Load Combinations with Overstrength ~~Factor~~

Where the seismic load effect including overstrength ~~factor~~ (E_m) is combined with the effects of other loads … the following seismic load combinations of *ASCE 7-16 – §2.3.6* (SD or LRFD) or *ASCE 7-16 – §2.4.5* (ASD) shall be used.

Basic Combinations for SD (or LRFD) with Overstrength ~~Factor~~ *ASCE 7 – §2.3.6*

6. $1.2D + E_v + E_{mh} + L + 0.2S$

   or … $(1.2 + 0.2S_{DS})D + \Omega_0 Q_E + L + 0.2S$

7. $0.9D - E_v + E_{mh}$

   or … $(0.9 - 0.2S_{DS})D - \Omega_0 Q_E$

NOTE: See *ASCE 7-16 – §2.3.6* exceptions for additional requirements on the equations above.

Basic Combinations for ASD with Overstrength ~~Factor~~ *ASCE 7 – §2.4.5*

8. $1.0D + 0.7E_v + 0.7E_{mh}$

   or … $(1.0 + 0.14S_{DS})D + 0.7\Omega_0 Q_E$

9. $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S$

   or … $(1.0 + 0.105S_{DS})D + 0.525\Omega_0 Q_E + 0.75L + 0.75S$

10. $0.6D - 0.7E_v + 0.7E_{mh}$

    or … $(0.6 - 0.14S_{DS})D - 0.7\Omega_0 Q_E$

NOTE: See *ASCE 7-16 – §2.4.5* exceptions for additional requirements on the equations above.

Cantilever Column Systems *ASCE 7 – §12.2.5.2*

Foundations and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength (Ω_0) of *ASCE 7-16 – §12.4.3*.

Elements Supporting Discontinuous Walls or Frames *ASCE 7 – §12.3.3.3*

Structural elements (e.g., columns, beams, trusses, slabs) supporting discontinuous walls or frames shall be designed to resist the seismic load effects, including overstrength (Ω_0) of *ASCE 7-16 – §12.4.3 …*for structures having either of the following:

- **Horizontal Structural Irregularity Type 4 –** Out-of-Plane Offset per *ASCE 7-16 – Table 12.3-1*
- **Vertical Structural Irregularity Type 4 –** In-Plane Discontinuity in Vertical Lateral Force-Resisting Element per *ASCE 7-16 – Table 12.3-2*

Collector Elements for *SDC* = C, D, E or F *ASCE 7 – §12.10.2.1*

In structures assigned to *SDC* = C, D, E or F, collector elements and their connections, including connections to vertical elements, shall be designed to resist the maximum of the following:

1. Forces calculated using the seismic load effects including overstrength (Ω_0) of *ASCE 7-16 – §12.4.3* with seismic forces determined by the ELF procedure *ASCE 7-16 – §12.8* (or the modal response spectrum analysis procedure of *ASCE 7-16 – §12.9.1*)

- Wall B: h/b_s = (12′ / 4′) = 3.00 > 2:1 → use $2b_s/h$ = 2(4′ / 12′) = 0.67 reduction in unit shear capacity
  Capacity of Wall B = 520 plf ($2b_s/h$)(b_s) = 520 plf (0.67)(4′) = 1,390 lbs

V_A = [(3,640 lbs) / (3,640 lbs + 1,390 lbs)] V_1 = 72% V_1 ←

V_B = [(1,390 lbs) / (3,640 lbs + 1,390 lbs)] V_1 = 28% V_1 ←

Seismic Design Category D, E or F *SDPWS §4.3.7.1, item 5C*

Where the required nominal unit shear capacity on either side of the shear wall > 700 plf:

- ✓ the width of the framing members and blocking shall be 3″ nominal or greater (i.e., 3x = net 2.5″) at adjoining panel edges, and
- ✓ all panel edges and sill plate nailing shall be staggered
- ✓ see *SDPWS §4.3.6.4.3* for sill plate anchorage requirements (i.e., sill bolting)

Foundation Sill Bolts

Sill bolts are designed to transfer the in-plane unit wall shear from the foundation sill plate and into the concrete (or masonry) foundation below. Below is a summary of the minimum sill bolt requirements from the *Conventional Light-Frame Construction* provisions of *IBC §2308.3* & *§2308.6.7.3*:

