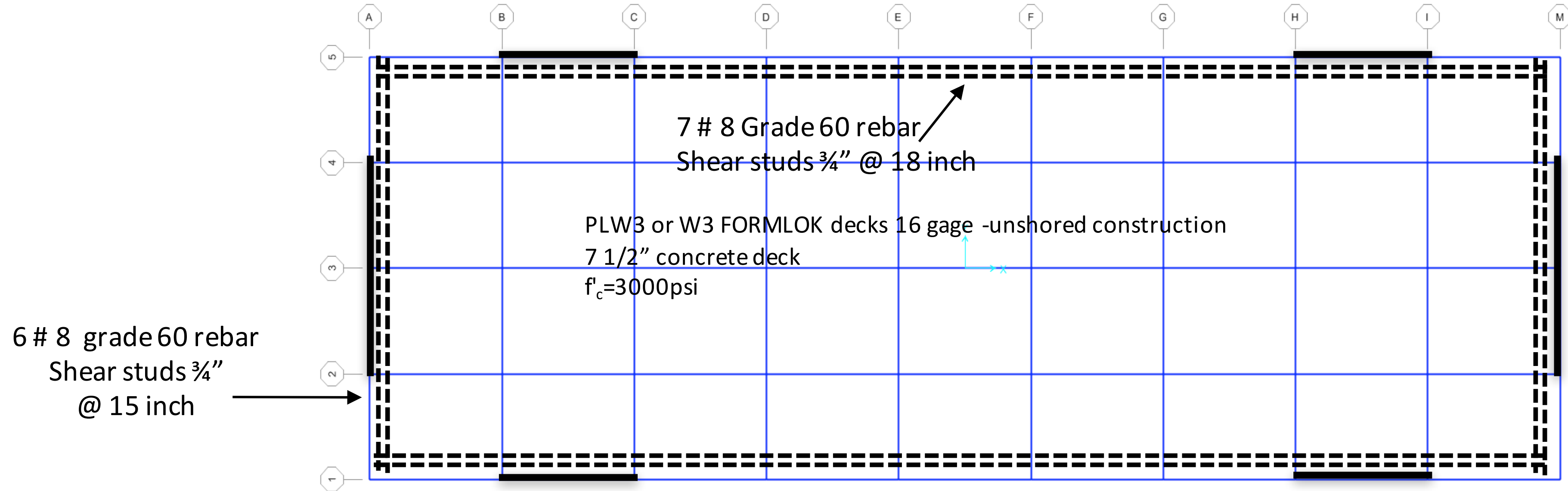
- The maximum required shear capacity (floor 2) $V_r = F_{pi}/2$ = 1374/2=687 kip. We also need to use <u>amplified seismic demands</u>: $V_{r\text{-}ampl}$=687x2.5=1717.5 kip
- Allowable capacity of each ¾" diameter stud 7.46 kip. The factored LRFD capacity is 7.46 x1.5=11.19 kip (DDM04: Nominal 21 kip, ϕ=0.55→ Factored 11.55 kip)
- Required number of studs= 1717.5/11.19=153
- Stud spacing = 100'/153x12= 7.85"<18" (gravity design)→ Stud Spacing for the exterior girders should be 7.5" o.c.

Short direction (X- Direction lateral load)

