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. \[ 1.2D + E_v + E_{mh} + L + 0.2S \]
   \[ \text{or ... } (1.2 + 0.2S_{DS})D + \Omega_0Q_E + L + 0.2S \]

7. \[ 0.9D - E_v + E_{mh} \]
   \[ \text{or ... } (0.9 - 0.2S_{DS})D - \Omega_0Q_E \]

**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. \[ 1.0D + 0.7E_v + 0.7E_{mh} \]
   \[ \text{or ... } (1.0 + 0.14S_{DS})D + 0.7\Omega_0Q_E \]

9. \[ 1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S \]
   \[ \text{or ... } (1.0 + 0.105S_{DS})D + 0.525\Omega_0Q_E + 0.75L + 0.75S \]

10. \[ 0.6D - 0.7E_v + 0.7E_{mh} \]
    \[ \text{or ... } (0.6 - 0.14S_{DS})D - 0.7\Omega_0Q_E \]

**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 \( (\Omega_0) \) 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 \( (\Omega_0) \) 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 \( (\Omega_0) \) 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: \( h/b_v = (12' / 4') = 3.00 > 2:1 \) → use \( 2b_v/h = 2(4' / 12') = 0.67 \) reduction in unit shear capacity

\[
\text{Capacity of Wall B} = 520 \text{ plf} \times (2b_v/h) = 520 \text{ plf} (0.67)(4') = 1,390 \text{ lbs}
\]

\[
V_A = [(3,640 \text{ lbs}) / (3,640 \text{ lbs} + 1,390 \text{ lbs})] V_1 = 72\% V_1 \quad \leftarrow
\]

\[
V_B = [(1,390 \text{ lbs}) / (3,640 \text{ lbs} + 1,390 \text{ lbs})] V_1 = 28\% V_1 \quad \leftarrow
\]

**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 11/4", 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 11/4", 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 \( \leq 980 \text{ plf} \) for seismic (i.e., \( \leq 490 \text{ plf} \) for ASD) or \( \leq 1370 \text{ plf} \) for wind (i.e., \( \leq 685 \text{ plf} \) 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 $SDS = 0.72$?
   a. ± 16 kips  
   b. ± 22 kips  
   c. ± 110 kips  
   d. ± 132 kips

5.24 Given a redundancy factor $\rho = 1.3$, 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. $1\frac{1}{4}$  
   c. $1\frac{1}{2}$  
   d. $2\frac{1}{2}$

6.2 What type of anchorage might require the use of the $\Omega_0$ 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
2.5  b  p. 1-66 to 67 - Story Drift Limit, $\Delta_{ax}$ & ASCE 7-16 p. 109 - §12.12.1
Medical Office building $\rightarrow$ IBC Table 1604.5 $\rightarrow$ RC = II
5-stories > 4-stories $\rightarrow$ “All other Structures” $\rightarrow$ Table 12.12-1 $\rightarrow$
$\Delta_{ax} \leq 0.020 \ h_{ax} = 0.020 (13 \text{ ft})(12 \text{ in/ft}) = 3.12 \text{ inches}$
$\therefore 3.1 \text{ inches} \leftarrow$

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 = 2\frac{1}{2} \& R_p = 2\frac{1}{2} – Table 13.5-1 - Cantilever elements (unbraced or braced to structural frame below its center of mass) - parapets
$z = h$ $\rightarrow$ use $(z/h) = 1.0$
$I_p = 1.5 \text{ per ASCE 7-16 – §13.1.3}$ since the failure of the parapet could affect the continuous operation of this RC = IV Police station.
$\frac{R_p}{I_p} = \left(\frac{2\frac{1}{2}}{1.5}\right) = 1.67$
$F_p = \frac{0.4 a_p S_{DS} W_p}{(R_p / I_p)} \left(1 + 2 \frac{z}{h}\right) \text{ ASCE 7 (13.3-1)}$
$= 0.4(2\frac{1}{2})(0.92) \ W_p \left[1 + 2 (1.0)\right] / (1.67) = 1.65 \ W_p \leftarrow \text{(governs)}$
maximum $F_p \leq 1.6 S_{DS} I_p W_p \text{ ASCE 7 (13.3-2)}$
$= 1.6(0.92)(1.5) \ W_p = 2.21 \ W_p$
minimum $F_p \geq 0.3 S_{DS} I_p W_p \text{ ASCE 7 (13.3-3)}$
$= 0.3(0.92)(1.5) \ W_p = 0.41 \ W_p$
$f_p = 1.65 (100 \text{ psf}) = 165 \text{ psf}$ - uniform load acting over the parapet height
The bending moment at the roof level –
$M = f_p h_p^2 / 2 = 165 \text{ psf} (4')^2 / 2 = 1320 \text{ lb-ft/ft}$
$\therefore 1320 \text{ lb-ft/ft} \leftarrow$

