number of well-defined fault systems, so deterministic MCE$_R$ ground motions are used to estimate the levels of ground shaking in those regions.

S_S shall be determined from the 0.2 second (i.e., short period) mapped MCE$_R$ spectral response accelerations, and S_1 shall be determined from the 1-second mapped MCE$_R$ spectral response accelerations shown on *IBC Figures 1613.2.1(1)* through *1613.2.1(10)* which are based on the national seismic hazard study conducted by the United States Geologic Survey (USGS).

NOTE: where $S_S \leq 0.15$ and $S_1 \leq 0.04$, the structure may be assigned to *Seismic Design Category* A.

Earthquake Ground Motion Parameters are most easily, and more accurately, determined utilizing the *ASCE 7 Hazard Tool* at - https://asce7hazardtool.online/ with the input of street address, or Latitude and Longitude, or Find on the Map, to calculate the appropriate response parameters (S_S & S_1, F_a & F_v, S_{MS} & S_{M1}, and S_{DS} & S_{D1}), MCE$_R$ response spectrum, design response spectrum, etc.

Site Class Definitions *IBC §1613.2.2*

Based on the site soil properties, the site shall be classified as either *Site Class* A, B, C, D, E or F in accordance with *ASCE 7-16 – Chapter 20.*

Site Classification *ASCE 7 – §20.1*

The *Site Class* shall be determined in accordance with *ASCE 7-16 – Table 20.3-1* and *§20.3* based on the upper 100 feet of the site profile.

Site class definitions are dependent on one or more of the following average soil properties:

- ✓ shear wave velocity ($\bar{v}_s$)
- ✓ standard penetration resistance ($\bar{N}$ or $\bar{N}_{ch}$)
- ✓ undrained shear strength ($\bar{s}_u$)

Site Class

A = Hard Rock (extremely rare in California … East coast only) with $\bar{v}_s > 5{,}000$ ft/second

B = Rock with $2{,}500 \leq \bar{v}_s \leq 5{,}000$ ft/second

C = Very dense soil & soft rock with $1{,}200 \leq \bar{v}_s \leq 2{,}500$ ft/second, etc.

D* = Stiff soil with $600 \leq \bar{v}_s \leq 1{,}200$ ft/second, etc. ←

E = Soft clay soil with $\bar{v}_s < 600$ ft/second, etc. … **or** any profile with > 10 feet of soil having the following characteristics:

1. Plasticity index $PI > 20$,
2. Moisture content $w \geq 40\%$, and
3. Undrained shear strength $\bar{s}_u < 500$ psf

F = Soil (requiring site-specific evaluation per *ASCE 7-16 – §20.3.1*) with the following characteristics:

1. Soils vulnerable to potential failure or collapse under seismic loading (e.g., liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils)
2. Peats and/or highly organic clays, where thickness of peat and/or highly organic clay $H > 10$ feet
3. Very high plasticity clays ($H > 25$ feet with $PI > 75$)
4. Very thick soft/medium stiff clay ($H > 120$ feet) with $s_u < 1{,}000$ psf

Table 3.2 - Design Spectral Response Acceleration Parameter at 1-Second Period (S_{D1})						
S_1	*Site Class*					
	A	B[1]	C	D[2]	E[3]	F
0.10	**0.053**	**0.053**	**0.100**	**0.160**	**0.280**	Values shall be determined per *ASCE 7-16 § 21.1*
0.12	0.064	0.064	0.120	0.189	0.322	
0.14	0.075	0.075	0.140	0.217	0.358	
0.16	0.085	0.085	0.160	0.243	0.390	
0.18	0.096	0.096	0.180	0.269	0.418	
0.20	**0.107**	**0.107**	**0.200**	***0.440***	***0.440***	
0.22	0.117	0.117	0.220	*0.475*	*0.469*	
0.24	0.128	0.128	0.240	*0.509*	*0.496*	
0.26	0.139	0.139	0.260	*0.541*	*0.520*	
0.28	0.149	0.149	0.280	*0.571*	*0.541*	
0.30	**0.160**	**0.160**	**0.300**	***0.600***	***0.560***	
0.32	0.171	0.171	0.320	*0.634*	*0.580*	
0.34	0.181	0.181	0.340	*0.666*	*0.598*	
0.36	0.192	0.192	0.360	*0.698*	*0.614*	
0.38	0.203	0.203	0.380	*0.730*	*0.628*	
0.40	**0.213**	**0.213**	**0.400**	***0.760***	***0.640***	
0.42	0.224	0.224	0.420	*0.790*	*0.661*	
0.44	0.235	0.235	0.440	*0.818*	*0.681*	
0.46	0.245	0.245	0.460	*0.846*	*0.699*	
0.48	0.256	0.256	0.480	*0.874*	*0.717*	
0.50	**0.267**	**0.267**	**0.500**	***0.900***	***0.733***	
0.52	0.277	0.277	0.513	*0.926*	*0.749*	
0.54	0.288	0.288	0.526	*0.950*	*0.763*	
0.56	0.299	0.299	0.538	*0.974*	*0.777*	
0.58	0.309	0.309	0.549	*0.998*	*0.789*	
0.60	**0.320**	**0.320**	**0.560**	***1.020***	***0.800***	
0.65	0.347	0.347	0.607	*1.105*	*0.867*	
0.70	**0.373**	**0.373**	**0.653**	***1.190***	***0.933***	
0.75	0.400	0.400	0.700	*1.275*	*1.000*	
0.80	**0.427**	**0.427**	**0.747**	***1.360***	***1.067***	
0.85	0.453	0.453	0.793	*1.445*	*1.133*	
0.90	**0.480**	**0.480**	**0.840**	***1.530***	***1.200***	
0.95	0.507	0.507	0.887	*1.615*	*1.267*	
1.00	**0.533**	**0.533**	**0.933**	***1.700***	***1.333***	
1.05	0.560	0.560	0.980	*1.785*	*1.400*	
1.10	**0.587**	**0.587**	**1.027**	***1.870***	***1.467***	
1.15	0.613	0.613	1.073	*1.955*	*1.533*	
1.20	**0.640**	**0.640**	**1.120**	***2.040***	***1.600***	
1.25	0.667	0.667	1.167	*2.125*	*1.667*	
1.30	**0.693**	**0.693**	**1.213**	***2.210***	***1.733***	

1. Where site investigations reveal rock conditions consistent with *Site Class* B, but site-specific velocity measurements were **NOT** made, multiply the values in this column by (1.0 / 0.8) to determine S_{D1}
2. For *Site Class* D with $S_1 \geq 0.2$ – in lieu of providing a GMHA, values in ***italics*** may be used
3. For *Site Class* E with $S_1 \geq 0.2$ – in lieu of providing a GMHA, values in ***italics*** may be used provided that the Equivalent Lateral Force (ELF) procedure is used for design and C_S is determined by *ASCE 7 (12.8-2)* for all values of T

3.7 ASCE 7 Seismic Design Criteria *ASCE 7 – Chapter 11*

Scope *ASCE 7 – §11.1.2*

Every structure (e.g., buildings and nonbuilding structures) and portion thereof, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions as prescribed by the seismic requirements of *ASCE 7-16*.

