Project #1 – Seismic Design of Steel Building Structure

CE 5506 – Seismic Design of Structures
Instructor: Dr. Zen Hoda, PE, SE

Design Team (Next Gen Structural Engineers):
Justin Vega
Mateo Rico
Oscar Henriquez
Antonio Robles

California State Polytechnic University, Pomona

Department of Civil Engineering

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1. Equivalent Lateral Force (ELF) Analysis

1.1 Building Description

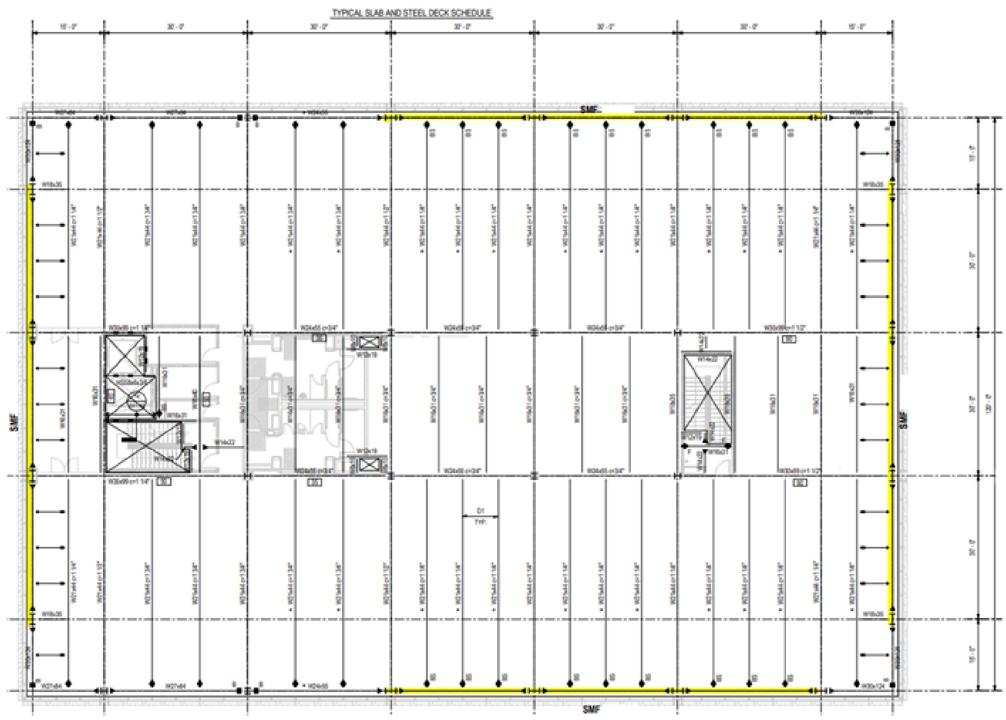

Figure 1. Structural System

Table 1. Structure Information

Length (ft)	180
Width (ft)	120
Area (ft^2)	21600
H_{max} (ft)	60
$H_{1st-story}$ (ft)	16.5
$H_{typ.}$ (ft)	14.5
Parapet Height (ft)	3
Type of Structure	Office

1.2 Design Loads

The following tables summarize the design gravity loads and seismic weight components based on typical office occupancy in accordance with ASCE 7-16.

Table 2. Typical Office Floor Loads

Load Component	Value (psf)	Use
Live Load (reducible)	100	Live Load
Metal deck + 3¼” LW concrete	46	Dead Load
Ceiling	5	Dead Load
Mechanical / Sprinkler	5	Dead Load
Steel Framing	16	Dead Load
Partition Load	10	Seismic Mass Only
Σ=	82	Seismic Weight

Table 3. Typical Office Roof Loads

Load Component	Value (psf)	Use
Roof Live load (reducible)	20	Live Load
Metal deck + 3¼” LW concrete	46	Dead Load
Ceiling	5	Dead Load
Rigid insulation	3	Dead Load
Roofing	6	Dead Load
Mechanical / Sprinkler	5	Dead Load
Steel Framing	16	Dead Load
Partition Load	5	Seismic Mass only
Σ=	86	Seismic Weight

Table 4. Exterior Cladding Loads

Load Component	Value (psf)	Use
Exterior Wall Weight	15	Dead Load
Screen Wall Weight	15	Dead Load

Table 5. Seismic Weight Components

Load Component	Value (psf)
Typical Office Floor	82
Typical Roof	86
Exterior Wall/ Screen Wall	15

Table 6. Seismic Weight

Level	Floor Seismic DL (psf)	Wall Seismic DL (psf)	Slab Area (ft^2)	Wall Area (ft^2)	Weight (k)
R	86	15	21,600	6150	1949.85
3	82	15	21,600	8700	1901.7
2	82	15	21,600	8700	1901.7
1	82	15	21,600	9300	1901.7
-	-	-	-	Σ=	7654.95

The slab area is taken as 180 ft by 120 ft. The building perimeter is 600 ft, and the wall area is calculated by multiplying the perimeter by the corresponding tributary height. The wall area at the roof level accounts for the 3 ft parapet, while for levels 3, 2, and 1, the wall areas are based on the tributary heights taken as half of the story height above and half below each level.

1.3 Fundamental Period

The calculation follows ASCE 7-16 Section 12.8.2:

$$T_a = C_t h_n^x \, (Eq.\ 12.8 - 7)$$

Table 7. Period Parameters

Parameter	Value
C_t	0.028
x	0.8
Structural Height, h_n (ft)	60
Upper Limit factor, C_u	1.4
Approximate fundamental period, T_a (s)	$C_t h_n^x = 0.028 * (60\,ft)^{0.8} = 0.7407\,s$

Per ASCE 7-16 Section 12.8.2, the period may be determined analytically but shall not exceed $C_u T_a$.

1.4 Seismic Design Parameters

Per ASCE 7-16, the seismic base shear is determined using the procedure outlined in 12.8.1. In ASCE 7-22, an additional alternative method has been introduced that allows the base shear to be determined directly from the site-specific design response spectrum obtained from seismic hazard data. The following sections present both approaches for comparison

1.4.1 ASCE 7-22 Method 1

Table 8. Seismic Parameters

Parameter	Value
Site Class	C
Mapped short-period spectral acceleration, S_s	1.27 g
Mapped 1-sec period spectral acceleration (1 s), S_1	0.4 g
Design short-period spectral acceleration, S_{DS}	0.89 g
Design 1-second spectral acceleration, S_{D1}	0.38 g
Risk Category	II
Seismic Design Category, SDC	D
Importance Factor, I_e	1
Response Modification Factor, R (SMF)	8

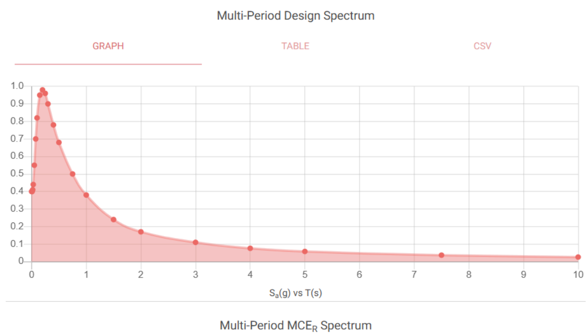

Figure 2. ASCE Hazard Tool Site Design Spectrum

Sa obtained by interpolation from the multi-period design response spectrum (ASCE 7-22 Section 11.4.5.1).

$$C_s = \frac{S_a}{\frac{R}{I_e}} \ (Eq.\ 12.8 - 2)$$

1.4.2 ASCE 7-16 Method

Table 9. Seismic Parameters

Parameter	Value
Site Class	C
Mapped short-period spectral acceleration, S_s	1.11 g
Mapped 1-sec period spectral acceleration (1 s), S_1	0.43 g
Design short-period spectral acceleration, S_{DS}	0.74 g
Design 1-second spectral acceleration, S_{D1}	0.391 g
Risk Category	II
Seismic Design Category, SDC	D
Importance Factor, I_e	1
Response Modification Factor, R (SMF)	8
Long-Period Transition, T_L	8 sec

Table 10. Seismic Base Shear Checks

Parameter	Value
$C_{s1} = \frac{S_{DS}}{\frac{R}{I_e}}\ (Eq.\ 12.8 - 2)$	$C_{s1} = \frac{0.74\,g}{\frac{8}{1}} = 0.0925$
$C_{s2} = \frac{S_{D1}}{T(\frac{R}{I_e})}\ (Eq.\ 12.8 - 3)$	$C_{s2} = \frac{0.391\,g}{1.4*0.7407\ s(\frac{8}{1})} = 0.04713$
$C_{s3} = 0.044 S_{DS} I_e \geq 0.01\ (Eq.\ 12.8 - 5)$	$C_{s3} = 0.044 * 0.74\,g * 1 = 0.03256 \geq 0.01$
Governing C_s	0.04713

C_{s1} shall not exceed C_{s2} and shall not be taken as less than C_{s3}. Equation 12.8-6 is not applicable since the site value of S_1 is less than 0.6.

Table 11. Seismic Response Factor

ASCE 7-22 Method 1	ASCE 7-22 Method 2 (Same as ASCE 7-16)
C_S = 0.047	C_S = 0.04713

In terms of the project design, Method 1 of base shear determination per ASCE 7-22 gives a slightly lower seismic response coefficient than Method 2, which follows the same procedure as ASCE 7-16. For this project, using Method 1 would result in a lower base shear. The difference is negligible for design purposes. However, from a computational standpoint, method 1 provides a simpler approach since it directly uses the spectral acceleration and requires only a single equation.

1.5 Base Shear

The base shear is taken as $V = C_s W\ (Eq.\ 12.8 - 1)$ in accordance with the code.

Table 12. Base Shear

ASCE 7-16	ASCE 7-22 Method 2
V_1 = 0.047 * 7654.95 k = 359.78 k	V_2 = 0.04713 * 7654.95 k = 360.77 k

1.6 Distribution of Forces

The story shear is calculated using the vertical distribution equations prescribed ASCE 7-16. The exponent k is interpolated based on the fundamental period.

Table 13. Story Shears

Level	Height, h_x (ft)	Seismic Weight, w_x (kips)	Exponent factor, k	$w_x h^k_n$	Vertical distribution factor, C_{vx}	Story Force, F_x (kips)	Story Shear, V_x (kips)
Roof	60.0	1949.85	1.27	353389.2146	0.435	157.16	157.16
3	45.5	1901.7	1.27	242558.4899	0.299	107.87	265.04
2	31.0	1901.7	1.27	148994.8844	0.183	66.26	331.30
1	16.5	1901.7	1.27	67204.30788	0.083	29.89	361.19
-	Σ=	7654.95	-	812146.8969	1.000	361.19	-

$F_x = VC_{vx}\ (Eq. 12.8-11)$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i\, h_i^k}\ (Eq. 12.8-12)$$

$$V_x = \sum_{i=x}^{n} F_i\ (Eq. 12.8-13)$$

2. Structural Modeling and Seismic Analysis

The structural model and seismic analysis were developed using the RAM Structural System from Bentley Systems. The floor plan geometry was modeled to closely match the spacing of secondary beams shown in the layout. Bracing elements located along the moment frames at the left and right edges of the floor plan were not included in the analytical model. A 12-inch slab offset was incorporated to better capture actual structural behavior, as opposed to centerline-based hand calculations. Floor diaphragms were modeled as rigid, consisting of a composite Verco W3 Formlok deck with 3 ¼ inches of lightweight concrete topping and 5 ¼-inch studs. Exterior wall loads were applied as line loads along the building perimeter, based on the corresponding tributary weight at each floor.

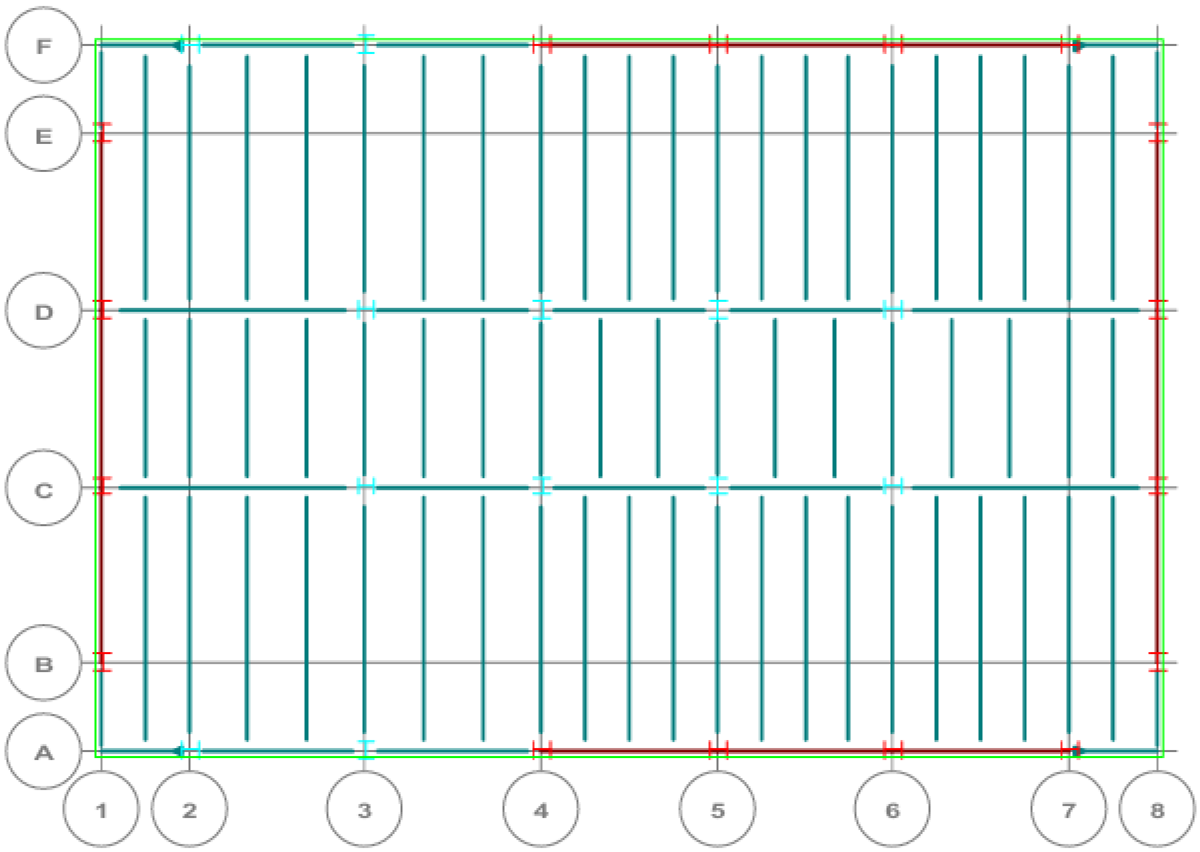

Figure 3. Typical floor plan as seen in RAM Modeler. Origin (0.00, 0.00) designated at A1

2.1 Seismic Weight, period and base shear w/ ELF

Incorporation of the slab offset results in an increase in the calculated seismic weight at each story of approximately 50 kips relative to the hand calculations. The modeled seismic weights are summarized in Table 14, with a total building seismic weight of 7864.48 kips.

Based on the *Loads and Applied Forces* report, the period from the model is 1.461 seconds. This value exceeds the code-prescribed upper limit, C_uT_a; therefore, the period used for the ELF procedure is capped at the code limit in accordance with §12.8.2 of the ASCE 7-16.

