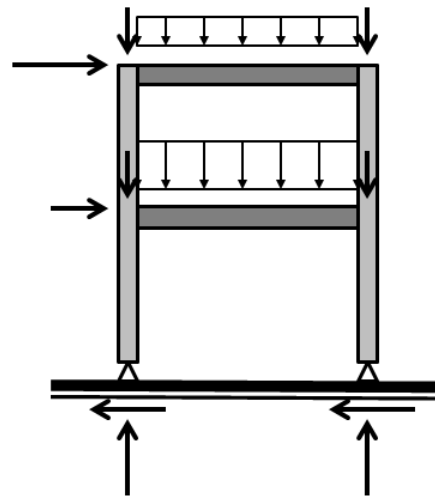


2.17 A 2-story single bay steel moment-resisting frame has been analyzed using the tributary gravity loads and tributary lateral loads. The structure is assigned to *Seismic Design Category D* with $S_{DS} = 0.71$ and $S_{D1} = 0.37$. The horizontal shear at the base of each column has been determined with the following individual load effects:

- $D = 15$ kips
- $L = 9$ kips based on $L_0 = 50$ psf Office live load
- $L_r = 4$ kips based on 16 psf roof live load
- $Q_e = +/- 20$ kips
- $W = +/- 8$ kips
- $S = 0$ kips
- $F = 0$ kips
- $H = 0$ kips
- $R = 0$ kips



Frame Elevation

Determine the maximum shear at the base of one column using the Strength Design (SD/LRFD) load combinations including seismic load effects.

- a. 48.5 kips
 - b. 50.6 kips
 - c. 53.0 kips
 - d. 55.1 kips
- 2.18 Vertical seismic force-resisting systems assumed to provide very limited ductility under seismic loading are typically only permitted for structures:
- a. provided with structural redundancy
 - b. of *Risk Category I* or *II*
 - c. assigned to a low hazard *Seismic Design Category*
 - d. without horizontal and vertical irregularities
- 2.19 Which of the following typically requires consideration of seismic load effects including the overstrength factor?
- a. Cantilever columns systems
 - b. Elements supporting discontinuous walls of frames
 - c. Collector elements for *Seismic Design Category C, D, E* or *F*
 - d. All the above
- 2.20 For structures assigned to *Seismic Design Category D*, which structural irregularity would NOT require design forces be increased 25% for connections of diaphragms to shear walls and to collectors, and collectors and their connections to shear walls, per *ASCE 7-16 – §12.3.3.4*?
- a. Torsional irregularity
 - b. Vertical geometric irregularity
 - c. Diaphragm discontinuity irregularity
 - d. Reentrant corner irregularity

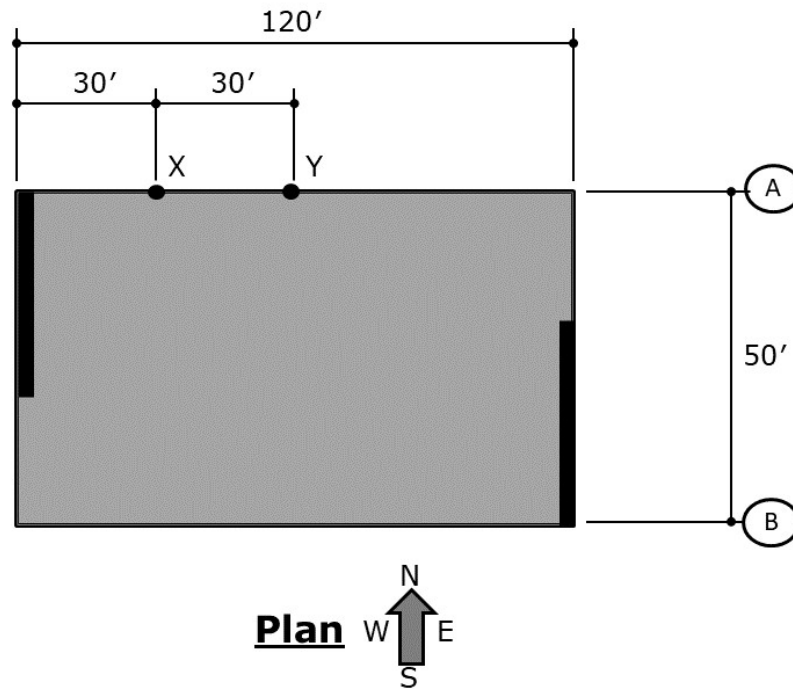
Practice Exam #2 – Answer Key

NOTE: Use this page to grade your answers from the Practice Exam #2 – Answer Sheet. Correct answers are shown in bold text below. Use the Content Area fields below by adding up the number of correct questions in that content area divided by the total number of questions in that content area. Your performance in each content area is estimated as follows: PROFICIENT $\geq 75\%$ in that content area, MARGINAL $< 75\%$ but $\geq 60\%$ in that content area, and DEFICIENT $< 60\%$ in that content area. Refer to the California CSP Test Plan on p. 4-1 to 4-5 for the content areas and activities.