Applicability *ASCE 7 – §11.1.3*

Structures and their nonstructural components shall be designed and constructed in accordance with the requirements of the following chapters based on the type of structure or component:

- Buildings: *ASCE 7-16 – Chapter 12*
- Nonbuilding Structures: *ASCE 7-16 – Chapter 15*
- Nonstructural Components: *ASCE 7-16 – Chapter 13*
- Seismically Isolated Structures: *ASCE 7-16 – Chapter 17*
- Structures with Damping Systems: *ASCE 7-16 – Chapter 18*

Seismic Importance Factor, I_e *ASCE 7 – §11.5.1*

Each structure shall be assigned an *importance factor* (I_e) in accordance with *ASCE 7-16 – Table 1.5-2* … based on the *Risk Category* of the building (or other structure) from *IBC – Table 1604.5*.

- *Risk Category* I → $I_e = 1.0$
- *Risk Category* II → $I_e = 1.0$
- *Risk Category* III (high occupancy) → $I_e = 1.25$*
- *Risk Category* IV (essential facilities) → $I_e = 1.5$*

The seismic *importance factor* (I_e) is used in the *Seismic Response Coefficient* (C_S) equations with the intent to raise the yield level for important structures (e.g., hospitals, fire stations, emergency operation centers, hazardous facilities, etc.).

Use of an *importance factor* greater than one is intended to provide for a lower inelastic demand on a structure which should result in lower levels of structural and nonstructural damage.

**Risk Category* III and IV structures will require *Structural Observations* for structures per *IBC §1704.6.1*

Seismic Design Category A *ASCE 7 – §11.7*

Structures may be assigned to *Seismic Design Category* A (i.e., *SDC* = A) under any of the following two conditions:

1. $S_S \leq 0.15$ and $S_1 \leq 0.04$ … per *IBC §1613.2.1*, **OR**
2. $S_{DS} < 0.167$ and $S_{D1} < 0.067$ … per *IBC Tables 1613.2.5(1) & 1613.2.5(2)*

Structures assigned to *Seismic Design Category* A need only comply with the requirements of *ASCE 7-16 – §1.4* (i.e., *ASCE 7-16 – Chapter 12* does not apply).

Nonstructural components assigned to *Seismic Design Category* A are exempt from seismic design requirements (i.e., *ASCE 7-16 – Chapter 13* does not apply).

Tanks assigned to *Risk Category* IV shall satisfy the freeboard requirement in *ASCE 7-16 – §15.6.5.1*.

4.6 Analysis Procedure Selection *ASCE 7 – §12.6*

The structural analysis required by *ASCE 7-16 – Chapter 12* shall consist of one of the types permitted in *ASCE 7-16 – Table 12.6-1* based on the structure's:

- *Seismic Design Category (SDC)*, and
- Structural characteristics –
  - ✓ construction type
  - ✓ number of stories
  - ✓ structure height
  - ✓ structure period
  - ✓ structural irregularities (horizontal or vertical)

The permitted analytical procedures are as follows:

1. **Equivalent Lateral Force (ELF)** - most common procedure, see *ASCE 7-16 – §12.8*
2. **Dynamic Analysis** -
   - Modal Response Spectrum Analysis - see *ASCE 7-16 – §12.9.1*
   - Linear Response History Analysis - see *ASCE 7-16 – §12.9.2*
   - Nonlinear Response History Procedures - see *ASCE 7-16 – Chapter 16*
3. **Simplified Design** - see *ASCE 7-16 – §12.14*, may be used subject to all of the limitations noted

ASCE 7-16 – Table 12.6-1 -

The *Equivalent Lateral Force* (ELF) procedure of *ASCE 7-16 – §12.8* is **permitted** on all structures assigned to *SDC* = B or C, and structures assigned to *SDC* = D, E or F with the following characteristics:

- ➢ *Risk Category* I or II buildings ≤ 2 stories, **or**
- ➢ Structures of light-framed construction (e.g., wood or metal studs), **or**
- ➢ Regular structures ≤ 160 feet in height, **or**
- ➢ Regular structures > 160 feet in height with $T < 3.5\,T_S$, **or**
- ➢ Irregular structures ≤ 160 feet in height **and** having only *horizontal* irregularities of Type 2, 3, 4, or 5 in *Table 12.3-1* or *vertical* irregularities of Type 4, 5a, or 5b in *Table 12.3-2.*

Therefore, the *Equivalent Lateral Force* (ELF) procedure is **not permitted** on structures assigned to *SDC* = D, E or F with the following characteristics:

- ➢ Regular structures > 160 feet in height with $T \geq 3.5\,T_S$ (e.g., tall - long period structures), **or**
- ➢ Irregular structures > 160 feet in height, **or**
- ➢ Irregular structures ≤ 160 feet in height **and** where any of the following applies:
  - ✓ Torsional *horizontal* irregularity - Type 1a per *Table 12.3-1*
  - ✓ Extreme Torsional *horizontal* irregularity - Type 1b per *Table 12.3-1*
  - ✓ Stiffness-Soft Story *vertical* irregularity - Type 1a per *Table 12.3-2*
  - ✓ Stiffness-Extreme Soft Story *vertical* irregularity - Type 1b per *Table 12.3-2*
  - ✓ Weight (Mass) *vertical* irregularity - Type 2 per *Table 12.3-2*
  - ✓ Vertical Geometric *vertical* irregularity - Type 3 per *Table 12.3-2*

NOTE: An alternative generally accepted procedure is permitted to be used when approved by the Authority Having Jurisdiction (AHJ).

Load Combinations for Allowable Stress Design *ASCE 7 – §2.4*

All applicable *Allowable Stress Design* (ASD) load combinations must be considered since the most critical load effect may occur when one or more of the contributing loads (e.g., D, E, L, L_r, S) are not acting.