Using the governing period, the total ELF base shear was calculated as 370.64 kips.

Table 14. Weight and mass per story from Criteria, Mass, and Exposure Data report

Story	Weight (kips)	Mass (k-s^2/ft)
Roof	2001.79	62.17
4th	1951.23	60.60
3rd	1951.23	60.60
2nd	1951.23	60.88
Σ	7864.48	244.25

2.2 Horizontal and Vertical Irregularities

Horizontal and vertical irregularities were evaluated in accordance with Tables 12.3-1 and 12.3-2 of ASCE 7-16.

Horizontal Irregularities

From the *Drift* report, the maximum ratio of story drifts at any level in either principal direction is 1.080, which is less than the 1.2 threshold. Therefore, torsional and extreme torsional irregularities are not present.

The building plan does not contain reentrant corners; therefore, the reentrant corner irregularity cannot apply.

The diaphragm is continuous with no abrupt openings and consists of a uniform deck throughout. Accordingly, the diaphragm discontinuity irregularity does not apply.

All vertical lateral force-resisting elements are aligned with the primary orthogonal axes of the seismic force-resisting system. Therefore, out-of-plane offset and nonparallel system irregularities are not present.

Vertical Irregularities

Based on the *Center of Rigidity* report, story stiffness increases progressively from the roof to the base of the structure. As a result, neither soft story nor extreme soft story irregularities are present.

Table 15. Story lateral stiffness

Level	Story lateral stiffness (kips/ft)	
	KX	KY
Roof	6304.77	6304.87
4	8542.61	8451.96
3	11966.00	11965.25
2	21975.04	21966.66

As shown in Table 14, the effective mass at each level is consistent with adjacent stories. Therefore, the weight irregularity does not apply.

The moment frame bays are continuous and aligned vertically throughout the height of the structure. Thus, vertical geometric and in-plane discontinuity irregularities are not present.

Member design reflects a strong column-weak beam system. From the *Member Code Check*, the sum of column shear strengths increases from the roof to the base (see Table 16). Therefore, weak story and extreme weak story irregularities are not present.

Table 16. Column Shears and Story Strength

Level	Column shear (kip)				Story Strength (kip)	Ratio
	1	2	3	4		
Roof	31.26	39.63	39.13	15.77	125.79	-
4	16.81	43.36	43.24	25.92	129.33	1.03
3	19.56	55.90	55.50	31.27	162.23	1.25
2	36.53	52.97	52.74	41.45	183.69	1.13

2.3 Verify Redundancy

In accordance with §12.3.4.2 of ASCE 7, the redundancy factor ρ, is taken as 1.0. The structure provides at least three moment frame bays in each orthogonal direction at every story, with each line of resistance contributing more than 35% of the total base shear. Therefore, one of the requirements for a reduced redundancy factor are satisfied.

2.4 Accidental Torsion Amplification

As established in Section 2.2, the structure does not exhibit torsional irregularity. Therefore, accidental torsion amplification in accordance with §12.8.4.2 of ASCE 7 is not required. Consequently, the torsional amplification factor A_x, specified in §12.8.4.3 is not applicable.

2.5 Modal Response Spectrum Analysis

The response spectrum analysis (RSA) was performed using site-specific parameters, including site class, and spectral accelerations S_s and S_1. The fundamental period obtained from the eigenvalue solution is 1.49 seconds.

Modal mass participation reaches 100% by mode 10 out of 12 total modes, satisfying the requirements of §12.9.1.1 of ASCE 7. Modal responses were combined using the Complete Quadratic Combination (CQC) method due to closely spaced modes and significant coupling between translational and torsional responses.

An initial unscaled RSA was used to determine the dynamic base shear V_D. In accordance with §12.9.1.4, the adjusted dynamic shear $V_D/(R/I_e)$, was compared to the ELF base shear V_B. Since the dynamic shear was less than the ELF base shear, the RSA results were scaled to meet the minimum base shear requirement.
For the x-direction:

$$\frac{V_{Dx}}{(R/I_e)} = \frac{2157.64\,k}{(8/1)} = 270\,k$$

$$V_{Dx} = 270\,k < 370.64\,k = V_B$$

Thus, a scale factor of

$$\frac{V_B}{V_{Dx}} = \frac{370.64}{2157.64} = 0.1718$$

was applied to the RSA Strength case.

Results for both orthogonal directions are summarized in Table 17 and documented in the *Building story Shears* report generated by RAM Structural System.

Table 17. Shear comparison and scale factors

Direction	Dynamic shear V_D (kip)	$\frac{V_D}{(R/I_e)}$ (kip)	Scale factor $\frac{V_B}{V_D}$
X	2157.64	270	0.1718
Y	2148.93	269	0.1725

2.6 P-Delta Effects

P-delta effects on the structure are not required to be considered where the stability coefficient θ is equal to or less than 0.10 according to §12.8.7. From the ASCE 7 Stability Coefficients report provided by the RAM Frame analysis, the value for the stability coefficient is 0.091. Thus, P-delta effects need not be considered for this analysis.

2.7 Drift Checks

Story drifts obtained from modal analysis must be scaled when the combined modal base shear is less than C_sW, where C_s is determined per Equation 12.8-6. However, since the structure is located in a region where S_1 is less than 0.6g, the additional scaling requirement of §12.9.1.4.2 does not apply.

Drift results are subject to the same scaling applied to force-based design parameters per § 12.9.1.4 and are further amplified by the factor C_d/I_e. The resulting drift scale factor is

$$\frac{I_e}{R} * \frac{C_d}{I_e} = \frac{C_d}{R} = \frac{5.5}{8} = 0.6875$$

This factor was applied to a separate load case (RSA Drift) to directly obtain inelastic drift values from the RAM *Drift* report. The design story drift is limited by the allowable story drift defined in Table 12.12-1 as

$$\Delta_a = 0.020 h_{sx}$$

Since the seismic force-resisting system consists of moment frames and the structure is classified at seismic design category D, the allowable drift is reduced to Δ_a/ρ, in accordance with Table 12.12-1 and §12.12.1.1.

Story drifts and allowable limits for Gridline A1 are summarized below. Reported drifts include a 1.1 amplification factor to account for the effects of reduced beam sections (RBS) in the moment frame connections.

Evaluation of all four building corners indicates a maximum story drift ratio of 1.18% (including RBS effects), which is within the allowable code limit of 2%. Therefore, the structure satisfies drift requirements.

Table 18. Story drift checks at grid A1

Level	h_{sx} (in)	Δ_a/ρ (in)	Δ (in)
R	174	3.48	1.56
4	174	3.48	1.78
3	174	3.48	1.69
2	198	3.96	1.20

2.8 Building Separation Check

Structural separation must accommodate the maximum inelastic response displacement δ_M, as defined in § 12.12.3 of ASCE 7-16:

$$\delta_M = \frac{C_d \delta_{max}}{I_e}$$

The RSA Drift load case already incorporates the inelastic displacement effects through application of the C_d/R scaling factor. Therefore, the maximum inelastic response displacement is taken directly from the analysis results. Using the maximum drift value and applying a 1.1 amplification factor to account for RBS effects:

$$\delta_M = 5.9026 * 1.1 = 6.49\ in$$

This value is less than the provided building separation of 24 inches. Therefore, the available separation is adequate.

3. Seismic Moment Frame Design

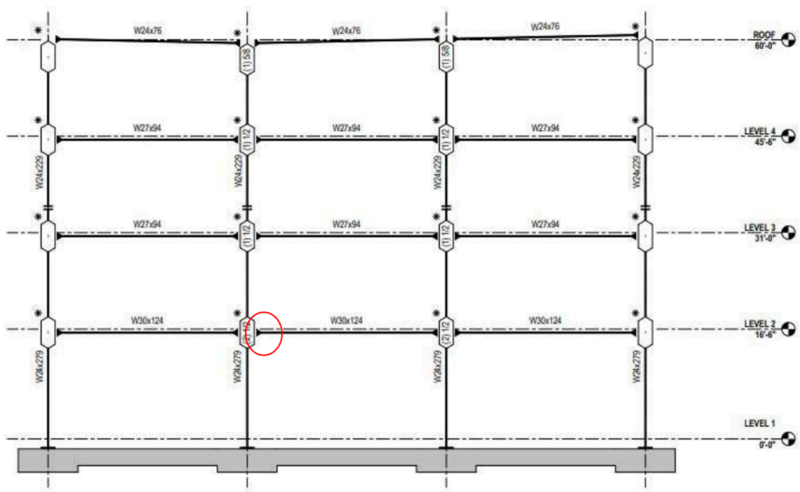

Figure 4. Moment Frame

3.1 Preliminary Design

The red circle region indicates the location of one Reduced Beam Section (RBS), where a portion of the beam flange is intentionally reduced to promote ductile behavior. This region is designed to act as the plastic hinge location, allowing inelastic deformation to occur away from the beam-to-column connection and thereby protecting the connection and column from premature failure.

3.1.1 Material Properties

Reduced Beam Sections (RBS) used in this design consist of rolled wide-flange members in accordance with AISC 341 requirements for Special Moment Frames (SMF). The interior moment frame beams are designed using ASTM A992 structural steel. The material properties used in this analysis are summarized in the table below.

Table 19. Material Properties

Property	Value
Yield Strength, F_y (ksi)	50
Ultimate Strength, F_u (ksi)	65
Overstrength Factor, R_y (ksi)	1.1

3.1.2 Reduced Beam Section (RBS) Geometry

The section properties shown correspond to the first-story beam and column used in the interior moment frame. The W24×279 section represents the column, while the W30×124 section represents the 1st story beam.

Table 20. Section Properties

	W24 x 279	W30 x 124
Width of flange, b_{bf} (in)	13.3	10.5
Depth, d (in)	26.7	30.2
Thickness of flange, t_{bf} (in)	2.09	0.93
Plastic Section modulus, Z_x (in^3)	835	408

The Reduced Beam Section (RBS) geometry is selected in accordance with AISC 341 provisions for Special Moment Frames. The RBS parameters must fall within the specified limits relative to the beam flange width and depth.

Table 21. Reduced Beam Section Geometry Parameters

Parameter	Lower Limit	Upper Limit	Selected Value
a	$0.5b_{bf}$	$0.75b_{bf}$	5.5
b	$0.65d$	$0.85d$	25
c	$0.1b_{bf}$	$0.25b_{bf}$	2.5

The RBS geometry is defined by a (distance from the column face to the start of the cut), b (length of the reduced section), and c (depth of flange reduction at the center). These parameters are selected in accordance with AISC 341 limits and determined iteratively based on the beam span.

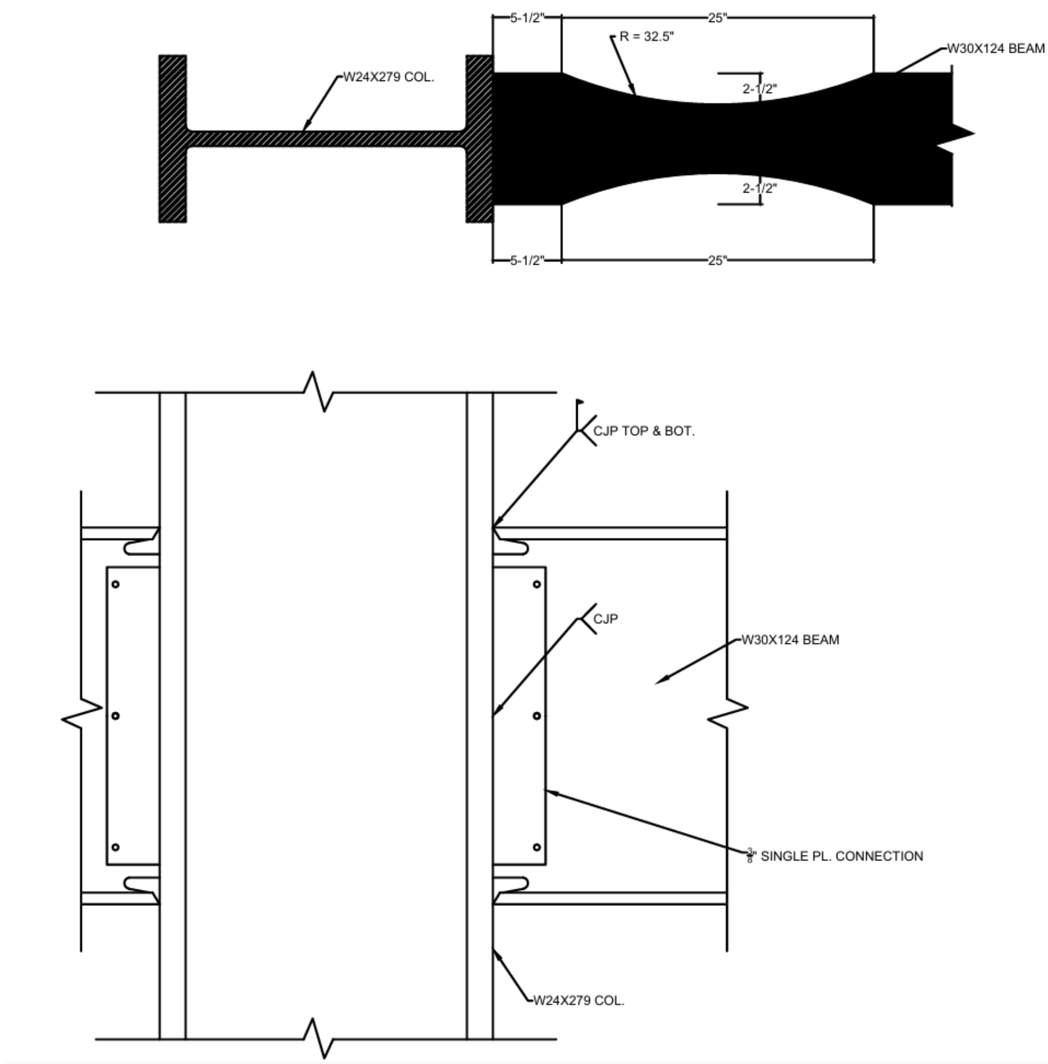

Figure 5. Interior MF Beam to Column Connection

Table 22. Spans for Beam

Parameter	Value
Clear Length, $L_c\ (ft)$	30
Clear Span Length, $L_n\ (ft)$	27.775
Hinge Length, $L_h\ (ft)$	24.775

3.1.3 Shear at Reduced Beam Section

To calculate V_{RBS} a tributary width of 22.5 ft is used, considering both gravity effects and the plastic hinge length. The shear force is determined from the probable maximum moment M_{pr} and the factored load W_u. Because gravity shear is positive at the left end span and negative at the right, while seismic shear remains positive, only the positive moment condition governs this calculation.

3.1.4 Probable Maximum Moment at the Face of the Column

The moment at the column face is calculated from a free-body diagram of the beam segment between the RBS center and the column face.

Table 23. Moments within Span

Property	Value
$S_h\ (in)$	18
Moment at column face, $M_f\ (k - ft)$	1655.64
Max Probable Moments, $M_{pr}\ (k - ft)$	1433.11

∴ W30 x 124 with dimensions RBS cut section a= 5.5 in, b= 25 in, and c= 2.5 in satisfies strength and stability requirements and is expected to yield in the reduced section as intended.