- Minimum 1/2″ϕ sill bolts for *SDC* = A, B, C & D, minimum 5/8″ϕ sill bolts for *SDC* = E (& F) … or approved anchor straps load rated per *IBC §2304.10.3.*
- 6′-0″ o.c. maximum spacing (4′-0″ o.c. maximum spacing in structures > 2 stories)
- Minimum of two sill bolts (or anchor straps) per sill plate piece with one bolt (or anchor strap) 12″ maximum & 4″ minimum from each end of each sill plate piece
- 7″ minimum embedment into concrete (or masonry)
- Sill bolt nut with standard washers for *SDC* = A, B & C, sill bolt nut with 0.229″x3″x3″ plate washers for *SDC* = D, E (& F)
- Hole in plate washer is permitted to be diagonally slotted with a width of up to 3/16″ larger than the sill bolt diameter and a slot length not to exceed 1¾″, provided a standard cut washer is placed between the plate washer and the nut of the sill bolt (see Figure 9.7)

Anchor Bolts *SDPWS §4.3.6.4.3*

Foundation anchor bolts (i.e., sill bolts) shall have a steel plate washer under each nut not less than 0.229″x3″x3″ in size:

- hole in plate washer is permitted to be diagonally slotted with a width of up to 3/16″ larger than the sill bolt diameter and a slot length not to exceed 1¾″, provided a standard cut washer is placed between the plate washer and the nut of the sill bolt (see Figure 9.7)
- steel plate washers shall extend within 1/2″ of the edge of the bottom (i.e., sill) plate on the side(s) with sheathing (or other material) with nominal unit shear capacity of 400 plf for wind or seismic

Exception: Standard cut washers shall be permitted to be used where sill plate anchor bolts are designed to resist shear only and all the following requirements are met:

a. The shear wall is designed per *SDPWS §4.3.5.1* with required uplift anchorage at shear wall ends sized to resist overturning neglecting dead load resisting moment (i.e., *RM* = 0)
b. Shear wall aspect ratio $h/b \leq 2{:}1$
c. The nominal unit shear capacity of the shear wall is ≤ 980 plf for seismic (i.e., ≤ 490 plf for ASD) or ≤ 1370 plf for wind (i.e., ≤ 685 plf for ASD)

5.20 What is the axial force in brace X1 due to the seismic forces in the given direction?

a. 6 kips
b. 9 kips
c. 18 kips
d. 23 kips

5.21 What is the horizontal reaction (i.e., shear) at support A due to the seismic forces in the given direction?

a. 0 kips
b. 9 kips
c. 18 kips
d. 23 kips

5.22 What is the horizontal reaction (i.e., shear) at support B due to the seismic forces in the given direction?

a. 0 kips
b. 9 kips
c. 18 kips
d. 23 kips

5.23 What would be the vertical seismic load effect at support A & B if the vertical dead load reaction at those supports was 110 kips (i.e., D = 110 kips) and S_{DS} = 0.72?

a. ± 16 kips
b. ± 22 kips
c. ± 110 kips
d. ± 132 kips

5.24 Given a *redundancy factor* ρ = 1.3, what would be the horizontal seismic load effect in brace X1 due to the seismic forces in the given direction?

a. 8 kips
b. 12 kips
c. 22 kips
d. 29 kips

6.1 What *component amplification factor* (a_p) should be used to design the required steel reinforcement size and spacing for a masonry unbraced cantilever parapet?

a. 1
b. 1¼
c. 1½
d. 2½

6.2 What type of anchorage might require the use of the Ω_0 factor in *ASCE 7-16 – Table 13.5-1* or *Table 13.6-1*?

a. Non-ductile anchorage to concrete
b. Non-ductile anchorage to masonry
c. Non-ductile anchorage to concrete and masonry
d. None of the above