- D *ASCE 7 (2.4-1)*
- $D + L$ *ASCE 7 (2.4-2)*
- $D + (L_r \text{ or } S \text{ or } R)$ *ASCE 7 (2.4-3)*
- $D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ *ASCE 7 (2.4-4)*
- $D + 0.6W$ *ASCE 7 (2.4-5)*
- $D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$ *ASCE 7 (2.4-6)*
- $0.6D + 0.6W$ *ASCE 7 (2.4-7)*

ASD Load Combinations with Seismic Load Effects *ASCE 7 – §2.4.5*

When a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic ASD load combinations of *ASCE 7 – §2.4.1*:

- $1.0D + 0.7E_v + 0.7E_h$ *ASCE 7 (2.4-8)*

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

- $1.0D + 0.525E_v + 0.525E_h + 0.75L + 0.75S$ *ASCE 7 (2.4-9)*

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

- $0.6D - 0.7E_v + 0.7E_h$ *ASCE 7 (2.4-10)*

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

NOTE: See *ASCE 7-16 – §2.4.5 - Exception 1 & 2* for additional requirements on the equations above (e.g., S in *ASCE 7 (2.4-9)*, $0.9D$ in lieu of $0.6D$ in *ASCE 7 (2.4-10)* for special masonry shear walls, etc.).

Alternative ASD Load Combinations *IBC §1605.2*

In lieu of the *Basic ASD Load Combinations* of *ASCE 7 – §2.4*, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following load combinations:

- $D + L + (L_r \text{ or } S \text{ or } R)$ *IBC (16-1)*
- $D + L + 0.6W$ *IBC (16-2)*
- $D + L + 0.6W + S/2$ *IBC (16-3)*
- $D + L + S + 0.6W/2$ *IBC (16-4)*
- $D + L + S + E/1.4$ *IBC (16-5)*

  or … $(1.0 + 0.14S_{DS})D + L + S + \rho Q_E/1.4$

- $0.9D + E/1.4$ *IBC (16-6)*

  or … $(0.9 - 0.14S_{DS})D - \rho Q_E/1.4$

NOTE: See *IBC §1605.2 - Exception 1 & 2* for crane hook loads, flat roof snow loads ≤ 30 psf, roof live loads ≤ 30 psf, and flat roof snow loads > 30 psf.

NOTE: When using the *Alternative ASD Load Combinations* that include wind or seismic loads, allowable stresses are permitted to be increased, or load combinations reduced, where permitted by the appropriate material chapter or the referenced standards. See *IBC §1605.2* for requirements and conditions when considering wind loads (W) and/or foundations for loadings considering vertical seismic load effects (i.e., $E_v = 0$, etc.).

Load Combinations with Overstrength Factor

Where the seismic load effect with overstrength (Ω_0) is combined with the effects of other loads, the following seismic load combination for structures (not subject to flood or atmospheric ice loads) shall be used.

SD/LRFD Load Combinations with Seismic Overstrength — *ASCE 7 – §2.3.6*

When a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic SD/LRFD load combinations of *ASCE 7 – §2.3.1*:

- $1.2D + E_v + E_{mh} + L + 0.2S$ *ASCE 7 (2.3-6*)*

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

- $0.9D - E_v + E_{mh}$ *ASCE 7 (2.3-7*)*

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

> **NOTE:** See *ASCE 7-16 – §2.3.6 - Exception 1 & 2* for additional requirements on the equations above (e.g., use of $0.5L$ in *ASCE 7 (2.3-6)* where $L_o \leq 100$ psf except for garages or areas occupied as places of public assembly, etc.).

ASD Load Combinations with Seismic Overstrength — *ASCE 7 – §2.4.5*

When a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic ASD load combinations of *ASCE 7 – §2.4.1*:

- $1.0D + 0.7E_v + 0.7E_{mh}$ *ASCE 7 (2.4-8*)*

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

- $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S$ *ASCE 7 (2.4-9*)*

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

- $0.6D - 0.7E_v + 0.7E_{mh}$ *ASCE 7 (2.4-10*)*

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

> **NOTE:** See *ASCE 7-16 – §2.4.5 - Exception 1 & 2* for additional requirements on the equations above (e.g., S in *ASCE 7 (2.4-9)*, $0.9D$ in lieu of $0.6D$ in *ASCE 7 (2.4-10)* special masonry shear walls, etc.).

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*

The maximum F_p force, per *ASCE 7 (13.3-2)*, typically will not govern unless the nonstructural component is located near the roof level of the supporting structure (i.e., $z \approx h$) and $(a_p / R_p) \geq 2.0$

The minimum F_p force, per *ASCE 7 (13.3-3)*, always needs to be checked, particularly when a nonstructural component is located at or near the base of the supporting structure (i.e., $z \approx 0$)

The (horizontal) seismic force (F_p) shall be applied independently in at least two orthogonal horizontal directions in combination with service or operating loads (e.g., D, S, etc.) associated with the component, as appropriate.

For vertically cantilevered systems, the (horizontal) seismic force (F_p) shall be assumed to act in any horizontal direction.

The load combinations and factors of *ASCE 7-16 – §2.3* (SD/LRFD), or *ASCE 7-16 – §2.4* (ASD), or *IBC §1605.2* (ASD) shall be used to design the members and connections that transfer the F_p forces to the supporting structure.

The *redundancy factor* (ρ) may be taken as 1.0 and the SFRS system *overstrength factor* (Ω_0) in *ASCE 7-16 – Table 12.2-1* does not apply. There will be an *overstrength factor* (Ω_0) for nonductile anchorage to concrete and masonry per *ASCE 7-16 – Table 13.5-1, footnote b* and *Table 13.6-1, footnote c*.

Vertical Force

The component shall be designed for a concurrent vertical force of $\pm 0.2S_{DS}W_p$ with the exception of lay-in access floor panels and lay-in ceiling panels.

Component Amplification Factor, a_p

a_p varies from 1 to 2½ and is determined from *ASCE 7-16 – Table 13.5-1* for Architectural components, and *ASCE 7-16 – Table 13.6-1* for Mechanical & Electrical components. a_p represents the dynamic amplification of the component.

- $a_p = 1$ is for rigid components and rigidly attached components.

  Rigid Component - a component (including its attachments) having a fundamental period less than or equal to 0.06 second ($T \leq 0.06$ second).

- $a_p = 2½$ is for flexible components and flexibly attached components.

  Flexible Component - a component (including its attachments) having a fundamental period greater than 0.06 second ($T > 0.06$ second).

Component Response Modification Factor, R_p

R_p varies from 1 to 12 and is determined from *ASCE 7-16 – Table 13.5-1* for Architectural components, and *ASCE 7-16 – Table 13.6-1* for Mechanical & Electrical components. R_p represents the energy absorption capability of the equipment's structure and attachments (similar to R for a building).