References

American Institute of Steel Construction (AISC). (2016). *Seismic provisions for structural steel buildings* (ANSI/AISC 341-16). Chicago, IL: AISC.

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American Society of Civil Engineers (ASCE). (2017). *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-16). Reston, VA: ASCE.

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Appendix

Material Specifications

- $F_{ye} = R_y F_y$ (expected strength)

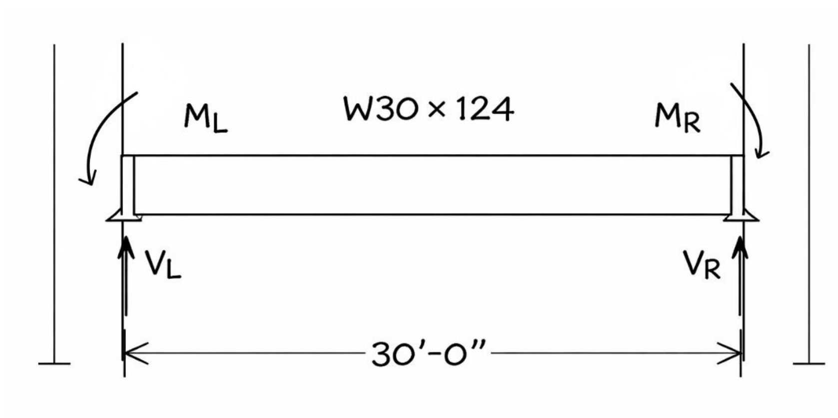
Beam: W30×124 (A992)

- $F_y = 50$ ksi
- $F_u = 65$ ksi

Column: W24×279 (A992)

- $F_y = 50$ ksi
- $F_u = 65$ ksi

<u>Design: Interior Moment Frame Beam (Level 1):</u>



Width–Thickness Checks (Compression)

AISC 341, Table D1.1

Flange: $\frac{b_f}{2t_f} \leq \lambda_{hd} \therefore \frac{10.5}{2(0.83)} \leq \sqrt{\frac{E}{R_y F_y}}(0.32)$ => $5.645 \leq 7.34$✓ OK (W30×124)

Web: $P_u = 0$ (beam) $\Rightarrow$ $C_a = 0 \leq 0.14$ (satisfied)

- $\frac{h}{t_w} \leq 2.57\sqrt{\frac{E}{R_y F_y}}\ (1 - 1.64C_a) \therefore\ 46.2 \leq 59$ ✓ OK

Beam Proportion Limits Check:

- Maximum Beam Depth: $W36 > W30$ ✓
- Maximum Beam Weight: 300 plf $>$ 124 plf ✓

- Maximum Flange Thickness: 1.75 in $>$ 0.93 in ✓
- Minimum Span-to-Depth Ratio: $7 < \frac{30\times12}{30.2} = 11.9$ ✓

Forces from Computer Output (At Face of Joint)

- $M_{DL} = 1{,}136.04$ kip-in, $V_{DL} = 17.48$ kip
- $M_{LL} = 1{,}313.04$ kip-in, $V_{LL} = 19.85$ kip
- $M_{EQ(+)} = \pm 3{,}201$ kip-in, $V_{EQ(+)} = 19.21$ kip
- $M_{EQ(-)} = \pm 3{,}515.5$ kip-in, $V_{EQ(-)} = 21.1$ kip
  Note: Governing forces occur at the right end of the beam.

Load Combinations

1.2D + 1.6L: $M_{D+L} = 1.2(1{,}136.04) + 1.6(1{,}313.04) = 3{,}464.112$ kip-in

- $V_{D+L} = 1.2(17.48) + 1.6(19.85) = 52.726$ kips

Seismic Load Combination 1: $(1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.2S$

- $M_{D+L+EQ} = [1.2 + 0.2(0.74)](1{,}136.04) + 1.0(3{,}515.5) + 0.5(1{,}313.04)$
- $M_{D+L+EQ} = 5{,}703.4$ kip-in
- $V_{D+L+EQ} = [1.2 + 0.2(0.74)](17.48) + 1.0(21.1) + 0.5(19.85)$
- $V_{D+L+E} = 54.588$ kips

Seismic Load Combination 2: $(0.9 - 0.2S_{DS})D - \rho Q_E - 1.6H$

- $M_{D-EQ} = [0.9 - 0.2(0.74)](1{,}136.04) - 3{,}515.5$
- $M_{D-EQ} = -2{,}661.197$ kip-in
- $V_{D-EQ} = [0.9 - 0.2(0.74)](17.48) - 21.1$
- $V_{D-EQ} = -7.955$ kips

Beam Proportion Limits Check ***(AISC 358 / 341)***

- Maximum Beam Depth: $W36 > W30$ ✓ OK
- Maximum Beam Weight: 300 lb/ft $>$ 124 lb/ft ✓ OK
- Maximum Flange Thickness: 1.75 in $>$ 0.93 in ✓ OK
- Minimum Span-to-Depth Ratio: $7 < \frac{30(12)}{30.2} = 11.9$ ✓ OK

Check Bracing: Maximum Brace Spacing *(AISC 341 D1-2b)*

- $L_b = \frac{0.19\, r_y\, E}{R_y F_y}$
- $L_b = \frac{0.19(0.5)(2.23)(29{,}000)}{(1.1)(50)} = 9.3$ ft

Use $L_b = 7.5$ ft < 9.3 ft ✓ OK

Brace Strength / Stability

(AISC 341 D1-4)

- $P_r = 0.06 R_y F_y Z_x/(d - t_f)$
- $P_r = \frac{0.06(1.1)(50)(408)}{30.2-0.93} = 46$ kips

Selected Brace (W21×44): $\phi P_n = 80.9$ kips (max $L_c = 20$ ft)✓ OK

Stiffness of Brace ***(AISC 360 A-6-8)***

- $\beta_{br} = \frac{1}{\phi}\left(\frac{10 \quad {}_c C_d}{L_b h_o}\right)$
- $\beta_{br} = \frac{1}{0.75}\left(\frac{10(1.1)(50)(408)(12)}{(45\times12)(29.27)}\right) = 18.929$ kip/in

Brace Stiffness Provided

- $k = \frac{AE}{L}\cos\,\theta$
- $k = \frac{13(29{,}000)}{45\times12}\cos\left(\tan^{-1}\left(\frac{30.2}{45\times12}\right)\right) = 697.05$ kip/in

$k > \beta_{br}$✓ OK

Check Design Flexural Strength (LRFD)

Section: W30×124

- $L_p > 7.5$ ft
- $\phi_b M_p = 18{,}360$ kip-in

Demand-Capacity Ratio (DCR)

- DCR $= \frac{5{,}703.4}{18{,}360} = 0.31\checkmark$ OK

Check Nominal Shear Strength (LRFD)

- $\phi V_n = 530$ kips

Demand-Capacity Ratio (DCR)

- DCR $= \frac{54.588}{530} = 0.10\checkmark$ OK

Final Selection

$$\boxed{\textbf{Use W30×124}}$$

Design Typical Interior MF Column — W24×279

Maximum Unfactored Column Forces (from RAM)

- $M_D = 193.08$ kip-in, $V_D = 1.14$ kip, $P_D = 193.26$ kip
- $M_L = 137.76$ kip-in, $V_L = 0.84$ kip, $P_L = 88.38$ kip
- $M_{EQ} = \pm 7{,}208.28$ kip-in, $V_{EQ} = \pm 51.01$ kip, $P_{EQ} = \pm 3.46$ kip

---

Load Combinations

Vertical Seismic Effect: $E_v = 0.2S_{DS}D = 0.2(0.74)D = 0.148D$

Combination 1: $(\mathbf{1.2 + 0.2}\boldsymbol{S_{DS}})\boldsymbol{D} + \boldsymbol{\rho Q_E} + \mathbf{0.5}\boldsymbol{L} + \mathbf{0.2}\boldsymbol{S}$

- $P_{D+L+E} = (1.2 + 0.148)(193.26) + 1.0(3.46) + 0.5(88.38) = 308.16$ kips
- $M_{D+L+E} = (1.2 + 0.148)(193.08) + 7{,}208.28 + 0.5(137.76) = 7{,}537.42$ kip-in
- $V_{D+L+E} = (1.2 + 0.148)(1.14) + 51.01 + 0.5(0.84) = 52.466$ kips

---

Combination 2: $(\mathbf{0.9 - 0.2}\boldsymbol{S_{DS}})\boldsymbol{D} - \boldsymbol{\rho Q_E} - \mathbf{1.6}\boldsymbol{H}$

- $P_{D-E} = (0.9 - 0.148)(193.26) - (-3.46) = 148.79$ kips

- $M_{D-E} = (0.9 - 0.148)(193.08) - (-7{,}208.28) = 7{,}353.47$ kip-in
- $V_{D-E} = (0.9 - 0.148)(1.14) - (-51.01) = 51.86$ kips

Section Compactness Checks

Flange Width-to-Thickness Ratio

- $\frac{b}{t} \leq 0.32\sqrt{\frac{E}{R_y F_y}}$ => $3.18 \leq 7.34$✓ OK

Web Width-to-Thickness Ratio

- $C_a = \frac{P_u}{\phi P_y} = \frac{308.16}{0.9(50)(81.19)} = 0.0836$
- $\frac{h}{t_w} \leq 2.57\sqrt{\frac{E}{R_y F_y}}(1 - 1.04C_a)$
- $18.6 \leq 53.8$✓ OK

Column Proportion Checks

Per AISC 341:

- Maximum Column Depth: $W36 > W24$✓
- Maximum Column Weight: Unlimited✓
- Flange Thickness: Compact ✓

Unbraced Column Height

- $L_b = 16.5 \text{ ft} - \left(\frac{15.1}{12}\right) = 15.25 \text{ ft}$
- $L_r = 53.4 \text{ ft}, L_b < L_r$
- $L_p = 11.2 \text{ ft}$

Combined Loading Check *(AISC 360 H1-1b)*

- $\frac{P_u}{P_c} = \frac{308.16}{2820} = 0.109 < 0.2$
- $\frac{P_u}{2P_c} + \frac{M_u}{M_c} = \frac{308.16}{2(2820)} + \frac{7{,}537.42}{35{,}280} = 0.2785 < 1.0$✓

Use W24×279

RBS CALCULATIONS

Beam Span and RBS Geometry: $L_n = 30 - \frac{26.7}{12} = 27.775$ ft

---

Reduced Beam Section (RBS) Properties

Plastic Section Modulus at RBS

- $Z_{RBS} = Z_x - 2c\, t_{bf}(d - t_{bf})$
- $Z_{RBS} = 408 - 2(2.5)(0.93)(30.2) =$
  $271.89\ in^3$

Probable Maximum Moment at RBS: $M_{pr} = C_{pr}\, R_y\, F_y\, Z_{RBS}$

Strain Hardening Factor: $C_{pr} = \frac{F_y + F_u}{2F_y} = \frac{50}{2(50)} = 1.15 \leq 1.2$

Substitution:

- $M_{pr} = (1.15)(1.1)(50)(271.89) = 14{,}331.1$ kip-in=1,433.11 k-ft

---

Shear at Reduced Beam Section: $V_{RBS} = \frac{2M_{pr}}{L_h} + \frac{w_u L_h}{2}$

- M_{pr}: probable maximum moment
- w_u: factored uniform load
- Governing case: positive moment condition

Calculation

- $V_{RBS} = \frac{2(1433.11)}{24.775} + \frac{(2.637)(24.775)}{2} = 148.35$ kips

---

Moment at Column Face

The moment at the column face is obtained from a free-body diagram of the beam segment between the RBS center and the column face:

- $M_f = M_{pr} + V_{RBS}\, S_h = 1433.11 + \frac{(148.35)(18)}{12} = 1655.64$ k-ft

Flexural Strength Check

- $M_{pe} = R_y F_y Z_x = (1.1)(50)(408) = 22{,}440$ kip-in $= 1870$ k-ft
- $\phi M_{pe} = 1870$ k-ft, $M_f = 1655.64$ k-ft

$$M_f < \phi M_{pe} \quad \Rightarrow \quad \checkmark \text{ OK}$$

Design Shear

- $V_u = \frac{2M_{pr}}{L_h} + V_{\text{gravity}} = \frac{2(1433.11)}{24.775} + 32.67 = 148.36$ kips
- $\phi V_n = 530$ kips (AISC Table 3-2)

$$V_u < \phi V_n \quad \Rightarrow \quad \checkmark \text{ OK}$$

Minimum Web Depth Check

- $d_{min} = \frac{V_u}{\phi_v \cdot 0.6 F_y t_w C_v} = \frac{152.4}{(1)(0.6)(50)(0.585)(1)} = 8.68$ in
- Beam depth: $d_{beam} = 30.2$ in
- Conclusion: ✓ Sufficient web depth remains

Flange Force

- $P_f = \frac{0.85 M_f}{d_s \cdot d^*}$ where $d^* = d - t_f$
- $d^* = 30.2 - 0.93 = 29.27$ in
- $P_f = \frac{0.85(17{,}868.21)}{(1)(29.27)} = 576.97$ kip

Flange Local Bending Check

- $\phi R_n = \phi \cdot 6.25 F_y t_f^2 = 1228.5$
- $\phi R_n = 1228.5 > P_f = 576.97$

✓ OK

Web Local Yielding

- $\phi R_n = F_{yw} t_w (5k + l_b) = 50(1.16)[5(2.59) + 0.93] = 805.04$
- $\phi R_n = 805.04 > P_f = 576.97$

✓ OK

Web Local Crippling

- $R_n = 0.8 t_w^2 \left(1+3 \frac{l_b}{d} \left(\frac{t_w}{t_f}\right)^{1.5}\right) \sqrt{\frac{E F_{yw} t_f}{t_w}}$
- $R_n = 1815.1 \Rightarrow \phi R_n = 0.75(1815.1) = 1361.3$
- $\phi R_n = 1361.3 > P_f = 576.97$

✓ OK

Column Flange Thickness Check

- $t_{lim} = \frac{b_{bf}}{6} = \frac{10.5}{6} = 1.75$
- $t_f = 2.09 > t_{lim}$

✓ No continuity plates required

Moment and Shear

- $\sum M_{uv} = (148.4 + 83.1)\left(5.5 + \frac{25}{2} + \frac{26.7}{2}\right) = 7257.5$ kip-in
- $\sum M_{be} = 2M_{pr} + \sum M_{uv} = 2(17197.09) + 7257.5 = 41652$ kip-in

---

Column Shear

- $V_c = \frac{\sum M_{be}}{h_b + h_t} = \frac{41652}{99+87} = 224$ kip

---

Required Shear Strength

- $R_u = \frac{\sum M_f}{d_b - t_{bf}} - V_c$

- $R_u = \frac{(19868.21+18692.84)}{30.2-0.93} - 224 = 1093.5$ kip

---

Axial Load

- $P_r = [1.2 + 0.2(0.71)](192.26) + 4.1 + 0.5(88.38) = 307.5$ kip

---

Axial Limit Check

- $0.75P_y = 0.75F_yA_g = 0.75(50)(81.9) = 3071.25$

$$P_r \leq 0.75P_y \Rightarrow \text{OK}$$

Shear Strength

- $R_n = 0.6F_yd_ct_w\left(1+\frac{3b_{cf}t_{cf}^2}{d_bd_ct_w}\right) = 1102.3$
- $\phi R_n = 1102.3 > R_u = 1093.5$

✔ OK

Strong Column Weak Beam Check: $\frac{\sum M_{pc}^*}{\sum M_{be}^*} > 1.0$

Column Plastic Moment Capacity: $\sum M_{pc}^* = \sum Z_c\left(F_{yc}-\frac{aP_r}{A_g}\right)\left(\frac{h/2}{h/2-d_b/2}\right)$

- $\sum M_{pc}^* = Z_x\left(F_y-\frac{aP_{uc}}{A_g}\right)\left(\frac{h_t}{h_t-\frac{d_b}{2}}\right) + Z_x\left(F_y-\frac{aP_{uc}}{A_g}\right)\left(\frac{h_b}{h_b-\frac{d_b}{2}}\right)$
- $\sum M_{pc}^* = (835)\left(50-\frac{307.5}{81.9}\right)\left[\frac{87}{87-\frac{30.2}{2}}+\frac{99}{99-\frac{30.2}{2}}\right] = 92{,}289$ kip-in

- $\frac{\sum M_{pc}^*}{\sum M_{be}^*} = \frac{92289}{41652} = 2.2 > 1.0$

✔ OK

ASCE 7 Stability Coefficients

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Ground Level:	Base		

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050

Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

STABILITY COEFFICIENTS:ASCE 7-10/16 Eq. (12.8-16)

β = 1.00
Cd : X-Dir = 5.50 Y-Dir = 5.50
Note that the reported drifts are unfactored elastic story drift values.
Calculated vertical load includes dead, live and roof loads. Live loads are reduced with live load reduction factors.
Calculated vertical load is the sum of the total vertical load at and above story.