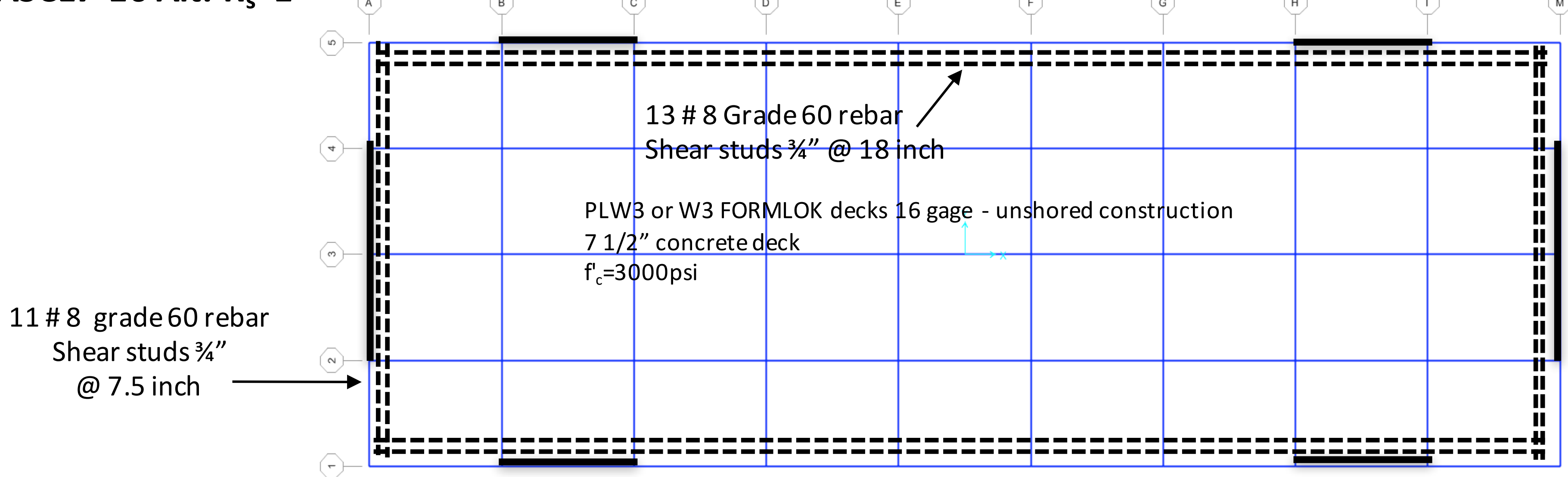
- Stud spacing = 300'/153x12= 23.55">18" (gravity design)→ gravity design stud spacing is adequate.

Diaphragm Design summary (2nd floor concrete floors)

ASCE7-16 and ASCE7-16 Alt. R_s=3



ASCE7-16 Alt. R_s=1



Diaphragm Chord and Collector Design as Perimeter Beams

- Diaphragm chords and collectors are designed as perimeter beams under gravity loads and axial load of the diaphragm. Chord and collector are designed as beam-columns in the roof deck and as composite beam-columns in the typical concrete floor decks.

- The composite beam-column design is performed assuming non- composite axial strength and composite flexural strength.

- The design method is an iterative process and has been performed in the spreadsheets attached to this document.

- Perimeter beams of the composite floors are assumed to be fully braced in minor axis direction and LTB. The beams can still experience torsional-flexural buckling. For the roof deck, the lateral bracing of the collectors are provided at the joists (typically 6.25' o.c.)

- Collectors are designed under amplified seismic loads as moderately ductile members.

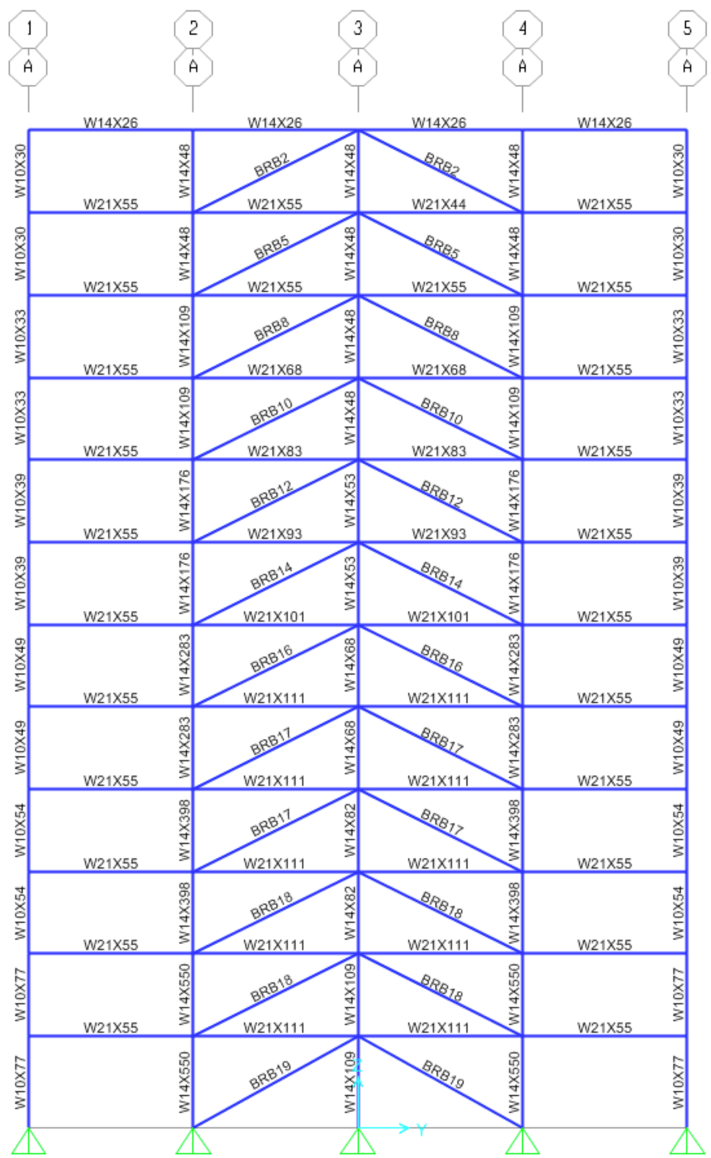
- Chord and collectors are deigned for two different demands in ASCE7-16, including ASCE7-16 and ASCE7-16 Alt. R_s=3 and R_s=1. The design is performed for th

