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*)

Chapter 6

Seismic Design Requirements for Nonstructural Components

6.1 ASCE 7 – Chapter 13 Overview

Generally, a building can be defined as an enclosed structure intended for human occupancy. While the building includes the structural elements of the vertical (i.e., gravity) force-resisting systems and lateral force-resisting systems, it also includes nonstructural components (e.g., exterior cladding, interior walls and partitions, ceilings, HVAC systems, mechanical systems, electrical systems, etc.) permanently attached to and supported by the structure.

According to *FEMA E-74 – Reducing the Risks of Nonstructural Earthquake Damage - A Practical Guide* - nonstructural failures have accounted for the majority of damage in recent earthquakes. In terms of construction cost, typically < 20% is structural while > 80% is nonstructural which includes architectural components, mechanical/electrical/plumbing (MEP) components, furniture, fixtures and equipment.

Scope — *ASCE 7 – §13.1.1*

ASCE 7-16 – Chapter 13 establishes minimum design criteria for *nonstructural components* that are permanently attached to structures, and for their supports and attachments.

A *nonstructural component* is a part or element of an architectural, mechanical or electrical system.

Seismic Design Category — *ASCE 7 – §13.1.2*

Nonstructural components shall be assigned to the same *Seismic Design Category* (*SDC*) as the structure that they occupy, or to which they are attached.

Component Importance Factor, I_p — *ASCE 7 – §13.1.3*

All components shall be assigned a *component importance factor* (I_p), which will be equal to 1.5 or 1.0. Use an I_p = 1.5 if any of the following conditions apply:

1. The component is required to function for life-safety purposes after an earthquake, including fire protection sprinkler systems and egress stairways
2. The component conveys, supports, or otherwise contains toxic, highly toxic, or explosive substances …
3. The component is in (or attached to) a *Risk Category* IV structure (i.e., essential facility), **and** it is needed for continued operation of the facility **or** its failure could impair the continued operation of the facility
4. The component conveys, supports, or otherwise contains hazardous substances …

All other components shall be assigned an I_p = 1.0

Exemptions — *ASCE 7 – §13.1.4*

The following *nonstructural components* are exempt from the requirements of *ASCE 7-16 – Chapter 13*:

1. Furniture (except floor-supported storage cabinets > 6 feet tall, etc.)
2. Temporary or movable equipment
3. Architectural components in *SDC* = B (other than parapets) provided I_p = 1.0

- Wall B: $h/b_s = (12' / 4') = 3.00 > 2{:}1 \rightarrow$ 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
12.7	c	p. 1-177 & 1-194 - Welded Steel Moment Frames Typical damage characteristics … … welded connection failure at the beam-column joints due to inadequate strength and ductility, and column web fractures due to inadequate panel zone strength and ductility. ∴ welded steel moment frames ←
12.8	a	p. 1-197 - Retrofit of Existing Structures - *compatibility* Stiff architectural elements (brick veneer) are not compatible with more flexible structural systems (e.g., steel SMF) and the architectural elements are likely to suffer damage during an earthquake (unless designed to accommodate the story drifts). ∴ Steel SMF with exterior brick veneer ←
12.9	d	p. 1-197 - Retrofit of Existing Structures Adding steel jackets to concrete bridge pier is intended to increase the … ductility and shear capacity. ∴ Add ductility (strength) ←
12.10	c	p. 1-197 - Retrofit of Existing Structures Adding stiffness will reduce deflection (i.e., story drift) … reducing likelihood of non-structural (i.e., architectural) damage. ∴ Add stiffness ←
12.11	c	p. 1-197 - Retrofit of Existing Structures Adding stiffness will reduce deflection (i.e., total drift) … decreasing required building separation. ∴ Add stiffness ←
12.12	b	p. 1-197 - Retrofit of Existing Structures Damping system will reduce inelastic demand on beam/column joints (i.e., steel jackets not practical at "joints"). ∴ Damping system ←
12.13	c	p. 1-197 - Retrofit of Existing Structures Adding stiffness will reduce deflection (i.e., story drift) … elminating the "soft" story ∴ Add stiffness ←
12.14	a	p. 1-197 - Retrofit of Existing Structures Base isolation is typically the least disruptive to the historic "fabric" of a historic building (but it is also very expensive). ∴ Base isolation ←
12.15	d	p. 1-197 - Retrofit of Existing Structures Steel moment-resisting frames (i.e., SMF, IMF or OMF) will provide the most "open" retrofit scenario while adding lateral strength and stiffness. ∴ Steel moment-resisting frames ←