Vertical Load Factors:

Dead Load : 1.00 Live Load : 1.00 Roof Load : 1.00 Snow Load : 1.00

LOAD CASE: ASCE 7-16

Type : EQ_ASCE716_X_+E_F

Level	**Diaph. #**	**Ht**	**Shear X**	**Shear Y**	**Drift X**	**Drift Y**	**Vertical Load**
		ft	kips	kips	in	in	kips
Roof	1	60.00	165.81	-0.00	0.40	-0.00	1800.19
4th	1	45.50	283.22	0.00	0.50	0.00	4314.29
3rd	1	31.00	356.09	0.00	0.48	0.00	6436.19
2nd	1	16.50	383.35	0.01	0.32	0.00	8630.47

Level	**Diaph. #**	**θx**	**θy**	**θx/(1+θx)**	**θy/(1+θy)**	**θxmax**	**θymax**
Roof	1	0.025	0.000	0.024	0.000	0.091	0.091
4th	1	0.044	0.000	0.042	0.000	0.091	0.091
3rd	1	0.049	0.000	0.047	0.000	0.091	0.091
2nd	1	0.036	0.000	0.035	0.000	0.091	0.091

Type : EQ_ASCE716_X_-E_F

Level	**Diaph. #**	**Ht**	**Shear X**	**Shear Y**	**Drift X**	**Drift Y**	**Vertical Load**

		ft	kips	kips	in	in	kips
Roof	1	60.00	165.81	0.00	0.40	0.00	1800.19
4th	1	45.50	283.22	-0.00	0.50	-0.00	4314.29
3rd	1	31.00	356.09	-0.00	0.48	-0.00	6436.19
2nd	1	16.50	383.35	-0.01	0.32	-0.00	8630.47

Level	**Diaph. #**	**θx**	**θy**	**θx/(1+θx)**	**θy/(1+θy)**	**θxmax**	**θymax**
Roof	1	0.025	0.000	0.024	0.000	0.091	0.091
4th	1	0.044	0.000	0.042	0.000	0.091	0.091
3rd	1	0.049	0.000	0.047	0.000	0.091	0.091
2nd	1	0.036	0.000	0.035	0.000	0.091	0.091

Type : EQ_ASCE716_Y_+E_F

Level	**Diaph. #**	**Ht**	**Shear X**	**Shear Y**	**Drift X**	**Drift Y**	**Vertical Load**
		ft	kips	kips	in	in	kips
Roof	1	60.00	-0.00	165.81	0.00	0.40	1800.19
4th	1	45.50	-0.00	283.22	0.00	0.50	4314.29
3rd	1	31.00	0.00	356.09	0.00	0.48	6436.19
2nd	1	16.50	-0.00	383.34	-0.00	0.32	8630.47

Level	**Diaph. #**	**θx**	**θy**	**θx/(1+θx)**	**θy/(1+θy)**	**θxmax**	**θymax**
Roof	1	0.000	0.025	0.000	0.024	0.091	0.091
4th	1	0.000	0.044	0.000	0.042	0.091	0.091
3rd	1	0.000	0.049	0.000	0.047	0.091	0.091
2nd	1	0.000	0.036	0.000	0.035	0.091	0.091

Type : EQ_ASCE716_Y_-E_F

Level	**Diaph. #**	**Ht**	**Shear X**	**Shear Y**	**Drift X**	**Drift Y**	**Vertical Load**
		ft	kips	kips	in	in	kips
Roof	1	60.00	0.00	165.81	-0.00	0.40	1800.19
4th	1	45.50	-0.00	283.23	-0.00	0.50	4314.29
3rd	1	31.00	-0.00	356.10	-0.00	0.48	6436.19
2nd	1	16.50	0.00	383.36	0.00	0.32	8630.47

Level	**Diaph. #**	**θx**	**θy**	**θx/(1+θx)**	**θy/(1+θy)**	**θxmax**	**θymax**
Roof	1	0.000	0.025	0.000	0.024	0.091	0.091
4th	1	0.000	0.044	0.000	0.042	0.091	0.091
3rd	1	0.000	0.049	0.000	0.047	0.091	0.091
2nd	1	0.000	0.036	0.000	0.035	0.091	0.091

Center of Rigidity

RAM Frame 25.00.02.42
DataBase: Seismic Design 1.5

03/14/26 19:11:01

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Ground Level:	Base		

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050

Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

			Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Type**	**Xr**	**Yr**	**Xm**	**Ym**
			ft	**ft**	**ft**	**ft**
Roof	1	Rigid	90.36	60.00	90.00	60.00
4th	1	Rigid	90.70	60.00	90.00	60.00
3rd	1	Rigid	91.04	60.00	90.00	60.00
2nd	1	Rigid	91.92	60.00	90.00	60.00

			Story Lateral Stiffness	
Level	**Diaph. #**	**Type**	**KX**	**KY**
			kips/ ft	**kips/ ft**
Roof	1	Rigid	6304.77	6304.87
4th	1	Rigid	8542.61	8541.96
3rd	1	Rigid	11966.00	11965.25
2nd	1	Rigid	21975.04	21966.66

NOTES:
Center of rigidity (CR) values given above are only used for load cases that require explicit calculation of CRs for use in calculation of load eccentricities (for example, ASCE 7-05 Wind Load Case).

Note that this information is never used for analysis. On the other hand, it should be noted that analysis results always include any torsional effects due to having center of rigidity and mass center at different locations. In other words, the analysis always accounts for locations and stiffnesses of frame members and diaphragms. Hence, any torsional effects of the masses being offset from the stiffnesses (i.e., CR) are implicitly and correctly accounted in the analysis.

The reported story stiffness is the inverse of the interstory drift that is calculated according to a unit load applied at the story.

Criteria, Mass and Exposure Data

CRITERIA:

Rigid End Zones:	Ignore Effects
Member Force Output:	At Face of Joint
P-Delta:	No
Ground Level:	Base

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050

Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

DIAPHRAGM DATA:

Story	Diaph #	Diaph Type
Roof	1	Rigid
4th	1	Rigid
3rd	1	Rigid
2nd	1	Rigid

Disconnect Internal Nodes of Beams:	Yes
Disconnect Nodes outside Slab Boundary:	Yes

STORY MASS DATA:

Includes Self Mass of:

Self masses not automatically included.

Calculated Values:

Story	Diaph #	Weight kips	Mass k-s2/ft	MMI ft-k-s2	Xm ft	Ym ft	EccX ft	EccY ft
Roof	1	2001.79	62.17	258737	90.00	60.00	9.10	6.10
4th	1	1951.23	60.60	256611	90.00	60.00	9.10	6.10
3rd	1	1951.23	60.60	256611	90.00	60.00	9.10	6.10
2nd	1	1960.23	60.88	258707	90.00	60.00	9.10	6.10

Story	Diaph #	Combine
Roof	1	None

4th	1	None
3rd	1	None
2nd	1	None

WIND EXPOSURE DATA:

Calculated Values:

Story	Diaph #	Building Extents (ft)				Expose	Parapet
		Min X	Max X	Min Y	Max Y		ft
Roof	1	-1.00	181.00	-1.00	121.00	Full	0.00
4th	1	-1.00	181.00	-1.00	121.00	Full	0.00
3rd	1	-1.00	181.00	-1.00	121.00	Full	0.00
2nd	1	-1.00	181.00	-1.00	121.00	Full	0.00

STORY GRAVITY LOADS DATA:

Includes Weight of:

Beams
Columns

Live Load Reduction (Calculated)

Reducible : 60.00 %
Storage : 0.00 %
Roof : 40.00 %

Calculated Values:

Story	Diaph #	Dead kips	Xc ft	Yc ft	Live kips	Xc ft	Yc ft
Roof	1	1530.28	90.68	60.00	0.00	0.00	0.00
4th	1	1338.37	90.85	60.00	888.16	90.00	60.00
3rd	1	1351.35	90.95	60.00	888.16	90.00	60.00
2nd	1	1375.47	91.13	60.00	888.16	90.00	60.00

Story	Diaph #	Roof kips	Xc ft	Yc ft	Combine
Roof	1	266.45	90.00	60.00	None
4th	1	0.00	0.00	0.00	None
3rd	1	0.00	0.00	0.00	None
2nd	1	0.00	0.00	0.00	None

Loads and Applied Forces

LOAD CASE: ASCE 7-16

Seismic ASCE 7-16 Equivalent Lateral Force

Importance Factor: 1.00 TL: 8.00 s

Site Class C: Very Dense Soil and Soft Rock

Ss: 1.100 g S1: 0.430 g

Use Specified: SDs: 0.740 g SD1: 0.391 g

Risk Category: II Seismic Design Category: D

Provisions for: Force

Ground Level: Base

Dir	Eccent	R	Ta Equation	Building Period-T
X	+ And -	8.00	Std,Ct=0.028,x=0.80	Calculated
Y	+ And -	8.00	Std,Ct=0.028,x=0.80	Calculated

Dir	Ta	Cu	T	T-used	Cs Eq12.8-2	Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
X	0.741	1.400	1.461	1.037	0.093	0.047	0.033	0.047	1.269

Dir	Ta	Cu	T	T-used	Cs Eq12.8-2	Cs(max) Eq12.8-3	Cs(min) Eq12.8-5	Cs-used	k
Y	0.741	1.400	1.461	1.037	0.093	0.047	0.033	0.047	1.269

Total Building Weight (kips) = 7864.48

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_+E_F

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Roof	1	60.00	161.26	0.00	90.00	66.10
4th	1	45.50	110.66	0.00	90.00	66.10
3rd	1	31.00	68.01	0.00	90.00	66.10
2nd	1	16.50	30.70	0.00	90.00	66.10

APPLIED STORY FORCES

Type: EQ_ASCE716_X_+E_F

Level	Ht ft	Fx kips	Fy kips
Roof	60.00	161.26	0.00
4th	45.50	110.66	0.00
3rd	31.00	68.01	0.00
2nd	16.50	30.70	0.00
		370.64	0.00

Loads and Applied Forces

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_X_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	60.00	161.26	0.00	90.00	53.90
4th	1	45.50	110.66	0.00	90.00	53.90
3rd	1	31.00	68.01	0.00	90.00	53.90
2nd	1	16.50	30.70	0.00	90.00	53.90

APPLIED STORY FORCES

Type: EQ_ASCE716_X_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	60.00	161.26	0.00
4th	45.50	110.66	0.00
3rd	31.00	68.01	0.00
2nd	16.50	30.70	0.00
		370.64	0.00

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	60.00	0.00	161.26	99.10	60.00
4th	1	45.50	0.00	110.66	99.10	60.00
3rd	1	31.00	0.00	68.01	99.10	60.00
2nd	1	16.50	0.00	30.70	99.10	60.00

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	60.00	0.00	161.26
4th	45.50	0.00	110.66
3rd	31.00	0.00	68.01
2nd	16.50	0.00	30.70
		0.00	370.64

Loads and Applied Forces

APPLIED DIAPHRAGM FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Roof	1	60.00	0.00	161.26	80.90	60.00
4th	1	45.50	0.00	110.66	80.90	60.00
3rd	1	31.00	0.00	68.01	80.90	60.00
2nd	1	16.50	0.00	30.70	80.90	60.00

APPLIED STORY FORCES

Type: EQ_ASCE716_Y_-E_F

Level	Ht	Fx	Fy
	ft	kips	kips
Roof	60.00	0.00	161.26
4th	45.50	0.00	110.66
3rd	31.00	0.00	68.01
2nd	16.50	0.00	30.70
		0.00	370.64

Building Story Shears

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Face of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Ground Level:	Base		

Mesh Criteria :

Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050

Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

Load Case: D   DeadLoad   RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.41	0.00
4th	1	0.63	0.00
3rd	1	0.68	0.00
2nd	1	0.35	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.41	0.41	0.00	0.00
4th	0.63	0.22	0.00	0.00
3rd	0.68	0.06	0.00	0.00
2nd	0.35	-0.33	0.00	0.00

Load Case: Lp   PosLiveLoad   RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.33	0.00
4th	1	0.72	0.00
3rd	1	0.64	0.00
2nd	1	0.31	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips

Roof	0.33	0.33	0.00	0.00
4th	0.72	0.39	0.00	-0.00
3rd	0.64	-0.08	0.00	0.00
2nd	0.31	-0.33	0.00	0.00

Load Case: Rfp PosRoofLiveLoad RAMUSER

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.08	0.00
4th	1	-0.01	0.00
3rd	1	0.01	0.00
2nd	1	0.00	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.08	0.08	0.00	0.00
4th	-0.01	-0.09	0.00	0.00
3rd	0.01	0.02	0.00	-0.00
2nd	0.00	-0.01	0.00	0.00

Load Case: E1 ELF EQ_ASCE716_X_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	165.81	-0.00
4th	1	283.22	0.00
3rd	1	356.09	0.00
2nd	1	383.35	0.01

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	165.81	165.81	-0.00	-0.00
4th	283.22	117.42	0.00	0.00
3rd	356.09	72.87	0.00	0.00
2nd	383.35	27.26	0.01	0.00

Load Case: E2 ELF EQ_ASCE716_X_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	165.81	0.00
4th	1	283.22	-0.00

3rd	1	356.09	-0.00
2nd	1	383.35	-0.01

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	165.81	165.81	0.00	0.00
4th	283.22	117.42	-0.00	-0.00
3rd	356.09	72.87	-0.00	-0.00
2nd	383.35	27.26	-0.01	-0.00

Load Case: E3 ELF EQ_ASCE716_Y_+E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	-0.00	165.81
4th	1	-0.00	283.22
3rd	1	0.00	356.09
2nd	1	-0.00	383.34