Summary of Diaphragm Chord and Collector Design as Perimeter Beams

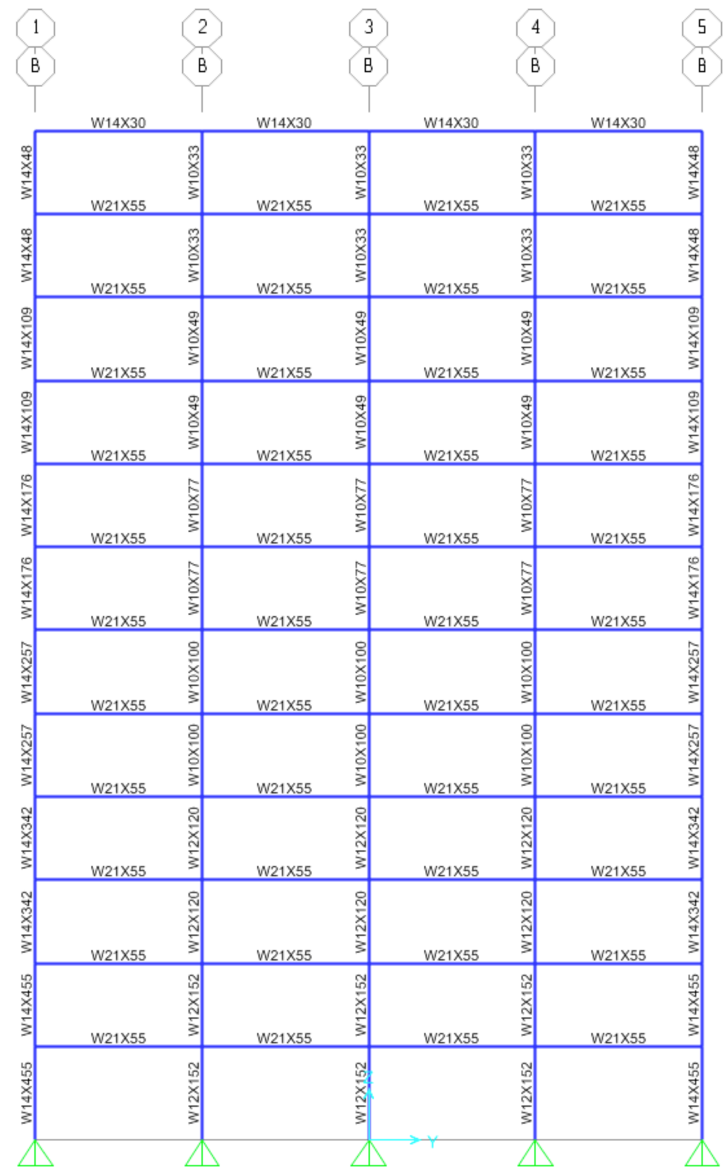
	Element	Location	Cross-section	Length (ft)	No. Studs	S (in)	S_selected	Camber (in)
Floor	Chord	Rs=3	W21x68	33.33	24	16.7	16	0.00
		Rs=1	W24x94	33.33	46	8.7	8	0.00
	Collector	Rs=3	W21x55	25	20	15.0	15	0.00
		Rs=1	W24x76	25	28	10.7	10	0.00
Roof	Chord	Rs=3	W14x30	33.33	-	-	-	-
		Rs=1	W14x38	33.33	-	-	-	-
	Collector	Rs=3	W14x26	25	-	-	-	-
		Rs=1	W14x30	25	-	-	-	-

Chord: Grids 1 and 5 between grids C and H
Collectors: Grids A and M between grids 1 and 5

Frame size summary
(chord and collectors are included)