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* → $\Delta_{ax} \le 0.020\, h_{sx} = 0.020$ (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 $F_p = \frac{0.4 a_p S_{DS} W_p}{(R_P / I_P)}\left(1 + 2\frac{z}{h}\right)$ *ASCE 7 (13.3-1)* = 0.4(2½)(0.92) W_p [1 + 2 (1.0)] / (1.67) = 1.65 W_p ← (governs) maximum $F_p \le 1.6 S_{DS} I_p W_p$ *ASCE 7 (13.3-2)* = 1.6(0.92)(1.5) W_p = 2.21 W_p minimum $F_p \ge 0.3 S_{DS} I_p W_p$ *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) $K_a = 1.0 + \frac{L_f}{100}$ = 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 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 $w_s = V / L = (35 \text{ kips}) / (40' + 55') = 0.368$ klf <u>Line 1</u>: $V_1 = w_s L_1 / 2 = (0.368 \text{ klf})(40') / 2 = 7.36$ kips Unit roof shear $v_1 = V_1 / d = (7.36 \text{ kips}) / (60') = 0.123$ 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 $v_2 = V_2 / d = (17.5 \text{ kips}) / (60') = 0.292$ klf Max drag force, F_d = (roof v_2)(27′) = (0.292 klf)(27′) = <u>7.88 kips</u> ← governs <u>Line 3</u>: $V_3 = w_s L_2 / 2 = (0.368 \text{ klf})(55') / 2 = 10.12$ kips Unit roof shear $v_3 = V_3 / d = (10.12 \text{ kips}) / (60') = 0.169$ 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)*

Problem	**Answer**	**Reference / Solution**
2.41	c	p. 1-96 - Nonbuilding Structures Supported by Other Structures & *ASCE 7-16* p. 146 - *15.3* Water storage tank required to maintain water pressure for fire suppression $\rightarrow$ *IBC Table 1604.5* $\rightarrow$ RC = IV $I_e = 1.5$ – *ASCE 7-16* p. 5 - *Table 1.5-2* for RC = IV Total effective seismic weight, W = 450 kips + 50 kips = 500 kips Weight of tank to total weight = W_p / W = 450 kips / 500 kips = 90% > 25% $\rightarrow$ use *15.3.2, item 1* Steel special concentrically braced frames $\rightarrow$ *ASCE 7-16 – Table 12.2-1, Type B.2* $\rightarrow$ $R = 6$ *Site Class* D & $S_S = 1.04$ $\rightarrow$ Table 3.1 $\rightarrow$ $S_{DS} = 0.75$ (by interpolation) *Site Class* D & $S_1 = 0.45$ $\rightarrow$ Table 3.2 $\rightarrow$ $S_{D1} = 0.56$ (by interpolation) $T_S = S_{D1}/S_{DS} = (0.56)/(0.75) = 0.75$ second $T = 0.55$ sec (given) < $T_s = 0.75$ sec $\rightarrow$ *ASCE 7 (12.8-2)* <u>will</u> govern for C_S $C_S = \dfrac{S_{DS}}{(R/I_e)}$ *ASCE 7 (12.8-2)* $= \dfrac{0.75}{(6/1.5)} = 0.188$ $V = C_S W$ *ASCE 7 (12.8-1)* $= 0.188$ (500 kips) = 94 kips $\therefore$ <u>94 kips</u> $\leftarrow$
2.42	c	p. 1-124 - Center of Mass, *CM* <u>By inspection</u>: $\overline{X}_{CM}$ should be slightly <u>greater than</u> $120'/2 = 60'$ and $\overline{Y}_{CM}$ should be slightly <u>less than</u> $80'/2 = 40'$ … which eliminates choices a, b & d (i.e., c must be the correct answer) <u>OR by calculation</u>: Wall weights W_w = 20 kips (given for 5 walls) Roof weight $W_1 = (120')(80' - 20')(80 \text{ psf}) = 576$ kips Roof weight $W_2 = W_3 = (40')(20')(80 \text{ psf}) = 64$ kips ΣW = 5 walls (20 kips) + 576 kips + 2 (64 kips) = 804 kips $\overline{X}_{CM} = \dfrac{\sum W \overline{x}}{\sum W}$ $= \dfrac{20^K(0'+20'+100'+100'+120') + 576^K(60') + 64^K(20'+100')}{804^K} = 61.0'$ $\overline{Y}_{CM} = \dfrac{\sum W \overline{y}}{\sum W}$ $= \dfrac{20^K(0'+20'+20'+80'+80') + 576^K(30') + 64^K(70'+70')}{804^K} = 37.6'$ $\therefore$ <u>(61.0′, 37.6′)</u> $\leftarrow$

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