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.00	-0.00	165.81	165.81
4th	-0.00	0.00	283.22	117.41
3rd	0.00	0.00	356.09	72.87
2nd	-0.00	-0.00	383.34	27.26

Load Case: E4 ELF EQ_ASCE716_Y_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.00	165.81
4th	1	-0.00	283.23
3rd	1	-0.00	356.10
2nd	1	0.00	383.36

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	0.00	0.00	165.81	165.81
4th	-0.00	-0.00	283.23	117.42
3rd	-0.00	-0.00	356.10	72.87
2nd	0.00	0.00	383.36	27.27

Building Story Shears

Load Case: Dyn1 RSA Seed Dyn_ASCE716_CQC_X_+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	1112.96	6.65
4th	1	1468.33	-6.36
3rd	1	1810.19	-6.79
2nd	1	2157.64	-9.90

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	1112.96	1112.96	6.65	6.65
4th	1468.33	-848.97	-6.36	7.48
3rd	1810.19	-985.78	-6.79	-7.48
2nd	2157.64	-832.77	-9.90	8.34

Load Case: Dyn2 RSA Seed Dyn_ASCE716_CQC_X_-E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	1112.96	-6.65
4th	1	1468.33	6.36
3rd	1	1810.19	6.79
2nd	1	2157.64	9.90

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	1112.96	1112.96	-6.65	-6.65
4th	1468.33	-848.97	6.36	-7.48
3rd	1810.19	-985.78	6.79	7.48
2nd	2157.64	-832.77	9.90	-8.34

Load Case: Dyn3 RSA Seed Dyn_ASCE716_CQC_Y_+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.00	1109.45
4th	1	0.00	1461.89
3rd	1	0.00	1802.04
2nd	1	0.00	2148.93

Summary - Total Story Shears

Building Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.00	-0.00	1109.45	1109.45
4th	-0.00	-0.00	1461.89	-846.52
3rd	-0.00	0.00	1802.04	-983.78
2nd	-0.00	-0.00	2148.93	-831.53

Load Case: Dyn4 RSA Seed Dyn_ASCE716_CQC_Y_-E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.00	1105.82
4th	1	0.00	1458.44
3rd	1	0.00	1797.96
2nd	1	0.00	2144.58

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.00	-0.00	1105.82	1105.82
4th	-0.00	0.00	1458.44	-842.28
3rd	-0.00	-0.00	1797.96	-980.40
2nd	-0.00	0.00	2144.58	-829.86

Load Case: Dyn5 RSA Strength Dyn_ASCE716_CQC_X_+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	191.21	1.14
4th	1	252.26	-1.09
3rd	1	310.99	-1.17
2nd	1	370.68	-1.70

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	191.21	191.21	1.14	1.14
4th	252.26	-145.85	-1.09	1.28
3rd	310.99	-169.36	-1.17	-1.29
2nd	370.68	-143.07	-1.70	1.43

Load Case: Dyn6 RSA Strength Dyn_ASCE716_CQC_X_-E

Level	Diaph. #	Shear-X kips	Shear-Y kips

Roof	1	191.21	-1.14
4th	1	252.26	1.09
3rd	1	310.99	1.17
2nd	1	370.68	1.70

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	191.21	191.21	-1.14	-1.14
4th	252.26	-145.85	1.09	-1.28
3rd	310.99	-169.36	1.17	1.29
2nd	370.68	-143.07	1.70	-1.43

Load Case: Dyn7 RSA Strength Dyn_ASCE716_CQC_Y_+E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.00	191.38
4th	1	0.00	252.18
3rd	1	0.00	310.85
2nd	1	0.00	370.69

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.00	-0.00	191.38	191.38
4th	-0.00	-0.00	252.18	-146.02
3rd	-0.00	0.00	310.85	-169.70
2nd	-0.00	-0.00	370.69	-143.44

Load Case: Dyn8 RSA Strength Dyn_ASCE716_CQC_Y_-E

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	0.00	190.75
4th	1	0.00	251.58
3rd	1	0.00	310.15
2nd	1	0.00	369.94

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.00	-0.00	190.75	190.75
4th	-0.00	0.00	251.58	-145.29

Building Story Shears

3rd	-0.00	-0.00	310.15	-169.12
2nd	-0.00	0.00	369.94	-143.15

BASE SHEAR (Dynamic Load Cases)

LdC	Shear-X	Shear-Y
Dyn1	**-2157.64**	**9.90**
Dyn2	**-2157.64**	**-9.90**
Dyn3	**0.00**	**-2148.93**
Dyn4	**0.00**	**-2144.58**
Dyn5	**-370.68**	**1.70**
Dyn6	**-370.68**	**-1.70**
Dyn7	**0.00**	**-370.69**
Dyn8	**0.00**	**-369.94**

Drift

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Rfp	PosRoofLiveLoad	RAMUSER
E1	ELF	EQ_ASCE716_X_+E_F
E2	ELF	EQ_ASCE716_X_-E_F
E3	ELF	EQ_ASCE716_Y_+E_F
E4	ELF	EQ_ASCE716_Y_-E_F
Dyn9	RSA Drift	Dyn_ASCE716_CQC_X_+E
Dyn10	RSA Drift	Dyn_ASCE716_CQC_X_-E
Dyn11	RSA Drift	Dyn_ASCE716_CQC_Y_+E
Dyn12	RSA Drift	Dyn_ASCE716_CQC_Y_-E

RESULTS:

Location (ft): (0.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0920	-0.0000	0.0354	0.0000	0.0002	0.0000
	Lp	0.1152	-0.0000	0.0285	-0.0000	0.0002	0.0000
	Rfp	0.0085	0.0000	0.0060	-0.0000	0.0000	0.0000
	E1	1.6372	0.0784	0.3834	0.0180	0.0022	0.0001
	E2	1.7413	-0.0784	0.4075	-0.0180	0.0023	0.0001
	E3	0.0741	1.5776	0.0183	0.3681	0.0001	0.0021
	E4	-0.0812	1.8117	-0.0177	0.4219	0.0001	0.0024
	Dyn9	5.3775	0.4601	1.4137	0.1190	0.0081	0.0007
	Dyn10	5.9026	-0.4601	1.5533	-0.1190	0.0089	0.0007
	Dyn11	0.4318	5.0679	0.1133	1.3299	0.0007	0.0076
	Dyn12	-0.4743	6.2221	-0.1222	1.6354	0.0007	0.0094
4th	D	0.0566	-0.0000	0.0275	-0.0000	0.0002	0.0000
	Lp	0.0868	-0.0000	0.0386	-0.0000	0.0002	0.0000
	Rfp	0.0025	0.0000	0.0018	0.0000	0.0000	0.0000

	E1	1.2537	0.0604	0.4822	0.0230	0.0028	0.0001
	E2	1.3337	-0.0604	0.5128	-0.0230	0.0029	0.0001
	E3	0.0558	1.2095	0.0219	0.4645	0.0001	0.0027
	E4	-0.0636	1.3897	-0.0237	0.5333	0.0001	0.0031
	Dyn9	4.1287	0.3550	1.6171	0.1380	0.0093	0.0008
	Dyn10	4.5325	-0.3550	1.7753	-0.1380	0.0102	0.0008
	Dyn11	0.3294	3.8953	0.1294	1.5241	0.0007	0.0088
	Dyn12	-0.3683	4.7850	-0.1426	1.8719	0.0008	0.0108
3rd	D	0.0290	-0.0000	0.0201	-0.0000	0.0001	0.0000
	Lp	0.0481	-0.0000	0.0329	-0.0000	0.0002	0.0000
	Rfp	0.0006	0.0000	0.0005	0.0000	0.0000	0.0000
	E1	0.7715	0.0374	0.4615	0.0221	0.0027	0.0001
	E2	0.8209	-0.0374	0.4908	-0.0221	0.0028	0.0001
	E3	0.0339	0.7450	0.0207	0.4449	0.0001	0.0026
	E4	-0.0398	0.8565	-0.0230	0.5109	0.0001	0.0029
	Dyn9	2.6080	0.2248	1.5351	0.1312	0.0088	0.0008
	Dyn10	2.8640	-0.2248	1.6847	-0.1312	0.0097	0.0008
	Dyn11	0.2061	2.4634	0.1226	1.4476	0.0007	0.0083
	Dyn12	-0.2348	3.0288	-0.1355	1.7766	0.0008	0.0102
2nd	D	0.0089	-0.0000	0.0089	-0.0000	0.0000	0.0000
	Lp	0.0152	-0.0000	0.0152	-0.0000	0.0001	0.0000
	Rfp	0.0001	0.0000	0.0001	0.0000	0.0000	0.0000
	E1	0.3100	0.0152	0.3100	0.0152	0.0016	0.0001
	E2	0.3301	-0.0152	0.3301	-0.0152	0.0017	0.0001
	E3	0.0132	0.3001	0.0132	0.3001	0.0001	0.0015
	E4	-0.0168	0.3456	-0.0168	0.3456	0.0001	0.0017
	Dyn9	1.0949	0.0954	1.0949	0.0954	0.0055	0.0005
	Dyn10	1.2037	-0.0954	1.2037	-0.0954	0.0061	0.0005
	Dyn11	0.0847	1.0372	0.0847	1.0372	0.0004	0.0052
	Dyn12	-0.1014	1.2786	-0.1014	1.2786	0.0005	0.0065

Location (ft): (0.000, 120.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0920	-0.0000	0.0354	0.0000	0.0002	0.0000
	Lp	0.1152	-0.0000	0.0285	-0.0000	0.0002	0.0000
	Rfp	0.0085	0.0000	0.0060	-0.0000	0.0000	0.0000

	E1	1.7413	0.0784	0.4075	0.0180	0.0023	0.0001
	E2	1.6372	-0.0784	0.3834	-0.0180	0.0022	0.0001
	E3	-0.0741	1.5776	-0.0183	0.3681	0.0001	0.0021
	E4	0.0812	1.8117	0.0177	0.4219	0.0001	0.0024
	Dyn9	5.9026	0.4601	1.5533	0.1190	0.0089	0.0007
	Dyn10	5.3775	-0.4601	1.4137	-0.1190	0.0081	0.0007
	Dyn11	-0.4318	5.0679	-0.1133	1.3299	0.0007	0.0076
	Dyn12	0.4743	6.2221	0.1222	1.6354	0.0007	0.0094
4th	D	0.0566	-0.0000	0.0275	-0.0000	0.0002	0.0000
	Lp	0.0868	-0.0000	0.0386	-0.0000	0.0002	0.0000
	Rfp	0.0025	0.0000	0.0018	0.0000	0.0000	0.0000
	E1	1.3337	0.0604	0.5128	0.0230	0.0029	0.0001
	E2	1.2537	-0.0604	0.4822	-0.0230	0.0028	0.0001
	E3	-0.0558	1.2095	-0.0219	0.4645	0.0001	0.0027
	E4	0.0636	1.3897	0.0237	0.5333	0.0001	0.0031
	Dyn9	4.5325	0.3550	1.7753	0.1380	0.0102	0.0008
	Dyn10	4.1287	-0.3550	1.6171	-0.1380	0.0093	0.0008
	Dyn11	-0.3294	3.8953	-0.1294	1.5241	0.0007	0.0088
	Dyn12	0.3683	4.7850	0.1426	1.8719	0.0008	0.0108
3rd	D	0.0290	-0.0000	0.0201	-0.0000	0.0001	0.0000
	Lp	0.0481	-0.0000	0.0329	-0.0000	0.0002	0.0000
	Rfp	0.0006	0.0000	0.0005	0.0000	0.0000	0.0000
	E1	0.8209	0.0374	0.4908	0.0221	0.0028	0.0001
	E2	0.7715	-0.0374	0.4615	-0.0221	0.0027	0.0001
	E3	-0.0339	0.7450	-0.0207	0.4449	0.0001	0.0026
	E4	0.0398	0.8565	0.0230	0.5109	0.0001	0.0029
	Dyn9	2.8640	0.2248	1.6847	0.1312	0.0097	0.0008
	Dyn10	2.6080	-0.2248	1.5351	-0.1312	0.0088	0.0008
	Dyn11	-0.2061	2.4634	-0.1226	1.4476	0.0007	0.0083
	Dyn12	0.2348	3.0288	0.1355	1.7766	0.0008	0.0102
2nd	D	0.0089	-0.0000	0.0089	-0.0000	0.0000	0.0000
	Lp	0.0152	-0.0000	0.0152	-0.0000	0.0001	0.0000
	Rfp	0.0001	0.0000	0.0001	0.0000	0.0000	0.0000
	E1	0.3301	0.0152	0.3301	0.0152	0.0017	0.0001
	E2	0.3100	-0.0152	0.3100	-0.0152	0.0016	0.0001
	E3	-0.0132	0.3001	-0.0132	0.3001	0.0001	0.0015
	E4	0.0168	0.3456	0.0168	0.3456	0.0001	0.0017
	Dyn9	1.2037	0.0954	1.2037	0.0954	0.0061	0.0005

	Dyn10	1.0949	-0.0954	1.0949	-0.0954	0.0055	0.0005
	Dyn11	-0.0847	1.0372	-0.0847	1.0372	0.0004	0.0052
	Dyn12	0.1014	1.2786	0.1014	1.2786	0.0005	0.0065

Location (ft): (180.000, 120.000)

Story	**LdC**	**Displacement**		**Story Drift**		**Drift Ratio**	
		X	**Y**	**X**	**Y**	**X**	**Y**
		in	**in**	**in**	**in**		
Roof	D	0.0920	0.0000	0.0354	-0.0000	0.0002	0.0000
	Lp	0.1152	0.0000	0.0285	0.0000	0.0002	0.0000
	Rfp	0.0085	-0.0000	0.0060	0.0000	0.0000	0.0000
	E1	1.7413	-0.0777	0.4075	-0.0181	0.0023	0.0001
	E2	1.6372	0.0777	0.3834	0.0181	0.0022	0.0001
	E3	-0.0741	1.7999	-0.0183	0.4229	0.0001	0.0024
	E4	0.0812	1.5680	0.0177	0.3689	0.0001	0.0021
	Dyn9	5.9026	-0.4612	1.5533	0.1203	0.0089	0.0007
	Dyn10	5.3775	0.4612	1.4137	-0.1203	0.0081	0.0007
	Dyn11	-0.4318	6.1702	-0.1133	1.6284	0.0007	0.0094
	Dyn12	0.4743	5.0058	0.1222	1.3167	0.0007	0.0076
4th	D	0.0566	0.0000	0.0275	0.0000	0.0002	0.0000
	Lp	0.0868	0.0000	0.0386	0.0000	0.0002	0.0000
	Rfp	0.0025	-0.0000	0.0018	-0.0000	0.0000	0.0000
	E1	1.3337	-0.0596	0.5128	-0.0229	0.0029	0.0001
	E2	1.2537	0.0596	0.4822	0.0229	0.0028	0.0001
	E3	-0.0558	1.3769	-0.0219	0.5303	0.0001	0.0030
	E4	0.0636	1.1991	0.0237	0.4621	0.0001	0.0027
	Dyn9	4.5325	-0.3543	1.7753	-0.1384	0.0102	0.0008
	Dyn10	4.1287	0.3543	1.6171	0.1384	0.0093	0.0008
	Dyn11	-0.3294	4.7321	-0.1294	1.8560	0.0007	0.0107
	Dyn12	0.3683	3.8386	0.1426	1.5049	0.0008	0.0086
3rd	D	0.0290	0.0000	0.0201	0.0000	0.0001	0.0000
	Lp	0.0481	0.0000	0.0329	0.0000	0.0002	0.0000
	Rfp	0.0006	-0.0000	0.0005	-0.0000	0.0000	0.0000
	E1	0.8209	-0.0368	0.4908	-0.0219	0.0028	0.0001
	E2	0.7715	0.0368	0.4615	0.0219	0.0027	0.0001
	E3	-0.0339	0.8467	-0.0207	0.5071	0.0001	0.0029
	E4	0.0398	0.7370	0.0230	0.4418	0.0001	0.0025
	Dyn9	2.8640	-0.2234	1.6847	-0.1312	0.0097	0.0008