Seismic Relative Displacements, D_p — *ASCE 7 – §13.3.2*

The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

Seismic relative displacements (D_p) shall be determined in accordance with the following:

- Displacements within Structures: *ASCE 7-16 – §13.3.2.1*
- Displacements between Structures: *ASCE 7-16 – §13.3.2.2*

Using the load combinations from *ASCE 7-16* and recognizing that horizontal diaphragms are specifically designed to resist only lateral loads (e.g., not vertical gravity loads) … therefore, typically only earthquake (or wind) loads are considered for the design of the diaphragm to resist shear forces (e.g., D, L, L_r, S, etc. do not apply a lateral load on the diaphragm):

- **Allowable Stress Design (ASD) –**

$$\text{Maximum shear} = 0.7E = 0.7\ V_1 = 0.7(w_s L/2)$$

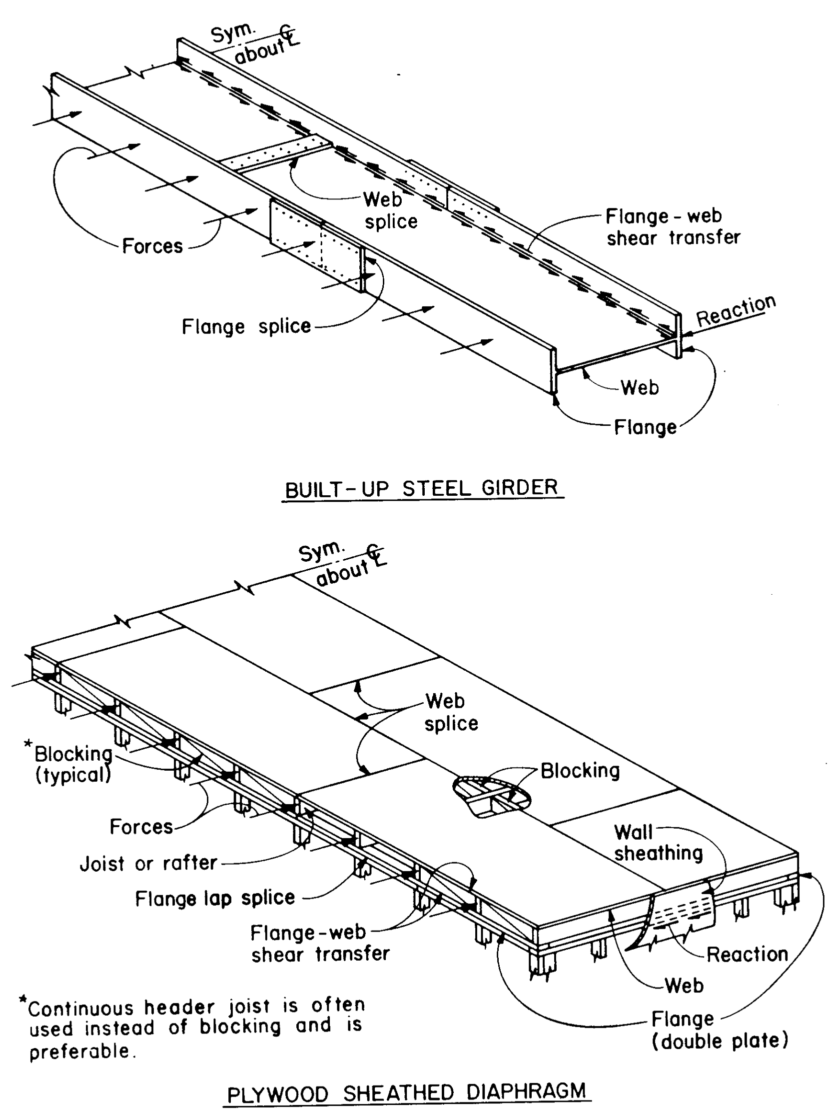
$$\text{Maximum unit diaphragm shear, } v_{max} = \frac{0.7(\text{max. shear})}{\text{diaphragm depth}} = \frac{0.7\ V_1}{d} \text{ (units of plf)}$$

- **Strength Design (SD or LRFD) –**

$$\text{Maximum shear} = 1.0E = 1.0\ V_1 = 1.0(w_s L/2)$$

$$\text{Maximum unit diaphragm shear, } v_{max} = \frac{\text{max. shear}}{\text{diaphragm depth}} = \frac{V_1}{d} \text{ (units of plf)}$$

Figure 8.8 – Diaphragm & Beam Analogy (Ref. 2)



(Applied Technology Council ATC-7 Report)

9.5 Wood-Frame Shear Walls *SDPWS §4.3*

Application Requirements *SDPWS §4.3.1*

Sheathed wood-frame *shear walls* are permitted to resist lateral forces provided the deflection of the shear wall … does not exceed the maximum permissible deflection limit (i.e., drift limits of *ASCE 7 – §12.12.1*).

Framing members, blocking, and connections shall extend into the shear wall a sufficient distance to develop the force transferred into the shear wall.

Shear Wall Types *SDPWS §4.3.2*

Individual Full-Height Wall Segments *SDPWS §4.3.2.1*

Where individual full-height wall segments are designed as shear walls (without openings), the aspect ratio limitations of *SDPWS §4.3.3* shall apply to each full height wall segment as illustrated in *SDPWS Figure 4.D* – see Figure 9.5. The following limitations shall apply:

1. Openings shall be permitted to occur beyond the ends of a shear wall … but the length of the shear wall segments shall exclude the length of any openings.
2. Where out-of-plane (wall) offsets occur, portions of the wall on each side of the offset shall be considered as separate shear wall lines.
3. Collectors for shear transfer to individual full-height wall segments shall be provided.

- **Shear Wall Height, *h***

  The height of a shear wall segment (*h*) is defined as the:

  - maximum clear height from the top of the foundation to bottom of the diaphragm framing above; or
  - maximum clear height from the top of the diaphragm below to bottom of the diaphragm framing above

- **Shear Wall Width, *b***

  The width (i.e., length) of a shear wall (*b*) is defined as the sheathed dimension of the shear wall in the direction of application of force … excluding openings.

Refer to Figure 9.5 for example height-to-width ratios of individual full-height wall segment shear walls.

Force-Transfer Around Openings (FTAO) Shear Walls *SDPWS §4.3.2.2*

Where shear walls *with openings* are designed for force transfer around the openings … the design shall be based on a rational analysis and shall meet *SDPWS §4.3.2.2*.