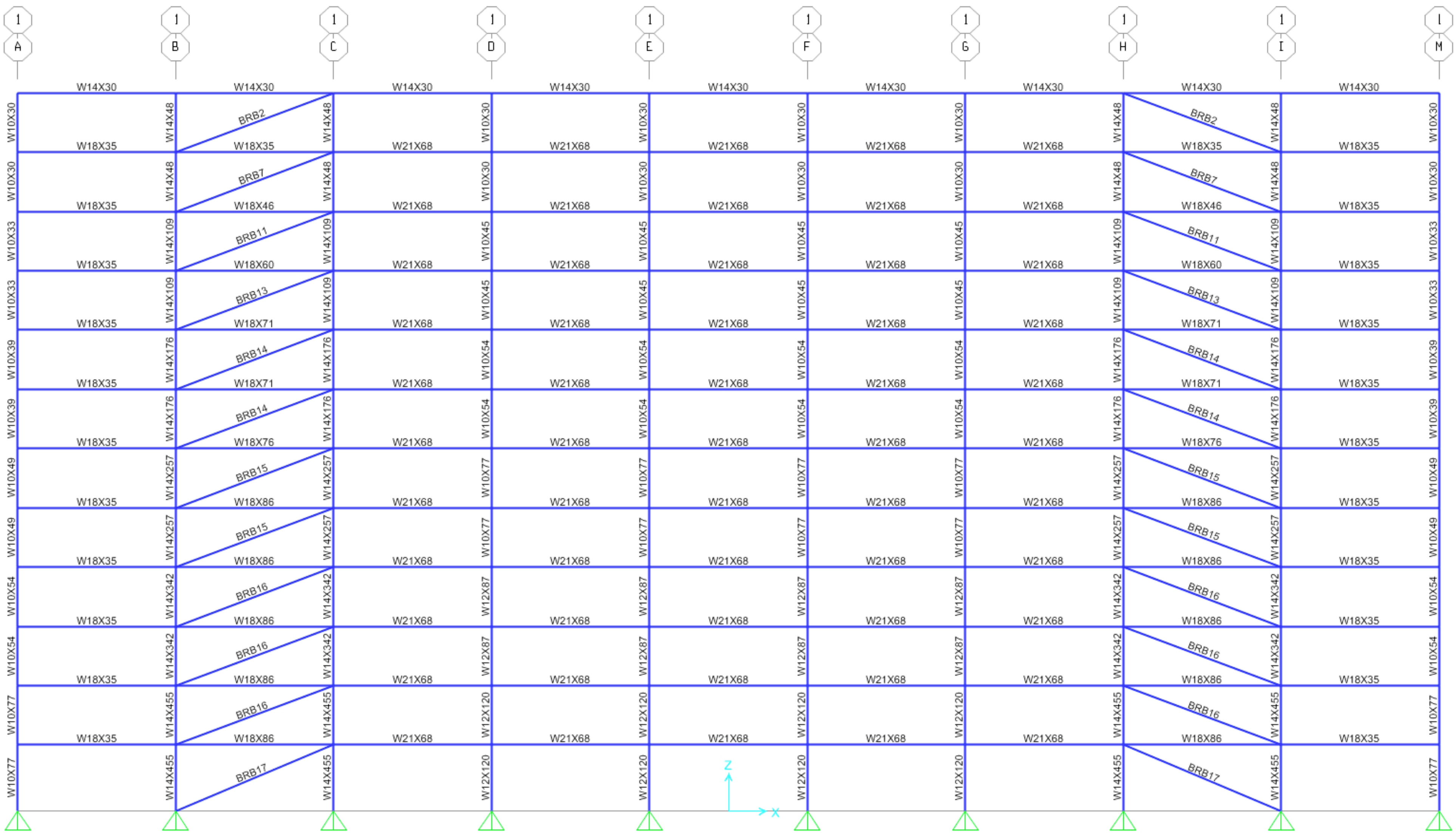


Grid A, Y-direction typical braced frame



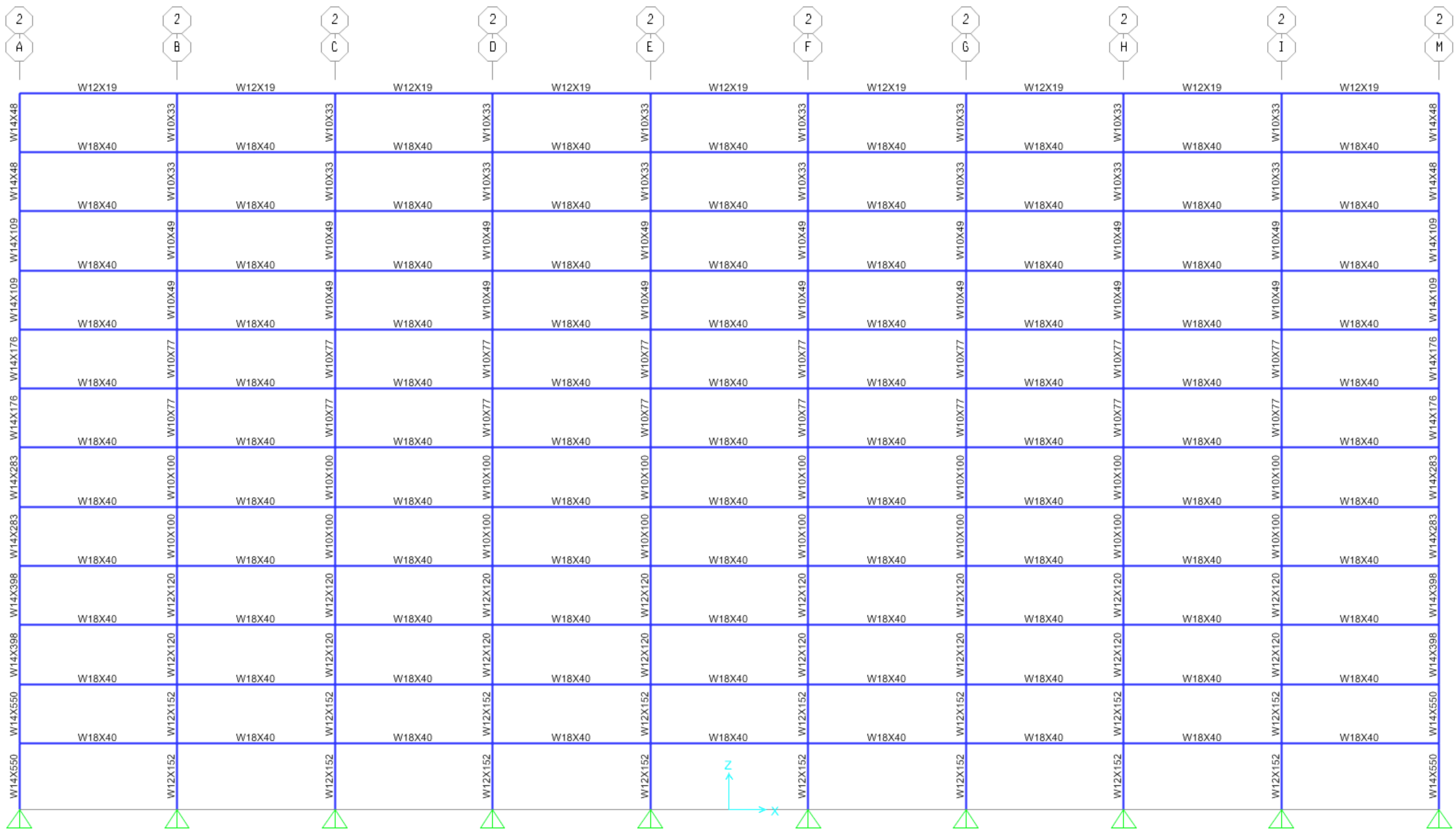
Grid B, Y-direction typical gravity frame

Frame size summary
(chord and collectors are included)



Grid 1, X-direction typical braced frame

Frame size summary
(chord and collectors are included)



Grid 2, X-direction typical gravity frame

Attached documents

Building Models (SAP2000 v19.0.0)

- 2017 07 07 Archetype building_100_300
  Elastic model of the building for gravity and lateral

- 2017 07 07 Archetype building_100_300_updated for chords and collectors
  Elastic model of the building for gravity and lateral including chord and collector designed as perimeter beams.

- 2017 07 07 Archetype building_100_300_ultimate
  Elastic model of the building for gravity and lateral

Spreadsheets:

- 2017 07 07 Archetype_Joist and Beam Design.xls
  (composite beam and beam-column design)
- 2017 07 07 Archetype_Design_100x300.xls
  (gravity and seismic loading and concrete diaphragm design)
- 2017 07 07 Archetype Design Summary_Tables.xls (All Tables and calculations)
- 2017 07 07 BRB_parameters.xls (BRB, KF an YLR factors)