Problem	Answer	Reference / Solution
2.5	b	p. 1-66 to 67 - Story Drift Limit, Δ_{ax} & *ASCE 7-16* p. 109 - *§12.12.1* Medical Office building → *IBC Table 1604.5* → *RC* = II 5-stories > 4-stories → "All other Structures" → *Table 12.12-1* → $\Delta_{ax} \leq 0.020\, h_{sx} = 0.020$ (13 ft)(12 in/ft) = 3.12 inches ∴ 3.1 inches ←
2.6	a	p. 1-88 to 89 - Seismic Design Force & *ASCE 7-16* p. 123 - *§13.3.1* $S_{DS} = 0.92$ (given) A cantilever parapet is an Architectural component per *ASCE 7-16 – Table §13.5-1* $a_p = 2½$ & $R_p = 2½$ – *Table 13.5-1* - Cantilever elements (unbraced or braced to structural frame below its center of mass) - parapets $z = h$ → use $(z/h) = 1.0$ $I_p = 1.5$ per *ASCE 7-16 – §13.1.3* since the failure of the parapet could affect the continuous operation of this *RC* = IV Police station. $R_p / I_p = (2½ / 1.5) = 1.67$ $F_p = \dfrac{0.4 a_p S_{DS} W_p}{(R_P / I_P)}\left(1 + 2\dfrac{z}{h}\right)$ *ASCE 7 (13.3-1)* $= 0.4(2½)(0.92)\, W_p\, [1 + 2\,(1.0)] / (1.67) = 1.65\, W_p$ ← (governs) maximum $F_p \leq 1.6 S_{DS} I_p W_p$ *ASCE 7 (13.3-2)* $= 1.6(0.92)(1.5)\, W_p = 2.21\, W_p$ minimum $F_p \geq 0.3 S_{DS} I_p W_p$ *ASCE 7 (13.3-3)* $= 0.3(0.92)(1.5)\, W_p = 0.41\, W_p$ $f_p = 1.65$ (100 psf) = 165 psf - uniform load acting over the parapet height The bending moment at the roof level – $M = f_p \cdot h_p^2 / 2 = 165$ psf $(4')^2 / 2 = 1320$ lb-ft/ft ∴ 1320 lb-ft/ft ←
2.7	d	p. 1-32 - Site Class & *ASCE 7-16* p. 203 - *§20.3.1, item 1* *Site Class* F = soils vulnerable to potential failure or collapse under seismic loading (e.g., liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils) ∴ All the above ←
2.8	a	p. 1-94 - Wall Anchorage Forces & *ASCE 7-16* p. 108 - *§12.11.2.1* *Site Class* D & $S_S = 0.65$ → *Table 3.1* → $S_{DS} = 0.56$ $L_f = 125'$ for flexible diaphragm (given) $K_a = 1.0 + \dfrac{L_f}{100} = 1.0 + (125' / 100') = 2.25 > 2.0$ max → use $K_a = 2.0$ $I_e = 1.5$ – *ASCE 7-16* p. 5 - *Table 1.5-2* for Police station (*RC* = IV) $W_{wall} = 150$ pcf (8″ wall thickness) (1 ft / 12″) = 100 psf $W_p = W_{wall}(h_w/2 + h_p)$ … for (one-story) walls with a parapet $= (100\text{ psf})(14'/2 + 2.5') = 950$ plf *(continued)*

Problem	Answer	Reference / Solution
		∴ 2-story Apartment building assigned to *SDC* = D ←
2.13	a	p. 1-124 - Center of Rigidity, *CR* By observation, the *CR* will be located in the center of the 125 foot building dimension (in the *X*-direction) because the rigidity on the left wall line is equal to the rigidity on the right wall line (i.e., $R = 2$). Also, by observation, the *CR* will be located above the center of the 75 foot building dimension (in the *Y*-direction) because the total rigidity on the top wall line is greater than the total rigidity on the bottom wall line. $\overline{X}_{CR} = \dfrac{\sum R_y \overline{x}}{\sum R_y} = 125'/2 = 62.5'$ (by observation) $\overline{Y}_{CR} = \dfrac{\sum R_x \overline{y}}{\sum R_x} = \dfrac{1.5(0)+1.0(75')+1.0(75')}{1.5+1.0+1.0} = 42.9'$ ∴ (62.5′, 42.9′) ←
2.14	b	p. 1-50 - Table 4.7b & *ASCE 7-16* p. 93 - *§12.2.3.1* Combination of framing systems in the same direction – Vertical Combination $R = 4$ → *ASCE 7-16* p. 90 - *Table 12.2-1, item A.18* – light-frame (cold-formed steel) wall systems with flat strap bracing (upper 3 stories). $R = 5$ → *ASCE 7-16* p. 90- *Table 12.2-1, item A.7* – special reinforced masonry shear walls (1st story). Where the upper system has a lower *R*, the design coefficients (R, Ω_0, and C_d) for the upper system shall be used for both systems (i.e., both the upper and lower systems). ∴ Vertical combination, $R = 4$ ←
2.15	a	*ASCE 7-16* p. 126 & 129 - *Table 13.5-1, footnote b* & *Table 13.6-1, footnote c* Overstrength as required for (nonductile) anchorage to concrete and masonry ∴ to design nonstructural component anchorage to concrete or masonry
2.16	a	p. 1-45 - Dual Systems & *ASCE 7-16* p. 91 to 92 - *Table 12.2-1 (type D & E)*, and p. 91 - *§12.2.5.1* Moment frames (SMF or IMF) shall be designed to independently resist at least 25% of the design seismic forces. ∴ at least 25% of the design seismic forces ←
2.17	d	p. 1-81 - Basic (SD or LRFD) Load Combinations & *2018 IBC* p. 358 - *§1605.2* By observation - *IBC* equation *(16-5)* will govern for the maximum shear in the column (i.e., *IBC* equation *(16-7)* will clearly provide a lower shear). $D = 15$ kips (given) $L = 9$ kips (given) … due to Office floor live load *(continued)*