Perforated Shear Walls *SDPWS §4.3.2.3*

Where wood structural panel (WSP) shear walls *with openings* are not designed for force transfer around openings, they shall be designed as perforated shear walls and shall meet *SDPWS §4.3.2.3*.

Shear Wall Aspect Ratios *SDPWS §4.3.3*

A *Shear wall* aspect ratio refers to the height to width (i.e., length) ratio of the individual shear wall element. Size and shape of shear walls shall be limited to the aspect ratios in *SDPWS Table 4.3.3* (see Table 9.4 below).

- **Diagonal Brace A**

$D = 0$
$L_r = 0$
$Q_E = \pm 33$ kips
$E_h = 1.3Q_E = 1.3(\pm 33 \text{ kips}) = \pm 42.9$ kips
$E_v = 0.13D = 0.13(0) = 0$

Column B
$D = 50$ kips
$L_r = 12$ kips
$Q_E = \pm 17$ kips
$E_h = 1.3Q_E = 1.3(\pm 17 \text{ kips}) = \pm 22.1$ kips
$E_v = 0.13D = 0.13(50 \text{ kips}) = 6.5$ kips

A.) STRENGTH DESIGN (SD/LRFD) BASIC LOAD COMBINATIONS:

1. Maximum Axial Force in Brace A

By inspection - since there are no gravity load effects in the brace, eqn *ASCE 7 (2.3-1)* through *(2.3-5)* will not govern and only the basic load combinations with seismic load effects need to be calculated.

$1.2D + E_v + E_h + L + 0.2S$ *ASCE 7 (2.3-6)*
$1.2\,(0) + 0 + 42.9 \text{ kips} + 0 + 0.2\,(0) = \underline{+\,42.9 \text{ kips}}$ (compression)

$0.9D - E_v + E_h$ *ASCE 7 (2.3-7)*
$1.2\,(0) - 0 + (-\,42.9 \text{ kips}) = \underline{-\,42.9 \text{ kips}}$ (tension)

∴ Maximum SD/LRFD axial force, $P = \boxed{\pm\,42.9 \text{ kips}}$ compression or tension

2. Maximum Axial Force in Column B

By inspection - since there are only dead load (D) and roof live load (L_r) gravity load effects in the column, eqn *ASCE 7 (2.3-1) & (2.3-3)* will be calculated in addition to basic load combination with seismic load effects eqn *ASCE 7 (2.3-6)* to determine the maximum axial force in the column.

$1.4\,D$ *ASCE 7 (2.3-1)*
$1.4\,(50 \text{ kips}) = +\,70.0$ kips (compression)

$1.2\,D + 1.6\,(L_r \text{ or } S \text{ or } R) + 0.5\,(L \text{ or } 0.5\,W)$ *ASCE 7 (2.3-3)*
$1.2\,(50 \text{ kips}) + 1.6\,(12 \text{ kips}) + 0.5\,(0) = +\,79.2$ kips (compression)

$1.2\,D + E_v + E_h + L + 0.2\,S$ *ASCE 7 (2.3-6)*
$1.2\,(50) + 6.5 \text{ kips} + 22.1 \text{ kips} + 0 + 0.2\,(0) = \underline{+\,88.6 \text{ kips}}$ (compression)

∴ Maximum SD/LRFD axial force, $P = \boxed{+\,88.6 \text{ kips}}$ compression

3. Minimum Axial Force in Column B

By inspection - *ASCE 7 (2.3-1)* through *(2.3-5)* will not govern and the minimum axial force in the column will be governed by basic load combination with seismic load effects eqn *ASCE 7 (2.3-7)*.

$0.9\,D - E_v + E_h$ *ASCE 7 (2.3-7)*
$0.9\,(50) - 6.5 \text{ kips} + (-22.1 \text{ kips}) = \underline{+\,16.4 \text{ kips}}$ (compression)

∴ Minimum SD/LRFD axial force, $P = \boxed{+\,16.4 \text{ kips}}$ compression

> **<u>NOTE</u>:** Typically **<u>all</u>** SD basic load combination equations need to be checked (i.e., *ASCE 7 (2.3-1)* to *(2.3-7)*) for structural members resisting gravity loads <u>and</u> seismic forces because sometimes a gravity load combination equation(s) can govern over those with seismic load effects.

B.) ALLOWABLE STRESS DESIGN (ASD) BASIC LOAD COMBINATIONS:

1. Maximum Axial Force in Brace A

By inspection - since there are no gravity load effects in the brace, eqn *ASCE 7 (2.4-1)* through *(2.4-7)* will not govern and only the basic load combinations with seismic load effects need to be calculated.

$1.0\,D + 0.7\,E_v + 0.7\,E_h$ *ASCE 7 (2.4-8)*
$1.0\,(0) + 0.7\,(0) + 0.7\,(42.9 \text{ kips}) = \underline{+\,30.0 \text{ kips}}$ (compression)

$1.0\,D + 0.525\,E_v + 0.525\,E_h + 0.75\,L + 0.75\,S$ *ASCE 7 (2.4-9)*
$1.0\,(0) + 0.525\,(0) + 0.525\,(42.9 \text{ kips}) + 0.75\,(0) + 0.75\,(0) = +\,22.5 \text{ kips}$ (compression)

$0.6\,D - 0.7\,E_v + 0.7\,E_h$ *ASCE 7 (2.4-10)*
$0.6\,(0) - 0.7\,(0) + 0.7\,(-\,42.9 \text{ kips}) = \underline{-\,30.0 \text{ kips}}$ (tension)

$\therefore$ Maximum ASD axial force, $P = \boxed{\pm\,30.0 \text{ kips}}$ compression <u>or</u> tension

2. Maximum Axial Force in Column B

By inspection - since there are only dead load (D) and roof live load (L_r) gravity load effects in the column, eqn *ASCE 7 (2.4-3)* will be calculated in addition to basic load combination with seismic load effects eqn *ASCE 7 (2.4-8)* & *ASCE 7 (2.4-9)* to determine the <u>maximum</u> axial force in the column.