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)
$\therefore \text{ All the above} \leftarrow$

2.8  a  p. 1-94 - Wall Anchorage Forces & ASCE 7-16 p. 108 - §12.11.2.1
Site Class D & $S_S = 0.65$ $\rightarrow$ Table 3.1 $\rightarrow$ $S_{DS} = 0.56$
$L_f = 125'$ for flexible diaphragm (given)
$K_a = 1.0 + \frac{L_f}{100} = 1.0 + (125' / 100') = 2.25 > 2.0 \text{ max} \rightarrow \text{ use } K_a = 2.0$
$I_c = 1.5 – ASCE 7-16 p. 5 - Table 1.5-2$ for Police station ($RC = IV$)
$W_{wall} = 150 \text{ pcf} (8'' \text{ wall thickness}) (1 \text{ ft} / 12'') = 100 \text{ psf}$
$W_p = W_{wall} \left(\frac{h_n}{2} + h_p\right) \ldots \text{ for (one-story) walls with a parapet}$
$= (100 \text{ psf})(14' / 2 + 2.5') = 950 \text{ plf}$
(continued)
:: 2-story Apartment building assigned to $SDC = D$ ←

<table>
<thead>
<tr>
<th>Problem</th>
<th>Answer</th>
<th>Reference / Solution</th>
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</thead>
<tbody>
<tr>
<td>2.13</td>
<td>a</td>
<td><a href="#">p. 1-124 - Center of Rigidity, $CR$</a> 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. $X_{CR} = \frac{\sum R_x x}{\sum R_x} = 125' / 2 = 62.5'$ (by observation) $Y_{CR} = \frac{\sum R_y y}{\sum R_y} = \frac{1.5(0) + 1.0(75') + 1.0(75')}{1.5 + 1.0 + 1.0} = 42.9'$ $\therefore (62.5', 42.9')$ ←</td>
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<tr>
<td>2.14</td>
<td>b</td>
<td><a href="#">p. 1-50 - Table 4.7b &amp; ASCE 7-16 p. 93 - §12.2.3.1</a> Combination of framing systems in the same direction – Vertical Combination $R = 4 \rightarrow ASCE 7-16$ p. 90 - Table 12.2-1, item A.18 – light-frame (cold-formed steel) wall systems with flat strap bracing (upper 3 stories). $R = 5 \rightarrow ASCE 7-16$ 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 ($R$, $\Omega_0$, and $C_d$) for the upper system shall be used for both systems (i.e., both the upper and lower systems). $\therefore$ Vertical combination, $R = 4$ ←</td>
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<tr>
<td>2.15</td>
<td>a</td>
<td><a href="#">ASCE 7-16 p. 126 &amp; 129 - Table 13.5-1, footnote b &amp; Table 13.6-1, footnote c</a> Overstrength as required for (nonductile) anchorage to concrete and masonry $\therefore$ to design nonstructural component anchorage to concrete or masonry</td>
</tr>
<tr>
<td>2.16</td>
<td>a</td>
<td><a href="#">p. 1-45 - Dual Systems &amp; ASCE 7-16 p. 91 to 92 - Table 12.2-1 (type D &amp; E), and p. 91 - §12.2.5.1</a> Moment frames (SMF or IMF) shall be designed to independently resist at least 25% of the design seismic forces. $\therefore$ at least 25% of the design seismic forces ←</td>
</tr>
<tr>
<td>2.17</td>
<td>d</td>
<td><a href="#">p. 1-81 - Basic (SD or LRFD) Load Combinations &amp; 2018 IBC p. 358 - §1605.2</a> 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)</td>
</tr>
<tr>
<td>Problem</td>
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| **2.25** | **a** | p. 1-116 - Flexible Diaphragm Analysis  
\[w_s = V / L = (35 \text{ kips}) / (40' + 55') = 0.368 \text{ klf}\]  
Line 1: \[V_1 = w_s L_1 / 2 = (0.368 \text{ klf})(40') / 2 = 7.36 \text{ kips}\]  
Unit roof shear \[v_1 = V_1 / d = (7.36 \text{ kips}) / (60') = 0.123 \text{ klf}\]  
Max drag force, \[F_d = (roof v_1)(25') = (0.123 \text{ klf})(25') = 3.08 \text{ kips}\]  
Line 2: \[V_2 = w_s L_2 / 2 = V_2 / d = 17.5 \text{ kips}\]  
Total (combined) unit roof shear \[v_2 = V_2 / d = (17.5 \text{ kips}) / (60') = 0.292 \text{ klf}\]  
Max drag force, \[F_d = (roof v_2)(25') = (0.292 \text{ klf})(25') = 7.88 \text{ kips} \leftarrow\]  
Line 3: \[V_3 = w_s L_3 / 2 = (0.368 \text{ klf})(55') / 2 = 10.12 \text{ kips}\]  
Unit roof shear \[v_3 = V_3 / d = (10.12 \text{ kips}) / (60') = 0.169 \text{ klf}\]  
Max drag force, \[F_d = (roof v_3)(35') = (0.169 \text{ klf})(35') = 5.92 \text{ kips}\]  
\[\therefore 7.9 \text{ kips} \leftarrow\] |
| **2.26** | **d** | p. 1-82 - Seismic Design Force & *ASCE 7-16* p. 88 & 89 - §13.3.1  
\[S_{DS} = 0.58 \text{ (given)}\]  
\[I_p = 1.5 \text{ ... equipment is needed for continued operation of this RC = IV emergency shelter}\]  
Spring-isolated component \[\rightarrow \text{ASCE 7-16 – Table 13.6-1 (Vibration-isolated components, 2nd line)} \rightarrow a_p = 2\frac{1}{2} \& R_p = 2\]  
\[W_p = 1,500 \text{ lbs} \text{ (given)}\]  
\[z = h_1 = 12' \text{ – since pipe is suspended from the 2nd floor (i.e., Level 1)}\]  
\[h = h_6 = (6 \text{ stories})(12 \text{ ft/story}) = 72'\]  
\[z / h = 12' / 72' = 0.167\]  
\[R_p / I_p = (2 / 1.5) = 1.33\]  
\[F_p = \frac{0.4a_p S_{DS} W_p}{(R_p / I_p)} \left(1 + 2 \frac{z}{h}\right) \text{ ASCE 7 (13.3-1)}\]  
(continued)
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 → IBC Table 1604.5 → 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% → use 15.3.2, item 1