	Dyn10	2.6080	0.2234	1.5351	0.1312	0.0088	0.0008
	Dyn11	-0.2061	2.9868	-0.1226	1.7599	0.0007	0.0101
	Dyn12	0.2348	2.4214	0.1355	1.4287	0.0008	0.0082
2nd	D	0.0089	0.0000	0.0089	0.0000	0.0000	0.0000
	Lp	0.0152	0.0000	0.0152	0.0000	0.0001	0.0000
	Rfp	0.0001	-0.0000	0.0001	-0.0000	0.0000	0.0000
	E1	0.3301	-0.0149	0.3301	-0.0149	0.0017	0.0001
	E2	0.3100	0.0149	0.3100	0.0149	0.0016	0.0001
	E3	-0.0132	0.3396	-0.0132	0.3396	0.0001	0.0017
	E4	0.0168	0.2952	0.0168	0.2952	0.0001	0.0015
	Dyn9	1.2037	-0.0939	1.2037	-0.0939	0.0061	0.0005
	Dyn10	1.0949	0.0939	1.0949	0.0939	0.0055	0.0005
	Dyn11	-0.0847	1.2519	-0.0847	1.2519	0.0004	0.0063
	Dyn12	0.1014	1.0128	0.1014	1.0128	0.0005	0.0051

Location (ft): (180.000, 0.000)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
Roof	D	0.0920	0.0000	0.0354	-0.0000	0.0002	0.0000
	Lp	0.1152	0.0000	0.0285	0.0000	0.0002	0.0000
	Rfp	0.0085	-0.0000	0.0060	0.0000	0.0000	0.0000
	E1	1.6372	-0.0777	0.3834	-0.0181	0.0022	0.0001
	E2	1.7413	0.0777	0.4075	0.0181	0.0023	0.0001
	E3	0.0741	1.7999	0.0183	0.4229	0.0001	0.0024
	E4	-0.0812	1.5680	-0.0177	0.3689	0.0001	0.0021
	Dyn9	5.3775	-0.4612	1.4137	0.1203	0.0081	0.0007
	Dyn10	5.9026	0.4612	1.5533	-0.1203	0.0089	0.0007
	Dyn11	0.4318	6.1702	0.1133	1.6284	0.0007	0.0094
	Dyn12	-0.4743	5.0058	-0.1222	1.3167	0.0007	0.0076
4th	D	0.0566	0.0000	0.0275	0.0000	0.0002	0.0000
	Lp	0.0868	0.0000	0.0386	0.0000	0.0002	0.0000
	Rfp	0.0025	-0.0000	0.0018	-0.0000	0.0000	0.0000
	E1	1.2537	-0.0596	0.4822	-0.0229	0.0028	0.0001
	E2	1.3337	0.0596	0.5128	0.0229	0.0029	0.0001
	E3	0.0558	1.3769	0.0219	0.5303	0.0001	0.0030
	E4	-0.0636	1.1991	-0.0237	0.4621	0.0001	0.0027
	Dyn9	4.1287	-0.3543	1.6171	-0.1384	0.0093	0.0008

Drift

	Dyn10	4.5325	0.3543	1.7753	0.1384	0.0102	0.0008
	Dyn11	0.3294	4.7321	0.1294	1.8560	0.0007	0.0107
	Dyn12	-0.3683	3.8386	-0.1426	1.5049	0.0008	0.0086
3rd	D	0.0290	0.0000	0.0201	0.0000	0.0001	0.0000
	Lp	0.0481	0.0000	0.0329	0.0000	0.0002	0.0000
	Rfp	0.0006	-0.0000	0.0005	-0.0000	0.0000	0.0000
	E1	0.7715	-0.0368	0.4615	-0.0219	0.0027	0.0001
	E2	0.8209	0.0368	0.4908	0.0219	0.0028	0.0001
	E3	0.0339	0.8467	0.0207	0.5071	0.0001	0.0029
	E4	-0.0398	0.7370	-0.0230	0.4418	0.0001	0.0025
	Dyn9	2.6080	-0.2234	1.5351	-0.1312	0.0088	0.0008
	Dyn10	2.8640	0.2234	1.6847	0.1312	0.0097	0.0008
	Dyn11	0.2061	2.9868	0.1226	1.7599	0.0007	0.0101
	Dyn12	-0.2348	2.4214	-0.1355	1.4287	0.0008	0.0082
2nd	D	0.0089	0.0000	0.0089	0.0000	0.0000	0.0000
	Lp	0.0152	0.0000	0.0152	0.0000	0.0001	0.0000
	Rfp	0.0001	-0.0000	0.0001	-0.0000	0.0000	0.0000
	E1	0.3100	-0.0149	0.3100	-0.0149	0.0016	0.0001
	E2	0.3301	0.0149	0.3301	0.0149	0.0017	0.0001
	E3	0.0132	0.3396	0.0132	0.3396	0.0001	0.0017
	E4	-0.0168	0.2952	-0.0168	0.2952	0.0001	0.0015
	Dyn9	1.0949	-0.0939	1.0949	-0.0939	0.0055	0.0005
	Dyn10	1.2037	0.0939	1.2037	0.0939	0.0061	0.0005
	Dyn11	0.0847	1.2519	0.0847	1.2519	0.0004	0.0063
	Dyn12	-0.1014	1.0128	-0.1014	1.0128	0.0005	0.0051

TORSIONAL IRREGULARITY DATA:

X-Axis:

Story	**LdC**	**Drift**	**Coord**	**Drift**	**Coord**	**Max/Min**	**Max/Ave**
		in	ft	in	ft		
Roof	Dyn10	1.5533	(0.00 0.00)	1.4137	(0.00 120.00)	1.099	1.047
4th	Dyn10	1.7753	(0.00 0.00)	1.6171	(0.00 120.00)	1.098	1.047
3rd	Dyn10	1.6847	(0.00 0.00)	1.5351	(0.00 120.00)	1.097	1.046
2nd	Dyn10	1.2037	(0.00 0.00)	1.0949	(0.00 120.00)	1.099	1.047

Y-Axis:

Story	**LdC**	**Drift**	**Coord**	**Drift**	**Coord**	**Max/Min**	**Max/Ave**
		in	ft	in	ft		
Roof	Dyn12	1.6354	(0.00 0.00)	1.3167	(180.00 120.00)	1.242	1.108

4th	Dyn12	1.8719	(0.00 0.00)	1.5049	(180.00 120.00)	1.244	1.109
3rd	Dyn12	1.7766	(0.00 0.00)	1.4287	(180.00 120.00)	1.243	1.109
2nd	Dyn12	1.2786	(0.00 0.00)	1.0128	(180.00 120.00)	1.262	1.116

Member Code Check

COLUMN INFORMATION:

Story Level	= Roof	Frame Number	= 4	Column Number=	1
Fy (ksi)	= 50.00				
Column Size	= W24X229				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	-31.26
		Vuy (kip)	0.01
Shear	Bot.	Vux (kip)	-31.26
		Vuy (kip)	0.01

SHEAR CHECK:

Vux (kip)	= -31.26	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.042
Vuy (kip)	= 0.01	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	= 66.25	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.022
		0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.028
		0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.028

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	56.32
Moment	Top	Mux (kip-ft)	-59.40
		Muy (kip-ft)	0.00
Moment	Bot.	Mux (kip-ft)	-182.44
		Muy (kip-ft)	0.16

CALCULATED PARAMETERS:

Pu (kip)	= 56.32	0.90Pnx (kip)	=	2965.76
		0.90Pny (kip)	=	2406.25
Mux (kip-ft)	= -182.44	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	= 0.16	0.90Mny (kip-ft)	=	577.50
		Mcx (kip-ft)	=	2429.91
KL/Rx	= 16.31	KL/Ry	=	55.90

Cbx = 1.37

INTERACTION EQUATION:

Pu/0.90*Pn=0.019

Eq H1-3: 0.035 + 0.003 = 0.038

Eq H1-1b Per H1.3: 0.009 + 0.072 + 0.000 = 0.082

Member Code Check

COLUMN INFORMATION:

Story Level = Roof   Frame Number = 4   Column Number= 2

Fy (ksi) = 50.00

Column Size = W24X229

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	39.63
		Vuy (kip)	-0.01
Shear	Bot.	Vux (kip)	39.63
		Vuy (kip)	-0.01

SHEAR CHECK:

Vux (kip)	=	39.63	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.053
Vuy (kip)	=	-0.01	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	70.77	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.024
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.029
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.029

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	70.77
Moment	Top	Mux (kip-ft)	-274.46
		Muy (kip-ft)	0.00
Moment	Bot.	Mux (kip-ft)	-223.34
		Muy (kip-ft)	-0.15

CALCULATED PARAMETERS:

Pu (kip)	=	70.77	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-274.46	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Member Code Check

Cbx	=	1.08

INTERACTION EQUATION:

Pu/0.90*Pn=0.024

Eq H1-3: 0.044 + 0.011 = 0.055

Eq H1-1b Per H1.3: 0.012 + 0.108 + 0.000 = 0.120

Member Code Check

COLUMN INFORMATION:

Story Level = Roof Frame Number = 4 Column Number= 3
Fy (ksi) = 50.00
Column Size = W24X229

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	39.13
		Vuy (kip)	-0.03
Shear	Bot.	Vux (kip)	39.13
		Vuy (kip)	-0.03

SHEAR CHECK:

Vux (kip)	=	39.13	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.052
Vuy (kip)	=	-0.03	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn7

AXIAL CHECK:

Pu (kip)	=	69.03	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.023
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.029
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.029

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	68.54
Moment	Top	Mux (kip-ft)	-270.26
		Muy (kip-ft)	0.00
Moment	Bot.	Mux (kip-ft)	-225.42
		Muy (kip-ft)	-0.41

CALCULATED PARAMETERS:

Pu (kip)	=	68.54	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-270.26	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Cbx	=	1.07

INTERACTION EQUATION:

Pu/0.90*Pn=0.023

Eq H1-3: 0.042 + 0.011 = 0.053

Eq H1-1b Per H1.3: 0.012 + 0.107 + 0.000 = 0.118

Member Code Check

COLUMN INFORMATION:

Story Level = Roof   Frame Number = 4   Column Number= 4

Fy (ksi) = 50.00

Column Size = W24X229

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	-15.77
		Vuy (kip)	-0.05
Shear	Bot.	Vux (kip)	-15.77
		Vuy (kip)	-0.05

SHEAR CHECK:

Vux (kip)	=	-15.77	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.021
Vuy (kip)	=	-0.05	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	74.40	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.025
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.031
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.031

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	74.40
Moment	Top	Mux (kip-ft)	-168.57
		Muy (kip-ft)	-0.00
Moment	Bot.	Mux (kip-ft)	-99.81
		Muy (kip-ft)	-0.69

CALCULATED PARAMETERS:

Pu (kip)	=	74.40	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-168.57	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	-0.00	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Cbx	=	1.19

INTERACTION EQUATION:

Pu/0.90*Pn=0.025

Eq H1-3: 0.046 + 0.003 = 0.049

Eq H1-1b Per H1.3: 0.013 + 0.067 + 0.000 = 0.079

Member Code Check

BEAM INFORMATION:

Story Level = Roof   Frame Number = 4   Beam Number= 1

Fy (ksi) = 50.00

Beam Size = W24X76

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-36.32	1.00Vnx (kip)	=	315.48	Vux/1.00Vnx	=	0.115
Vuy (kip)	=	-0.00	0.90Vny (kip)	=	330.11	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00

j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1008.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1008.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1008.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1008.00
Mux (kip-ft)	=	-276.62	0.90Mnx (kip-ft)	=	750.00
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	107.25
Cbx	=	1.50			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.369

Member Code Check

BEAM INFORMATION:

Story Level = Roof   Frame Number = 4   Beam Number= 2
Fy (ksi) = 50.00
Beam Size = W24X76

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-34.90	1.00Vnx (kip)	=	315.48	Vux/1.00Vnx	=	0.111
Vuy (kip)	=	0.00	0.90Vny (kip)	=	330.11	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00
j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1008.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1008.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1008.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1008.00
Mux (kip-ft)	=	-266.06	0.90Mnx (kip-ft)	=	750.00
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	107.25
Cbx	=	1.50			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.355

Member Code Check

BEAM INFORMATION:

Story Level	=	Roof	Frame Number	=	4	Beam Number=	3
Fy (ksi)	=	50.00					
Beam Size	=	W24X76					

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end	22.50
j - end	30.00

SHEAR CHECK:

Vux (kip)	=	-35.70	1.00Vnx (kip)	=	315.48	Vux/1.00Vnx	=	0.113
Vuy (kip)	=	0.00	0.90Vny (kip)	=	330.11	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end	0.00
j - end	7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1008.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1008.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1008.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end	22.50
j - end	30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1008.00
Mux (kip-ft)	=	-281.76	0.90Mnx (kip-ft)	=	750.00
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	107.25
Cbx	=	1.47			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.376

Member Code Check

COLUMN INFORMATION:

Story Level =	4th	Frame Number =	4	Column Number=	1
Fy (ksi) =	50.00				
Column Size =	W24X229				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	16.81
		Vuy (kip)	0.02
Shear	Bot.	Vux (kip)	16.81
		Vuy (kip)	0.02

SHEAR CHECK:

Vux (kip) =	16.81	1.00Vnx (kip) =	748.80	Vux/1.00Vnx =	0.022
Vuy (kip) =	0.02	0.90Vny (kip) =	1223.80	Vuy/0.90Vny =	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip) =	143.43	0.90Pnx (kip) =	2965.76	Pu/0.90Pnx =	0.048
		0.90Pny (kip) =	2406.25	Pu/0.90Pny =	0.060
		0.90Pn (kip) =	2406.25	Pu/0.90Pn =	0.060

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

Axial		Load (kip)	143.43
Moment	Top	Mux (kip-ft)	69.08
		Muy (kip-ft)	-9.97
Moment	Bot.	Mux (kip-ft)	-26.57
		Muy (kip-ft)	-10.37

CALCULATED PARAMETERS:

Pu (kip) =	143.43	0.90Pn (kip) =	2406.25
Mux (kip-ft) =	69.08	0.90Mnx (kip-ft) =	2531.25
Muy(kip-ft) =	-9.97	0.90Mny (kip-ft) =	577.50
KL/Rx =	16.31	KL/Ry =	55.90
Cbx =	2.15		

INTERACTION EQUATION:

Pu/0.90*Pn=0.060
Eq H1-1b: 0.030 + 0.027 + 0.017 = 0.074

Member Code Check

COLUMN INFORMATION:

Story Level = 4th   Frame Number = 4   Column Number= 2

Fy (ksi) = 50.00

Column Size = W24X229

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	43.36
		Vuy (kip)	0.01
Shear	Bot.	Vux (kip)	43.36
		Vuy (kip)	0.01

SHEAR CHECK:

Vux (kip)	=	43.36	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.058
Vuy (kip)	=	0.01	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	152.40	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.051
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.063
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.063

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	152.40
Moment	Top	Mux (kip-ft)	-284.74
		Muy (kip-ft)	-0.15
Moment	Bot.	Mux (kip-ft)	267.05
		Muy (kip-ft)	-0.15

CALCULATED PARAMETERS:

Pu (kip)	=	152.40	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-284.74	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	-0.15	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Cbx	=	2.26

INTERACTION EQUATION:

Pu/0.90*Pn=0.051

Eq H1-3: 0.093 + 0.003 = 0.096

Eq H1-1b Per H1.3: 0.026 + 0.112 + 0.000 = 0.138

COLUMN INFORMATION:

Story Level = 4th		Frame Number = 4	Column Number= 3
Fy (ksi)	= 50.00		
Column Size	= W24X229		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	43.24
		Vuy (kip)	-0.04
Shear	Bot.	Vux (kip)	43.24
		Vuy (kip)	-0.04

SHEAR CHECK:

Vux (kip)	=	43.24	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	= 0.058
Vuy (kip)	=	-0.04	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	= 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	=	149.20	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	= 0.050
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	= 0.062
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	= 0.062

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	147.89
Moment	Top	Mux (kip-ft)	-284.22
		Muy (kip-ft)	-0.41
Moment	Bot.	Mux (kip-ft)	266.15
		Muy (kip-ft)	-0.40

CALCULATED PARAMETERS:

Pu (kip)	=	147.89	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-284.22	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	-0.41	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Cbx	=	2.26

INTERACTION EQUATION:

Pu/0.90*Pn=0.050

Eq H1-3: 0.090 + 0.003 = 0.093

Eq H1-1b Per H1.3: 0.025 + 0.112 + 0.000 = 0.137

COLUMN INFORMATION:

Story Level = 4th   Frame Number = 4   Column Number= 4
Fy (ksi) = 50.00
Column Size = W24X229

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	25.92
		Vuy (kip)	-0.06
Shear	Bot.	Vux (kip)	25.92
		Vuy (kip)	-0.06

SHEAR CHECK:

Vux (kip)	=	25.92	1.00Vnx (kip)	=	748.80	Vux/1.00Vnx	=	0.035
Vuy (kip)	=	-0.06	0.90Vny (kip)	=	1223.80	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	168.13	0.90Pnx (kip)	=	2965.76	Pu/0.90Pnx	=	0.057
			0.90Pny (kip)	=	2406.25	Pu/0.90Pny	=	0.070
			0.90Pn (kip)	=	2406.25	Pu/0.90Pn	=	0.070

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	168.13
Moment	Top	Mux (kip-ft)	-181.18
		Muy (kip-ft)	-0.69
Moment	Bot.	Mux (kip-ft)	175.03
		Muy (kip-ft)	-0.67

CALCULATED PARAMETERS:

Pu (kip)	=	168.13	0.90Pnx (kip)	=	2965.76
			0.90Pny (kip)	=	2406.25
Mux (kip-ft)	=	-181.18	0.90Mnx (kip-ft)	=	2531.25
Muy(kip-ft)	=	-0.69	0.90Mny (kip-ft)	=	577.50
			Mcx (kip-ft)	=	2429.91
KL/Rx	=	16.31	KL/Ry	=	55.90

Cbx	=	2.27

INTERACTION EQUATION:

Pu/0.90*Pn=0.070

Eq H1-1b Per H1.3: 0.028 + 0.072 + 0.000 = 0.100

Eq H1-3: 0.102 + 0.001 = 0.103

BEAM INFORMATION:

Story Level = 4th Frame Number = 4 Beam Number= 1
Fy (ksi) = 50.00
Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-50.67	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.128
Vuy (kip)	=	-0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00
j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-417.94	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.45			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.401

Member Code Check

BEAM INFORMATION:

Story Level = 4th   Frame Number = 4   Beam Number= 2
Fy (ksi) = 50.00
Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-49.26	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.125
Vuy (kip)	=	0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00
j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-407.94	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.44			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.391

Member Code Check

BEAM INFORMATION:

Story Level = 4th Frame Number = 4 Beam Number= 3

Fy (ksi) = 50.00

Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-50.22	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.127
Vuy (kip)	=	0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00

j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-427.46	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.43			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.410

Member Code Check

COLUMN INFORMATION:

Story Level	= 3rd	Frame Number = 4		Column Number= 1
Fy (ksi)	= 50.00			
Column Size	= W24X279			

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	19.56
		Vuy (kip)	0.01
Shear	Bot.	Vux (kip)	19.56
		Vuy (kip)	0.01

SHEAR CHECK:

Vux (kip)	= 19.56	1.00Vnx (kip)	= 929.16	Vux/1.00Vnx	= 0.021
Vuy (kip)	= 0.01	0.90Vny (kip)	= 1501.04	Vuy/0.90Vny	= 0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	= 216.23	0.90Pnx (kip)	= 3616.55	Pu/0.90Pnx	= 0.060
		0.90Pny (kip)	= 2956.81	Pu/0.90Pny	= 0.073
		0.90Pn (kip)	= 2956.81	Pu/0.90Pn	= 0.073

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	170.23
Moment	Top	Mux (kip-ft)	-86.32
		Muy (kip-ft)	0.16
Moment	Bot.	Mux (kip-ft)	187.36
		Muy (kip-ft)	0.14

CALCULATED PARAMETERS:

Pu (kip)	= 170.23	0.90Pnx (kip)	= 3616.55
		0.90Pny (kip)	= 2956.81
Mux (kip-ft)	= 187.36	0.90Mnx (kip-ft)	= 3131.25
Muy(kip-ft)	= 0.14	0.90Mny (kip-ft)	= 723.75
		Mcx (kip-ft)	= 3033.66
KL/Rx	= 16.07	KL/Ry	= 54.89

Cbx	=	2.17

INTERACTION EQUATION:

Pu/0.90*Pn=0.058

Eq H1-1b Per H1.3: 0.024 + 0.060 + 0.000 = 0.083

Eq H1-3: 0.085 + 0.001 = 0.086

Member Code Check

COLUMN INFORMATION:

Story Level = 3rd   Frame Number = 4   Column Number= 2

Fy (ksi) = 50.00

Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	55.90
		Vuy (kip)	-0.01
Shear	Bot.	Vux (kip)	55.90
		Vuy (kip)	-0.01

SHEAR CHECK:

Vux (kip)	= 55.90	1.00Vnx (kip)	=	929.16	Vux/1.00Vnx	=	0.060
Vuy (kip)	= -0.01	0.90Vny (kip)	=	1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	= 229.03	0.90Pnx (kip)	=	3616.55	Pu/0.90Pnx	=	0.063
		0.90Pny (kip)	=	2956.81	Pu/0.90Pny	=	0.077
		0.90Pn (kip)	=	2956.81	Pu/0.90Pn	=	0.077

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	229.03
Moment	Top	Mux (kip-ft)	-277.95
		Muy (kip-ft)	-0.15
Moment	Bot.	Mux (kip-ft)	415.14
		Muy (kip-ft)	-0.13

CALCULATED PARAMETERS:

Pu (kip)	= 229.03	0.90Pnx (kip)	=	3616.55
		0.90Pny (kip)	=	2956.81
Mux (kip-ft)	= 415.14	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	= -0.13	0.90Mny (kip-ft)	=	723.75
		Mcx (kip-ft)	=	3033.66
KL/Rx	= 16.07	KL/Ry	=	54.89

Cbx = 2.21

INTERACTION EQUATION:

Pu/0.90*Pn=0.063

Eq H1-3: 0.113 + 0.004 = 0.117

Eq H1-1b Per H1.3: 0.032 + 0.133 + 0.000 = 0.164

Member Code Check

COLUMN INFORMATION:

Story Level = 3rd   Frame Number = 4   Column Number= 3
Fy (ksi) = 50.00
Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	55.50
		Vuy (kip)	-0.02
Shear	Bot.	Vux (kip)	55.50
		Vuy (kip)	-0.02

SHEAR CHECK:

Vux (kip)	= 55.50	1.00Vnx (kip)	=	929.16	Vux/1.00Vnx	=	0.060
Vuy (kip)	= -0.02	0.90Vny (kip)	=	1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	= 224.49	0.90Pnx (kip)	=	3616.55	Pu/0.90Pnx	=	0.062
		0.90Pny (kip)	=	2956.81	Pu/0.90Pny	=	0.076
		0.90Pn (kip)	=	2956.81	Pu/0.90Pn	=	0.076

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	222.52
Moment	Top	Mux (kip-ft)	-276.11
		Muy (kip-ft)	-0.40
Moment	Bot.	Mux (kip-ft)	412.20
		Muy (kip-ft)	-0.34

CALCULATED PARAMETERS:

Pu (kip)	=	222.52	0.90Pnx (kip)	=	3616.55
			0.90Pny (kip)	=	2956.81
Mux (kip-ft)	=	412.20	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	=	-0.34	0.90Mny (kip-ft)	=	723.75
			Mcx (kip-ft)	=	3033.66
KL/Rx	=	16.07	KL/Ry	=	54.89

Cbx = 2.21

INTERACTION EQUATION:

Pu/0.90*Pn=0.062

Eq H1-3: 0.110 + 0.004 = 0.114

Eq H1-1b Per H1.3: 0.031 + 0.132 + 0.000 = 0.162

COLUMN INFORMATION:

Story Level = 3rd   Frame Number = 4   Column Number= 4

Fy (ksi) = 50.00

Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	14.50	14.50
Lu for Bending (ft)	14.50	14.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	31.27
		Vuy (kip)	-0.03
Shear	Bot.	Vux (kip)	31.27
		Vuy (kip)	-0.03

SHEAR CHECK:

Vux (kip)	31.27	1.00Vnx (kip)	929.16	Vux/1.00Vnx	0.034
Vuy (kip)	-0.03	0.90Vny (kip)	1501.04	Vuy/0.90Vny	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	256.57	0.90Pnx (kip)	3616.55	Pu/0.90Pnx	0.071
		0.90Pny (kip)	2956.81	Pu/0.90Pny	0.087
		0.90Pn (kip)	2956.81	Pu/0.90Pn	0.087

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	256.57
Moment	Top	Mux (kip-ft)	-156.24
		Muy (kip-ft)	-0.67
Moment	Bot.	Mux (kip-ft)	259.34
		Muy (kip-ft)	-0.56

CALCULATED PARAMETERS:

Pu (kip)	256.57	0.90Pn (kip)	2956.81
Mux (kip-ft)	259.34	0.90Mnx (kip-ft)	3131.25
Muy(kip-ft)	-0.56	0.90Mny (kip-ft)	723.75
KL/Rx	16.07	KL/Ry	54.89
Cbx	2.19		

INTERACTION EQUATION:

Pu/0.90*Pn=0.087
Eq H1-1b: 0.043 + 0.083 + 0.001 = 0.127

Member Code Check

BEAM INFORMATION:

Story Level = 3rd   Frame Number = 4   Beam Number= 1

Fy (ksi) = 50.00

Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-51.84	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.131
Vuy (kip)	=	-0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00

j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-433.41	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.43			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.416

Member Code Check

BEAM INFORMATION:

Story Level = 3rd   Frame Number = 4   Beam Number= 2

Fy (ksi) = 50.00

Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-50.46	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.128
Vuy (kip)	=	0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00

j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-423.29	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.43			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.406

Member Code Check

BEAM INFORMATION:

Story Level = 3rd   Frame Number = 4   Beam Number= 3
Fy (ksi) = 50.00
Beam Size = W27X94

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-51.37	1.00Vnx (kip)	=	395.43	Vux/1.00Vnx	=	0.130
Vuy (kip)	=	0.00	0.90Vny (kip)	=	402.30	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00
j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1242.00	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1242.00	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1242.00	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1242.00
Mux (kip-ft)	=	-441.42	0.90Mnx (kip-ft)	=	1042.50
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	145.50
Cbx	=	1.42			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.423

Member Code Check

COLUMN INFORMATION:

Story Level = 2nd   Frame Number = 4   Column Number= 1
Fy (ksi) = 50.00
Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	16.50	16.50
Lu for Bending (ft)	16.50	16.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	36.53
		Vuy (kip)	-0.04
Shear	Bot.	Vux (kip)	36.53
		Vuy (kip)	-0.04

SHEAR CHECK:

Vux (kip)	=	36.53	1.00Vnx (kip)	=	929.16	Vux/1.00Vnx	=	0.039
Vuy (kip)	=	-0.04	0.90Vny (kip)	=	1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	=	289.53	0.90Pnx (kip)	=	3596.47	Pu/0.90Pnx	=	0.081
			0.90Pny (kip)	=	2770.84	Pu/0.90Pny	=	0.104
			0.90Pn (kip)	=	2770.84	Pu/0.90Pn	=	0.104

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	220.92
Moment	Top	Mux (kip-ft)	-29.54
		Muy (kip-ft)	0.14
Moment	Bot.	Mux (kip-ft)	547.87
		Muy (kip-ft)	-0.66

CALCULATED PARAMETERS:

Pu (kip)	=	220.92	0.90Pnx (kip)	=	3596.47
			0.90Pny (kip)	=	2770.84
Mux (kip-ft)	=	547.87	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	=	-0.66	0.90Mny (kip-ft)	=	723.75
			Mcx (kip-ft)	=	2974.56
KL/Rx	=	18.29	KL/Ry	=	62.46

Cbx	=	1.73

INTERACTION EQUATION:

Pu/0.90*Pn=0.061

Eq H1-3: 0.116 + 0.011 = 0.128

Eq H1-1b Per H1.3: 0.031 + 0.175 + 0.000 = 0.206

Member Code Check

COLUMN INFORMATION:

Story Level = 2nd Frame Number = 4 Column Number= 2

Fy (ksi) = 50.00

Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	16.50	16.50
Lu for Bending (ft)	16.50	16.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	52.97
		Vuy (kip)	0.03
Shear	Bot.	Vux (kip)	52.97
		Vuy (kip)	0.03

SHEAR CHECK:

Vux (kip)	=	52.97	1.00Vnx (kip)	=	929.16	Vux/1.00Vnx	=	0.057
Vuy (kip)	=	0.03	0.90Vny (kip)	=	1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	308.17	0.90Pnx (kip)	=	3596.47	Pu/0.90Pnx	=	0.086
			0.90Pny (kip)	=	2770.84	Pu/0.90Pny	=	0.111
			0.90Pn (kip)	=	2770.84	Pu/0.90Pn	=	0.111

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	308.17
Moment	Top	Mux (kip-ft)	-185.58
		Muy (kip-ft)	-0.13
Moment	Bot.	Mux (kip-ft)	628.12
		Muy (kip-ft)	0.58

CALCULATED PARAMETERS:

Pu (kip)	=	308.17	0.90Pnx (kip)	=	3596.47
			0.90Pny (kip)	=	2770.84
Mux (kip-ft)	=	628.12	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	=	0.58	0.90Mny (kip-ft)	=	723.75
			Mcx (kip-ft)	=	2974.56
KL/Rx	=	18.29	KL/Ry	=	62.46