$D + (L_r \text{ or } S \text{ or } R)$ *ASCE 7 (2.4-3)*
$50 \text{ kips} + 12 \text{ kips} = +\,62.0 \text{ kips}$ (compression)

$1.0\,D + 0.7\,E_v + 0.7\,E_h$ *ASCE 7 (2.4-8)*
$1.0\,(50 \text{ kips}) + 0.7\,(6.5 \text{ kips}) + 0.7\,(22.1 \text{ kips}) = \underline{+\,70.0 \text{ kips}}$ (compression)

$1.0\,D + 0.525\,E_v + 0.525\,E_h + 0.75\,L + 0.75\,S$ *ASCE 7 (2.4-9)*
$1.0\,(50 \text{ kips}) + 0.525\,(6.5 \text{ kips}) + 0.525\,(22.1 \text{ kips}) + 0.75\,(0) + 0.75\,(0) = +\,65.0 \text{ kips}$ (compression)

$\therefore$ Maximum ASD axial force, $P = \boxed{+\,70.0 \text{ kips}}$ compression

3. Minimum Axial Force in Column B

By inspection - *ASCE 7 (2.4-1)* through *(2.4-7)* will not govern and the <u>minimum</u> axial force in the column will be governed by basic load combination with seismic load effects eqn *ASCE 7 (2.4-10)*.

$0.6\,D - 0.7\,E_v + 0.7\,E_h$ *ASCE 7 (2.4-10)*
$0.6\,(50 \text{ kips}) - 0.7\,(6.5 \text{ kips}) + 0.7\,(-\,22.1 \text{ kips}) = \underline{+\,10.0 \text{ kips}}$ (compression)

$\therefore$ Minimum ASD axial force, $P = \boxed{+\,10.0 \text{ kips}}$ compression

> **<u>NOTE</u>:** Typically **<u>all</u>** ASD basic load combination equations need to be checked (i.e., *ASCE 7 (2.4-1)* to *(2.4-10)*) for structural members resisting gravity loads <u>and</u> seismic forces because sometimes a gravity load combination equation(s) can govern over those with seismic load effects.

By inspection, the maximum drag force will occur on line 3 (i.e., 25′ > 10′) -

- Wall Line 1: roof $v_1 = 175$ plf

  max. $F_d = v_1 (10') = (175 \text{ plf})(10') = 1{,}750$ lbs (SD/LRFD force level)

- Wall Line 2: total "combined" roof $v_2 = 175 + 175 = 350$ plf, wall $v_2 = 350$ plf

  max. $F_d = 0$ lbs (SD/LRFD force level)

- Wall Line 3: roof $v_3 = 175$ plf

  max. $F_d = v_3 (25') = (175 \text{ plf})(25') = \boxed{4{,}375 \text{ lbs}}$ ← governs (SD/LRFD force level)

  From Part A.4 - drag force $F_d = 8{,}750$ lbs → $\boxed{\therefore \text{ 50\% reduction in maximum drag force}}$

NOTE: When a flexible diaphragm building consists of two perimeter lines of lateral resistance (i.e., shear walls on lines 1 & 3) and one interior line of lateral resistance (i.e., shear wall on line 2), the total shear to the interior line of lateral resistance will theoretically be equal to one-half of the base shear (i.e., $V_2 = V/2$) even when the interior line of resistance is not located at the center of the building plan (i.e., $L = L_1 + L_2$ and $L_1 \neq L_2$).

$$V_2 = w_s (L_1/2) + w_s (L_2/2)$$
$$= w_s (L_1 + L_2)/2 = w_s L/2 = V/2$$

3.27 Determine the *Site Class* adjusted maximum considered earthquake (MCE_R) spectral acceleration parameters (S_{MS} & S_{M1}) for the previous problem.

a. $S_{MS} = 1.20$ & $S_{M1} = 0.84$
b. $S_{MS} = 0.85$ & $S_{M1} = 0.66$
c. $S_{MS} = 0.70$ & $S_{M1} = 0.44$
d. $S_{MS} = 0.65$ & $S_{M1} = 0.30$

3.28 Given $S_S = 1.82$ & $S_1 = 0.68$, and a geotechnical report identifying the average soil properties of the upper 100 feet as a "very dense soil", determine the appropriate site coefficients F_a & F_v.

a. $F_a = 1.0$ & $F_v = 1.7$
b. $F_a = 1.2$ & $F_v = 1.7$
c. $F_a = 1.2$ & $F_v = 1.4$
d. Site-specific ground motion must be used per *ASCE 7-16 – §11.4.8*

3.29 Determine the *site class* adjusted maximum considered earthquake (MCE_R) spectral acceleration parameters (S_{MS} & S_{M1}) for the previous problem.

a. $S_{MS} = 2.18$ & $S_{M1} = 0.95$
b. $S_{MS} = 1.82$ & $S_{M1} = 1.74$
c. $S_{MS} = 1.82$ & $S_{M1} = 1.16$
d. Site-specific ground motion must be used per *ASCE 7-16 – §11.4.8*

3.30 Given a soil profile of "soft rock", $S_S = 0.40$ & $S_1 = 0.15$, determine the appropriate site coefficients F_a & F_v.

a. $F_a = 1.0$ & $F_v = 1.0$
b. $F_a = 1.3$ & $F_v = 1.5$
c. $F_a = 1.2$ & $F_v = 1.65$
d. $F_a = 1.48$ & $F_v = 2.3$

3.31 Determine the site adjusted maximum considered earthquake (MCE_R) spectral acceleration parameters (S_{MS} & S_{M1}) for the previous problem.

a. $S_{MS} = 0.59$ & $S_{M1} = 0.35$
b. $S_{MS} = 0.48$ & $S_{M1} = 0.25$
c. $S_{MS} = 0.52$ & $S_{M1} = 0.23$
d. $S_{MS} = 0.40$ & $S_{M1} = 0.15$

3.32 The <u>design</u> spectral response acceleration parameters (i.e., S_{DS} & S_{D1}) are a function of:

I. *Site Class*
II. *Risk Category*
III. Mapped MCE_R spectral accelerations parameters (S_S & S_1)

a. I
b. I & II
c. I & III
d. II & III

3.46 What would be the most appropriate MCE_R spectral response acceleration parameters (S_S & S_1) for a building project proposed at 35º00′00″ Latitude and -120º00′00″ Longitude?

a. $S_S = 1.50$ & $S_1 = 0.70$
b. $S_S = 1.25$ & $S_1 = 0.50$
c. $S_S = 1.00$ & $S_1 = 0.40$
d. $S_S = 0.75$ & $S_1 = 0.30$

3.47 MCE_R mapped spectral response acceleration parameters S_S & S_1 are determined based on which *site class*?

a. B
b. C
c. D
d. None of the above

3.48 Given $S_S = 0.63$ & $S_1 = 0.25$, with no soils report, what site coefficients F_a & F_v would be most appropriate per the *IBC?*

a. $F_a = 1.2$ & $F_v = 2.0$
b. $F_a = 1.3$ & $F_v = 2.1$
c. $F_a = 1.4$ & $F_v = 2.2$
d. Site-specific ground motion procedure is required to determine F_a & F_v