Question No.	Answer	Content Area	Question No.	Answer	Content Area	Question No.	Answer	Content Area
2.1	b	I	2.21	b	I	2.41	c	V
2.2	d	I	2.22	b	II	2.42	c	VI
2.3	d	VI	2.23	b	VI	2.43	b	IV
2.4	c	V	2.24	c	V	2.44	c	I
2.5	b	II	2.25	a	VI	2.45	d	II
2.6	a	IV	2.26	d	V	2.46	a	IV
2.7	d	I	2.27	b	VI	2.47	c	V
2.8	a	IV	2.28	c	I	2.48	d	VII
2.9	d	II	2.29	b	VI	2.49	c	I
2.10	d	V	2.30	b	II	2.50	c	IV
2.11	a	IV	2.31	c	VII	2.51	c	I
2.12	d	VII	2.32	c	VI	2.52	b	VII
2.13	a	VI	2.33	a	III	2.53	a	I
2.14	b	IV	2.34	c	IV	2.54	c	V
2.15	a	V	2.35	d	IV	2.55	c	I
2.16	a	II	2.36	d	III	I = 11 II = 7 III = 2 IV = 10 V = 9 VI = 12 VII = 4		
2.17	b	VI	2.37	b	IV			
2.18	c	I	2.38	d	VI			
2.19	d	VI	2.39	b	VI			
2.20	b	II	2.40	c	VI			

Problem	Answer	Reference / Solution
2.17	b	<p>p. 1-83 - SD/LRFD Load Combinations with Seismic Load Effects & <i>ASCE 7-16</i> p. 2 – §2.3.6</p> <p>By observation – <i>ASCE 7 (2.3-6)</i> will govern for the <u>maximum</u> shear in the column when considering seismic load effects (i.e., <i>ASCE 7 (2.3-7)</i> will clearly provide a lower shear).</p> <p>$D = 15$ kips (given)</p> <p>$L = 9$ kips (given) ... due to Office <u>floor live load</u> of $L_0 = 50$ psf</p> <p>$\rho = 1.3$ for <i>SDC</i> = D, E or F (if not otherwise given)</p> <p>$E_h = \rho Q_E$ <i>ASCE 7 (12.4-3)</i></p> <p>$= 1.3 (\pm 20 \text{ kips}) = \pm 26$ kips</p> <p>$E_v = \pm 0.2 S_{DS} D$ <i>ASCE 7 (12.4-4a)</i></p> <p>$= \pm 0.2 (0.71)(15 \text{ kips}) = \pm 2.1$ kips</p> <p>$1.2 D + E_v + E_h + *0.5 L + 0.2 S$ <i>ASCE 7 (2.3-6)</i></p> <p>max. shear, $V = 1.2 (15) + 2.1 + 26 + 0.5 (9) + 0.2 (0) = 50.6$ kips ←</p> <p>*NOTE: <i>ASCE 7-16 – §2.3.6, Exception 1</i> ... the load factor on L is permitted to equal 0.5 ... in which $L_0 \leq 100$ psf, with the exception of garages, etc.</p>
2.18	c	<p>p. 1-46 & <i>ASCE 7-16</i> p. 90 to 92 – <i>Table 12.2-1</i></p> <p>1. Ordinary - basic (gravity & wind) detailing for low seismic hazard levels (e.g., <i>SDC</i> = B or <i>SDC</i> = B & C)</p> <p>Per <i>ASCE 7-16 – Table 12.2-1 – ordinary</i> shear walls, <i>ordinary</i> braced frames, <i>ordinary</i> moment frames, etc. are <u>typically</u> not permitted (NP) in structures assigned to <i>SDC</i> = D, E or F</p> <p>\therefore <u>assigned to a low hazard Seismic Design Category</u> ←</p>
2.19	d	<p>p. 1-85 to 86 - Load Combinations with Overstrength & <i>ASCE 7-16</i> p. 94 – §12.2.5.2, p. 97 – §12.3.3.3, and p. 106 to 107 – §12.10.2.1</p> <p>Load combinations including overstrength (Ω_0) require consideration for:</p> <ul style="list-style-type: none"> ▪ Cantilever column systems ▪ Elements supporting discontinuous walls or frames ▪ Collector elements for <i>SDC</i> = C, D, E or F <p>\therefore <u>All the above</u> ←</p>
2.20	b	<p><i>ASCE 7-16</i> p. 95 and 97 – <i>Table 12.3-1</i> and <i>Table 12.3-2</i></p> <p>Torsional irregularity → Type 1a per <i>Table 12.3-1</i> → references §12.3.3.4 for <i>SDC</i> = D, E & F</p> <p>Vertical geometric irregularity → Type 3 per <i>Table 12.3-2</i> → <u>does NOT</u> reference §12.3.3.4 for <u>any</u> <i>SDC</i></p> <p>Diaphragm discontinuity irregularity → Type 3 per <i>Table 12.3-1</i> → references §12.3.3.4 for <i>SDC</i> = D, E & F</p> <p>Reentrant corner irregularity → Type 2 per <i>Table 12.3-1</i> → references §12.3.3.4 for <i>SDC</i> = D, E & F</p> <p>\therefore <u>Vertical geometric irregularity</u> ←</p>

- 3.48 For the flexible diaphragm structure below, determine the ratio of the chord force at "X" to the maximum chord at "Y" for N-S diaphragm loading.