Steel special concentrically braced frames → ASCE 7-16 – Table 12.2-1, Type B.2 → $R = 6$

Site Class D & $S_S = 1.04$ → Table 3.1 → $S_{DS} = 0.75$ (by interpolation)

Site Class D & $S_1 = 0.45$ → Table 3.2 → $S_{D1} = 0.56$ (by interpolation)

$T_s = S_{D1}/S_{DS} = (0.56)/(0.75) = 0.75$ sec

$T = 0.55$ sec (given) < $T_s = 0.75$ sec → ASCE 7 (12.8-2) will govern for $C_S$

$C_S = S_{DS} (R/I_e)$  

$= 0.75 (6/1.5) = 0.188$

$V = C_S W$  

$= 0.188 (500$ kips$) = 94$ kips

∴ 94 kips ←

2.42  c  p. 1-124 - Center of Mass, $CM$

By inspection: $X_{CM}$ should be slightly greater than $120' / 2 = 60'$ and $Y_{CM}$ should be slightly less than $80' / 2 = 40'$ … which eliminates choices a, b & d (i.e., c must be the correct answer)

OR by calculation:

Wall weights $W_w = 20$ kips (given for 5 walls)

Roof weight $W_1 = (120')(80' - 20')(80 $psf$) = 576$ kips

Roof weight $W_2 = W_3 = (40')(20')(80 $psf$) = 64$ kips

$\sum W = 5$ walls (20 kips) + 576 kips + 2 (64 kips) = 804 kips

$X_{CM} = \frac{\sum W \bar x}{\sum W}$

$= \frac{20^K (0' + 20' + 100' + 100' + 120') + 576^K (60') + 64^K (20' + 100')}{804^K} = 61.0'\ K$

$Y_{CM} = \frac{\sum W \bar y}{\sum W}$

$= \frac{20^K (0' + 20' + 20' + 80' + 80') + 576^K (30') + 64^K (70' + 70')}{804^K} = 37.6'\ K$

∴ (61.0', 37.6') ←