3.49 Given $S_S = 0.63$ & $S_1 = 0.25$, with no soils report, what design spectral response acceleration parameters would be most appropriate (i.e., S_{DS} & S_{D1})?

a. Site-specific ground motion procedure is required to determine S_{DS} & S_{D1}
b. $S_{DS} = 0.59$ & $S_{D1} = 0.37$
c. $S_{DS} = 0.55$ & $S_{D1} = 0.53$
d. $S_{DS} = 0.55$ & $S_{D1} = 0.35$

3.50 Given a 3-story State Emergency Operations Center (EOC) with $S_S = 1.92$, $S_1 = 0.80$, $S_{DS} = 1.29$ & $S_{D1} = 0.80$, what is the *Seismic Design Category*?

a. C
b. D
c. E
d. F

3.51 Which of the following occupancies types would <u>never</u> be assigned to *Seismic Design Category* E (*SDC* = E)?

a. Police station
b. Apartment building
c. County jail
d. Medical office building

3.52 Which of the following would be considered a *Risk Category* III structure?

a. 2-story Group E classroom building with an occupant load of 250
b. Multiplex cinema with 10 theaters each with an occupant load of 320
c. State University classroom building with an occupant load of 500
d. High-rise apartment building with an occupant load of 5,000

Problem	Answer	Reference / Solution
3.27	b	p. 1-33 & *2021 IBC* p. 16-41 - *§1613.2.3* $S_{MS} = F_a\, S_S$ *IBC (16-20)* $= 1.7\,(0.50) = 0.85$ $S_{M1} = F_v\, S_1$ *IBC (16-21)* $= 3.3\,(0.20) = 0.66$ $\therefore\ S_{MS} = \underline{0.85}$ & $S_{M1} = \underline{0.66}$ ←
3.28	c	1-33 & *2021 IBC* p. 16-42 - *Tables 1613.2.3(1)* & *1613.2.3(2)* "very dense soil" → *ASCE 7-16* p. 204 - *Table 20.3-1* → *Site Class* C *Site Class* C & $S_S = 1.82 > 1.5$ → *Table 1613.2.3(1)* → $F_a = 1.2$ *Site Class* C & $S_1 = 0.68 > 0.6$ → *Table 1613.2.3(2)* → $F_v = 1.4$ $\therefore\ F_a = \underline{1.2}$ & $F_v = \underline{1.4}$ ←
3.29	a	p. 1-33 & *2021 IBC* p. 16-41 - *§1613.2.3* $S_{MS} = F_a\, S_S$ *IBC (16-20)* $= 1.2\,(1.82) = 2.18$ $S_{M1} = F_v\, S_1$ *IBC (16-21)* $= 1.4\,(0.68) = 0.95$ $\therefore\ S_{MS} = \underline{2.18}$ & $S_{M1} = \underline{0.95}$ ←
3.30	b	p. 1-33 & *2021 IBC* p. 16-42 - *Tables 1613.2.3(1)* & *1613.2.3(2)* "soft rock" → *ASCE 7-16* p. 204 - *Table 20.3-1* → *Site Class* C *Site Class* C & $S_S = 0.40$ → *Table 1613.2.3(1)* → $F_a = 1.3$ *Site Class* C & $S_1 = 0.15$ → *Table 1613.2.3(2)* → $F_v = 1.5$ $\therefore\ F_a = \underline{1.3}$ & $F_v = \underline{1.5}$ ←
3.31	c	p. 1-33 & *2021 IBC* p. 16-41 - *§1613.2.3* $S_{MS} = F_a\, S_S$ *IBC (16-20)* $= 1.3\,(0.40) = 0.52$ $S_{M1} = F_v\, S_1$ *IBC (16-21)* $= 1.5\,(0.15) = 0.23$ $\therefore\ S_{MS} = \underline{0.52}$ & $S_{M1} = \underline{0.23}$ ←
3.32	c	p. 1-34 & *2021 IBC* p. 16-41 - *§1613.2.4* S_{DS} & S_{D1} are equal to 2/3 of the S_{MS} & S_{M1} respectively, which means they all are a function of the the *Site Class* (i.e., F_a & F_v) <u>and</u> mapped MCE_R spectral acceleration parameters S_S & S_1 ∴ <u>I & III</u> ←
3.33	b	p. 1-34 & *2021 IBC* p. 16-41 - *§1613.2.4* $S_{DS} = 2/3\ S_{MS}$ *IBC (16-22)* $= 2/3\,(0.90) = 0.60$ $S_{D1} = 2/3\ S_{M1}$ *IBC (16-23)* $= 2/3\,(0.74) = 0.49$ $\therefore\ S_{DS} = \underline{0.60}$ & $S_{D1} = \underline{0.49}$ ←

Problem	Answer	Reference / Solution
3.42	c	p. 1-37 - Table 3.3 *Seismic Design Category* C = Moderate seismic hazard level ←
3.43	d	p. 1-37 & *2021 IBC* p. 16-41 - *§1613.2.5* a. Hospital (Group I-2, Condition 2) → *IBC Table 1604.5* → *RC* = IV b. Single-family residence (Group R-3) → *IBC Table 1604.5* → *RC* = II c. County jail (Group I-3) → *IBC Table 1604.5* → *RC* = III *Seismic Design Category* F applies only to *Risk Category* IV structures (i.e., essential facilities, etc.) *SDC* = F does not apply to *Risk Category* I, II or III structures … ∴ Both b & c ←
3.44	d	*2021 IBC* p. 16-4 & 5 - *§1604.5.1* & *Table 1604.5* Where a building or structure is occupied by two or more occupancies not included in the same *Risk Category,* it shall be assigned the classification of the highest *Risk Category* corresponding to the various occupancies. Office building → *IBC Table 1604.5* → *RC* = II Fire station → *IBC Table 1604.5* → *RC* = IV ← governs ∴ use *Risk Category* IV ←
3.45	c	*2021 IBC* p. 16-44 & 46 - *Figures 1613.2.1(1)* & *1613.2.1(3)* At 45º00′00″ Latitude and -120º00′00″ Longitude … *Figure 1613.2.1(1)* → $S_S \approx 36\% = 0.36$ *Figure 1613.2.1(3)* → $S_1 \approx 15\% = 0.15$ ∴ $S_S = 0.36$ & $S_1 = 0.15$ ←
3.46	c	*2021 IBC* p. 16-44 & 46 - *Figures 1613.2.1(1)* & *1613.2.1(3)* At 35º00′00″ Latitude and -120º00′00″ Longitude … *Figure 1613.2.1(1)* → $S_S \approx 100\% = 1.00$ *Figure 1613.2.1(3)* → $S_1 \approx 40\% = 0.40$ ∴ $S_S = 1.00$ & $S_1 = 0.40$ ←
3.47	d	*2021 IBC* p. 16-44 & 46 - *Figures 1613.2.1(1)* & *1613.2.1(3)* Unlike the *2015 IBC* and *ASCE 7-10* where the S_S & S_1 maps were based on a *Site Class* B soil profile, the S_S & S_1 maps for the *2021 IBC* and *ASCE 7-16* are based on a soil profile interface between *Site Class* B and *Site Class* C (i.e., average shear wave velocity = 2,500 ft/sec) ∴ None of the above ←
3.48	b	1-33 - NOTE & *2021 IBC* p. 16-41 - *Table 1613.2.3(1)* & *Table 1613.2.3(2)* No soils report → assume *Site Class* D (default) Using linear interpolation … for intermediate values of S_S and S_1 per Note *Site Class* D & $S_S = 0.63$ → *Table 1613.2.3(1)* → $F_a = 1.3$ *Site Class* D & $S_1 = 0.25$ → *Table 1613.2.3(2)* → $F_v = 2.1$ ∴ $F_a = 1.3$ & $F_v = 2.1$ ←