- a. $CF_X = 1/4 CF_Y$
 b. $CF_X = 1/2 CF_Y$
 c. $CF_X = 2/3 CF_Y$
 d. $CF_X = 3/4 CF_Y$
- 3.49 Given a 2-story Office building assigned to *Seismic Design Category D*. The following are axial load effects determined in the first story column of a 2-story steel special moment frame. Determine the maximum axial load effect using SD/LRFD load combinations that include the earthquake load effect (E).
- $D = 125$ kips
 - $L = 95$ kips
 - $L_r = 30$ kips
 - $S = 72$ kips
 - $Q_E = 45$ kips
 - $S_{DS} = 0.53$
- a. 257 kips
 b. 284 kips
 c. 318 kips
 d. 331 kips
- 3.50 What component amplification factor and component response modification factor applies to the design of an architectural parapet that extends 5-feet above the roof level, but is braced 2-feet down from the top of the parapet and down to the roof framing?
- a. $a_p = 2\frac{1}{2}$ & $R_p = 2\frac{1}{2}$
 b. $a_p = 1$ & $R_p = 2\frac{1}{2}$
 c. $a_p = 2\frac{1}{2}$ & $R_p = 3\frac{1}{2}$
 d. $a_p = 1$ & $R_p = 3\frac{1}{2}$

Problem	Answer	Reference / Solution
3.49	d	<p>p. 1-83 - Basic (SD or LRFD) Load Combinations & <i>ASCE 7-16</i> p. 8 – §2.3.6 $\rho = 1.3$ for <i>SDC</i> = D, E or F (if not otherwise given) $E_h = \rho Q_E$ <i>ASCE 7 (12.4-3)</i> $= 1.3 (45 \text{ kips}) = 58.5 \text{ kips}$ $E_v = 0.2 S_{DS} D$ <i>ASCE 7 (12.4-4a)</i> $= 0.2 (0.53)(125 \text{ kips}) = 13.3 \text{ kips}$ By observation, <i>ASCE 7 (2.3-6)</i> will govern for the <u>maximum</u> compression in the column over <i>ASCE 7 (2.3-7)</i>: $1.2D + E_v + E_h + L + 0.2S$ <i>ASCE 7 (2.3-6)</i> max. $P = 1.2 (125 \text{ kips}) + 13.3 \text{ kips} + 58.5 \text{ kips} + 95 \text{ kips} + 0.2 (72 \text{ kips})$ $= 331 \text{ kips}$ $\therefore 331 \text{ kips} \leftarrow$</p>
3.50	b	<p>p. 1-94 - Architectural Components & <i>ASCE 7-16</i> p. 126 – <i>Table 13.5-1</i> A 5-foot tall parapet that is braced 2 feet down from the top of the parapet is braced 3 feet above the roof line which is above the center of mass of the parapet (i.e., $3' > 5'/2 = 2.5'$) ... therefore use <i>ASCE 7-16 – Table 13.5-1</i> with “Cantilever elements (braced to structural frame above its center of mass) - Parapets” $\therefore a_p = 1$ & $R_p = 2\frac{1}{2} \leftarrow$</p>
3.51	c	<p>p. 1-122 - Drag Force For flexible diaphragm, $V_1 = V_2 = F_{p1} / 2 = 30 \text{ kips}$ Unit roof shear $v_1 = v_2 = V_1 / d = (15 \text{ kips}) / (90') = 0.333 \text{ klf}$ <u>Line 1:</u> Drag force, max. $F_d = (\text{roof } v_1)(20') - 30 \text{ kips} / 2$ $= (0.333 \text{ klf})(20') - 15 \text{ kips} = -8.3 \text{ kips} \leftarrow$ governs <u>Line 2:</u> Drag force, max. $F_d = (\text{roof } v_2)(20')$ $= (0.333 \text{ klf})(20') = 6.7 \text{ kips}$ $\therefore 8.3 \text{ kips} \leftarrow$</p>
3.52	b	<p>p. 1-96 - Design for Out-of-Plane Forces, F_p & <i>ASCE 7-16</i> p. 108 – §12.11.1 $S_{DS} = 0.92$ (given) WWTP (electrical building) \rightarrow <i>IBC Table 1604.5</i> p. 16-5 \rightarrow <i>RC</i> = III \rightarrow $ASCE 7-16$ p. 5 – <i>Table 1.5-2</i> \rightarrow $I_e = 1.25$ $W_{\text{wall}} = 85 \text{ psf}$ (given) $F_p = 0.4 S_{DS} I_e W_{\text{wall}}$ $= 0.4 (0.92) (1.25) 85 \text{ psf} = 39 \text{ psf} \leftarrow$ governs $\geq 0.10 W_{\text{wall}} = 0.10 (85 \text{ psf}) = 8.5 \text{ psf}$ minimum $\therefore 39 \text{ psf} \leftarrow$</p>