Cbx	=	2.08

INTERACTION EQUATION:

Pu/0.90*Pn=0.086

Eq H1-3: 0.161 + 0.010 = 0.171

Eq H1-1b Per H1.3: 0.043 + 0.201 + 0.000 = 0.243

COLUMN INFORMATION:

Story Level	= 2nd	Frame Number	= 4	Column Number=	3
Fy (ksi)	= 50.00				
Column Size	= W24X279				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	16.50	16.50
Lu for Bending (ft)	16.50	16.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	52.74
		Vuy (kip)	0.11
Shear	Bot.	Vux (kip)	52.74
		Vuy (kip)	0.11

SHEAR CHECK:

Vux (kip)	=	52.74	1.00Vnx (kip)	= 929.16	Vux/1.00Vnx	=	0.057
Vuy (kip)	=	0.11	0.90Vny (kip)	= 1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn8

AXIAL CHECK:

Pu (kip)	=	300.78	0.90Pnx (kip)	= 3596.47	Pu/0.90Pnx	=	0.084
			0.90Pny (kip)	= 2770.84	Pu/0.90Pny	=	0.109
			0.90Pn (kip)	= 2770.84	Pu/0.90Pn	=	0.109

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	297.04
Moment	Top	Mux (kip-ft)	-183.17
		Muy (kip-ft)	-0.34
Moment	Bot.	Mux (kip-ft)	627.00
		Muy (kip-ft)	1.74

CALCULATED PARAMETERS:

Pu (kip)	=	297.04	0.90Pnx (kip)	=	3596.47
			0.90Pny (kip)	=	2770.84
Mux (kip-ft)	=	627.00	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	=	1.74	0.90Mny (kip-ft)	=	723.75
			Mcx (kip-ft)	=	2974.56
KL/Rx	=	18.29	KL/Ry	=	62.46

Member Code Check

Cbx	=	2.07

INTERACTION EQUATION:

Pu/0.90*Pn=0.083

Eq H1-3: 0.155 + 0.010 = 0.165

Eq H1-1b Per H1.3: 0.041 + 0.200 + 0.000 = 0.242

COLUMN INFORMATION:

Story Level = 2nd Frame Number = 4 Column Number= 4

Fy (ksi) = 50.00

Column Size = W24X279

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	16.50	16.50
Lu for Bending (ft)	16.50	16.50
K	1.00	1.00

CONTROLLING COLUMN FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Shear	Top	Vux (kip)	41.45
		Vuy (kip)	0.18
Shear	Bot.	Vux (kip)	41.45
		Vuy (kip)	0.18

SHEAR CHECK:

Vux (kip)	=	41.45	1.00Vnx (kip)	=	929.16	Vux/1.00Vnx	=	0.045
Vuy (kip)	=	0.18	0.90Vny (kip)	=	1501.04	Vuy/0.90Vny	=	0.000

CONTROLLING COLUMN FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

AXIAL CHECK:

Pu (kip)	=	349.05	0.90Pnx (kip)	=	3596.47	Pu/0.90Pnx	=	0.097
			0.90Pny (kip)	=	2770.84	Pu/0.90Pny	=	0.126
			0.90Pn (kip)	=	2770.84	Pu/0.90Pn	=	0.126

CONTROLLING COLUMN FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Axial		Load (kip)	349.05
Moment	Top	Mux (kip-ft)	-80.48
		Muy (kip-ft)	-0.56
Moment	Bot.	Mux (kip-ft)	571.86
		Muy (kip-ft)	2.92

CALCULATED PARAMETERS:

Pu (kip)	=	349.05	0.90Pnx (kip)	=	3596.47
			0.90Pny (kip)	=	2770.84
Mux (kip-ft)	=	571.86	0.90Mnx (kip-ft)	=	3131.25
Muy(kip-ft)	=	2.92	0.90Mny (kip-ft)	=	723.75
			Mcx (kip-ft)	=	2974.56
KL/Rx	=	18.29	KL/Ry	=	62.46

Cbx	=	1.84

INTERACTION EQUATION:

Pu/0.90*Pn=0.097

Eq H1-3: 0.181 + 0.011 = 0.192

Eq H1-1b Per H1.3: 0.049 + 0.183 + 0.000 = 0.231

Member Code Check

BEAM INFORMATION:

Story Level	= 2nd	Frame Number	= 4	Beam Number=	1
Fy (ksi)	= 50.00				
Beam Size	= W30X124				

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end	22.50
j - end	30.00

SHEAR CHECK:

Vux (kip) =	-57.69	1.00Vnx (kip) =	530.01	Vux/1.00Vnx =	0.109
Vuy (kip) =	0.00	0.90Vny (kip) =	527.31	Vuy/0.90Vny =	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end	0.00
j - end	7.50

AXIAL CHECK:

Pu (kip) =	0.00	0.90Pnx (kip) =	1642.50	Pu/0.90Pnx =	0.000
		0.90Pny (kip) =	1642.50	Pu/0.90Pny =	0.000
		0.90Pn (kip) =	1642.50	Pu/0.90Pn =	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end	22.50
j - end	30.00

CALCULATED PARAMETERS:

Pu (kip) =	0.00	0.90Pn (kip) =	1642.50
Mux (kip-ft) =	-499.52	0.90Mnx (kip-ft) =	1530.00
Muy(kip-ft) =	0.00	0.90Mny (kip-ft) =	202.50
Cbx =	1.41		

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.326

BEAM INFORMATION:

Story Level = 2nd   Frame Number = 4   Beam Number= 2
Fy (ksi) = 50.00
Beam Size = W30X124

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-54.58	1.00Vnx (kip)	=	530.01	Vux/1.00Vnx	=	0.103
Vuy (kip)	=	0.00	0.90Vny (kip)	=	527.31	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00
j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1642.50	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1642.50	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1642.50	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50
j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1642.50
Mux (kip-ft)	=	-475.19	0.90Mnx (kip-ft)	=	1530.00
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	202.50
Cbx	=	1.41			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000
Mrx/Mcx = 0.311

Member Code Check

BEAM INFORMATION:

Story Level = 2nd   Frame Number = 4   Beam Number= 3

Fy (ksi) = 50.00

Beam Size = W30X124

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu for Axial (ft)	30.00	7.50
Lu for Bending (ft)	30.00	7.50
K	1.00	1.00
Top Flange Continuously Braced	No	
Bottom Flange Continuously Braced	No	

CONTROLLING BEAM SEGMENT FORCES - SHEAR

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

SHEAR CHECK:

Vux (kip)	=	-56.78	1.00Vnx (kip)	=	530.01	Vux/1.00Vnx	=	0.107
Vuy (kip)	=	-0.00	0.90Vny (kip)	=	527.31	Vuy/0.90Vny	=	0.000

CONTROLLING BEAM SEGMENT FORCES - AXIAL

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn5

Segment distance (ft) i - end 0.00

j - end 7.50

AXIAL CHECK:

Pu (kip)	=	0.00	0.90Pnx (kip)	=	1642.50	Pu/0.90Pnx	=	0.000
			0.90Pny (kip)	=	1642.50	Pu/0.90Pny	=	0.000
			0.90Pn (kip)	=	1642.50	Pu/0.90Pn	=	0.000

CONTROLLING BEAM SEGMENT FORCES - FLEXURE

Load Combination: 1.348 D + 0.500 Lp + 1.000 Dyn6

Segment distance (ft) i - end 22.50

j - end 30.00

CALCULATED PARAMETERS:

Pu (kip)	=	0.00	0.90Pn (kip)	=	1642.50
Mux (kip-ft)	=	-517.92	0.90Mnx (kip-ft)	=	1530.00
Muy(kip-ft)	=	0.00	0.90Mny (kip-ft)	=	202.50
Cbx	=	1.38			

INTERACTION EQUATION:

Pu/0.90*Pn=0.000

Mrx/Mcx = 0.339

Periods and Modes

CRITERIA:

Rigid End Zones: Ignore Effects
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.
Rigid Links Included at Fixed Beam-to-Wall Locations
Eigenvalue Analysis : Eigen Vectors (Subspace Iteration)

Load Case: Eigen Solution

FREQUENCIES AND PERIODS:

Mode	Period (T)	Cyclic Frequency (f)	Circular Frequency (ω)	Eigenvalue
	sec	Hz	rad/sec	(rad/sec)**2
1	1.4898	0.6712	4.2175	17.7871
2	1.4898	0.6713	4.2176	17.7882
3	0.8864	1.1281	7.0881	50.2417
4	0.4472	2.2364	14.0515	197.4434
5	0.4471	2.2366	14.0528	197.4824
6	0.2704	3.6985	23.2382	540.0160
7	0.2184	4.5783	28.7661	827.4903
8	0.2184	4.5796	28.7744	827.9673
9	0.1354	7.3877	46.4185	2154.6763
10	0.1353	7.3923	46.4472	2157.3449
11	0.1342	7.4537	46.8328	2193.3128
12	0.0840	11.9096	74.8301	5599.5406

MODAL PARTICIPATION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	-0.0006	48.4226	-1.2715
2	48.4224	0.0006	0.0000
3	-0.0000	0.1540	262.5199
4	0.0000	-20.3282	0.4619
5	-20.3293	-0.0000	0.0000
6	0.0000	-0.1314	-109.6174
7	0.0000	11.6529	-0.1137
8	11.6554	-0.0000	0.0001
9	-0.0001	-6.0292	6.4778

10	-6.0832	0.0001	0.0009
11	0.0001	0.8172	62.1352
12	-0.0000	-0.1765	-32.4291

MODAL DIRECTION FACTORS:

Mode	X-Dir	Y-Dir	Rotation
1	0.00	100.00	0.00
2	100.00	0.00	0.00
3	0.00	0.00	100.00
4	0.00	99.99	0.01
5	100.00	0.00	0.00
6	0.00	0.02	99.98
7	0.00	99.95	0.05
8	100.00	0.00	0.00
9	0.00	99.06	0.94
10	100.00	0.00	0.00
11	0.00	0.90	99.10
12	0.00	0.07	99.93

MODAL EFFECTIVE MASS FACTORS:

Mode	X-Dir		Y-Dir		Rotation	
	%Mass	%SumM	%Mass	%SumM	%Mass	%SumM
1	0.00	0.00	80.00	80.00	0.00	0.00
2	80.00	80.00	0.00	80.00	0.00	0.00
3	0.00	80.00	0.00	80.00	80.24	80.24
4	0.00	80.00	14.10	94.10	0.00	80.24
5	14.10	94.10	0.00	94.10	0.00	80.24
6	0.00	94.10	0.00	94.10	13.99	94.23
7	0.00	94.10	4.63	98.74	0.00	94.23
8	4.64	98.74	0.00	98.74	0.00	94.23
9	0.00	98.74	1.24	99.98	0.05	94.28
10	1.26	100.00	0.00	99.98	0.00	94.28
11	0.00	100.00	0.02	100.00	4.50	98.78
12	0.00	100.00	0.00	100.00	1.22	100.00

MODE SHAPES:

Mode Shapes for Rigid Diaphragms:

Story	Diaph. #	Dir	Mode 1	Mode 2	Mode 3
Roof	1	X	-3.793110E-06	3.245464E-01	-6.296235E-08
		Y	3.245365E-01	3.793338E-06	2.179425E-03
		R	-1.612641E-06	0.000000E+00	4.162139E-04
4th	1	X	-2.918476E-06	2.496838E-01	1.985941E-08

Periods and Modes

		Y	2.496841E-01	2.918349E-06	7.827314E-04
		R	-1.563601E-06	0.000000E+00	3.217952E-04
3rd	1	X	-1.801971E-06	1.541603E-01	1.491910E-08
		Y	1.541644E-01	1.801864E-06	-2.970415E-05
		R	-1.132440E-06	0.000000E+00	1.996440E-04
2nd	1	X	-7.243589E-07	6.199983E-02	-4.551743E-08
		Y	6.200836E-02	7.247259E-07	-4.453554E-04
		R	-6.278454E-07	0.000000E+00	8.126126E-05

Story	Diaph. #	Dir	Mode 4	Mode 5	Mode 6
Roof	1	X	-1.994555E-07	2.504049E-01	5.475813E-08
		Y	2.503838E-01	1.993281E-07	3.728398E-03
		R	-4.336351E-06	0.000000E+00	3.249832E-04
4th	1	X	8.457798E-08	-1.061312E-01	-9.604251E-08
		Y	-1.060931E-01	-8.436173E-08	-4.006186E-03
		R	-1.963568E-08	0.000000E+00	-1.299877E-04
3rd	1	X	2.315596E-07	-2.900125E-01	-5.826123E-08
		Y	-2.899912E-01	-2.308341E-07	-2.957651E-03
		R	3.240570E-06	0.000000E+00	-3.698569E-04
2nd	1	X	1.540542E-07	-1.953319E-01	2.427400E-07
		Y	-1.953523E-01	-1.558810E-07	9.654033E-04
		R	2.927287E-06	0.000000E+00	-2.529378E-04

Story	Diaph. #	Dir	Mode 7	Mode 8	Mode 9
Roof	1	X	1.646663E-08	1.475799E-01	1.237890E-06
		Y	1.475475E-01	-1.819776E-08	5.667782E-02
		R	-6.744278E-06	-1.207914E-10	1.618845E-05
4th	1	X	-3.191049E-08	-2.943257E-01	-4.244532E-06
		Y	-2.942430E-01	3.564668E-08	-1.940398E-01
		R	8.368022E-06	1.221335E-10	-2.881159E-05
3rd	1	X	2.231554E-09	3.867484E-02	6.497985E-06
		Y	3.863333E-02	-4.619437E-09	2.964772E-01
		R	3.007963E-06	2.441338E-10	-6.104909E-06
2nd	1	X	3.415330E-08	2.952279E-01	-5.684805E-06
		Y	2.951778E-01	-3.342806E-08	-2.588856E-01
		R	-4.978381E-06	1.105437E-10	4.348255E-05

Story	Diaph. #	Dir	Mode 10	Mode 11	Mode 12
Roof	1	X	5.669252E-02	-9.962109E-07	-6.094981E-09
		Y	-1.284222E-06	-2.768965E-03	1.380097E-03
		R	3.133247E-09	1.932253E-04	7.459499E-05

4th	1	X	-1.944109E-01	3.415007E-06	1.596782E-08
		Y	4.454233E-06	1.264316E-02	-4.926317E-03
		R	-6.044699E-09	-3.752855E-04	-2.501099E-04
3rd	1	X	2.977595E-01	-5.243375E-06	-6.941008E-09
		Y	-6.938205E-06	-2.662706E-02	7.651024E-03
		R	8.141847E-10	4.519041E-05	3.820910E-04
2nd	1	X	-2.606952E-01	4.616692E-06	-1.462549E-08
		Y	6.183266E-06	3.017172E-02	-7.020794E-03
		R	5.570897E-09	3.743484E-04	-3.308657E-04

Note 1: Mode shapes are normalized with respect to building mass matrix: Qi^T x M x Qi = 1
where Qi: ith mode shape and M is the building mass matrix.
Normalized mode shapes are consistent with units of "kip" and "in".

Note 2: Mode shapes for meshed diaphragms are not included by default.
Turn on "Include Results for All Nodes" to include mode shapes for meshed diaphragms.
This may substantionally increase the length of the report for large models.