Problem	Answer	Reference / Solution
		$C_S = \dfrac{S_{D1}}{T(R/I_e)}$ *ASCE 7 (12.8-3)* $= \dfrac{1.03}{0.72(4/1.5)} = \underline{0.537}$ ← governs C_S <u>shall not</u> be less than: $C_S = 0.044 S_{DS} I_e$ *ASCE 7 (12.8-5)* $= 0.044\,(1.65)(1.5) = 0.109 << 0.537$ In addition, when $S_1 \geq 0.6$, C_S <u>shall not</u> be less than: $C_S = \dfrac{0.5S_1}{(R/I_e)}$ *ASCE 7 (12.8-6)* $= \dfrac{0.5(1.03)}{(4/1.5)} = 0.193 << 0.537$ $V = C_S W$ *ASCE 7 (12.8-1)* $= 0.537$ (4,020 lbs) = <u>2,160 lbs</u> ∴ <u>2200 lbf</u> ←
2.25	a	p. 1-116 - Flexible Diaphragm Analysis $w_s = V / L =$ (35 kips) / (40′ + 55′) = 0.368 klf <u>Line 1</u>: $V_1 = w_s L_1 / 2$ = (0.368 klf)(40′) / 2 = 7.36 kips Unit roof shear $v_1 = V_1 / d$ = (7.36 kips) / (60′) = 0.123 klf Max drag force, F_d = (roof v_1)(25′) = (0.123 klf)(25′) = 3.08 kips <u>Line 2</u>: $V_2 = w_s L_1 / 2 + w_s L_2 / 2 = V / 2$ = 17.5 kips Total (combined) unit roof shear $v_2 = V_2 / d$ = (17.5 kips) / (60′) = 0.292 klf Max drag force, F_d = (roof v_2)(27′) = (0.292 klf)(27′) = <u>7.88 kips</u> ← governs <u>Line 3</u>: $V_3 = w_s L_2 / 2$ = (0.368 klf)(55′) / 2 = 10.12 kips Unit roof shear $v_3 = V_3 / d$ = (10.12 kips) / (60′) = 0.169 klf Max drag force, F_d = (roof v_3)(35′) = (0.169 klf)(35′) = 5.92 kips ∴ <u>7.9 kips</u> ←
2.26	d	p. 1-82 - Seismic Design Force & *ASCE 7-16* p. 88 & 89 - *§13.3.1* $S_{DS} = 0.58$ (given) $I_p = 1.5$ … equipment is needed for continued operation of this *RC* = IV emergency shelter Spring-isolated component → *ASCE 7-16 – Table 13.6-1* (Vibration-isolated components, 2nd line) → $a_p = 2½$ & $R_p = 2$ W_p = 1,500 lbs (given) $z = h_1 = 12′$ – since pipe is suspended from the 2nd floor (i.e., Level 1) $h = h_6$ = (6 stories)(12 ft/story) = 72′ $z / h = 12′ / 72′ = 0.167$ $R_p / I_p = (2 / 1.5) = 1.33$ $F_p = \dfrac{0.4 a_p S_{DS} W_p}{(R_P/I_P)}\left(1 + 2\dfrac{z}{h}\right)$ *ASCE 7 (13.3-1)* *(continued)*