Problem	Answer	Reference / Solution
3.49	c	1-33 to 34 & *2021 IBC* p. 16-41 & 42 - *§1613.2.3, §1613.3.4, Table 1613.2.3(1)* & *Table 1613.2.3(2)*, and *ASCE 7-16 Supplement 3* to *§11.4.8* From Problem 3.48 – $F_a = 1.3$ & $F_v = 2.1$ $S_{MS} = F_a\, S_S$ *IBC (16-20)* $= 1.3\,(0.63) = 0.82$ ***NOTE:** per Exception for *Site Class* D sites w/ $S_1 \geq 0.2$ … a GMHA is not required where S_{M1} is increased by 50% for all applications of S_{M1}, etc. $S_{M1} = {}^{*}1.5\, F_v\, S_1$ *IBC (16-21)* $= 1.5\,(2.1)(0.25) = 0.79$ $S_{DS} = 2/3\ S_{MS}$ *IBC (16-22)* $= 2/3\,(0.82) = \underline{0.55}$ $S_{D1} = 2/3\ S_{M1}$ *IBC (16-23)* $= 2/3\,(0.79) = \underline{0.53}$ $\therefore\ S_{DS} = \underline{0.55}$ & $S_{D1} = \underline{0.53}$ ←
3.50	d	1-37 - Table 3.4 & *2021 IBC* p. 16-41 - *§1613.2.5* Emergency Operations Center (EOC) → *IBC Table 1604.5* → *RC* = IV $S_1 = 0.80 > 0.75$ → for *Risk Category* IV → *Seismic Design Category* F ∴ *SDC* = F ←
3.51	a	p. 1-37 - Table 3.4 & *2021 IBC* p. 16-41 - *§1613.2.5* Police station → *IBC Table 1604.5* → *RC* = IV Apartment building (Group R-2) → *IBC Table 1604.5* → *RC* = II County jail (Group I-3) → *IBC Table 1604.5* → *RC* = III Medical office building (Group B) → *IBC Table 1604.5* → *RC* = II *Seismic Design Category* E applies only to *Risk Category* I, II or III structures … *SDC* = E does not apply to *Risk Category* IV structures ∴ Police station ←
3.52	b	p. 1-28 & *2021 IBC* p. 16-5 - *Table 1604.5* Group E classroom bldg. w/ occupant load (OL) = 250 ≯ 250 → *RC* = II Multiplex cinema bldg. OL = 10 (300) = 3,200 > 2,500 → *RC* = III State University classroom bldg. w/ OL = 500 ≯ 500 → *RC* = II High-rise apartment bldg. w/ OL = 5,000 ≯ 5,000 → *RC* = II ∴ Multiplex cinema w/ 10 theaters each with an occupant load of 320 ←
3.53	d	p. 1-31 - Mapped MCE_R Acceleration Parameters, S_S & S_1 The MCE_R ground motions are expected to result in structures with a 1% probability of collapse in 50 years, which is assumed to be an acceptable level of seismic safety. ∴ 1% probability of collapse in 50 years ←
3.54	c	p. 1-28 & *2021 IBC* p. 3-9 - *§308.4* Group I-3 occupancies include correctional centers, detention centers, jails, prerelease centers, prisons, reformatories, etc. ∴ Group I-3 ←

Problem	Answer	Reference / Solution
7.6	b	Horizontal force per brace: H = 6.0 kips / 2 frames = 3.0 kips per frame (horizontal component in brace) Length of brace, $L_b = \sqrt{(8')^2 + (6')^2} = 10'$ By similar triangles (or trigonometry), the resultant axial force in brace: $Q_E = R$ = (3.0 kips)(10′) / (8′) = 3.8 kips per brace ←
7.7	b	Dead load effect: Tank is supported by 4 columns D = (20 kips) / 4 columns = 5 kips per column ←
7.8	b	p. 1-80 - Vertical Seismic Load Effect, E_v & *ASCE 7-16* p. 99 - *§12.4.2.2* $E_v = 0.2 S_{DS} D$ *ASCE 7 (12.4-4a)* E_v = 0.2(0.60)(5 kips) = ± 0.6 kips per column ←
7.9	d	p. 1-28 to 29 - Risk Category & *2021 IBC* p. 16-5 - *Table 1604.5* Power-generating stations ... required as emergency backup facilities for *Risk Category* IV structures (e.g., EOC) – *Risk Category* IV ←
7.10	b	1-37 to 38 & *2021 IBC* p. 16-41 - *§1613.2.5* & *Tables 1613.2.5(1)* & *(2)* S_1 < 0.75 … therefore, determine *SDC* from *Tables 1613.2.5(1)* & *(2)* only S_{DS} = 0.60 & *RC* = IV → *Table 1613.2.5(1)* → *SDC* = D S_{D1} = 0.24 & *RC* = IV → *Table 1613.2.5(2)* → *SDC* = D ∴ *SDC* = D ←
7.11	a	p. 1-39 - Seismic Importance Factor, I_e & *ASCE 7-16* p. 5 - *Table 1.5-2* *Risk Category* IV – *2021 IBC Table 1604.5* ∴ I_e = 1.5 ←
7.12	b	p. 1-106 to 107 & *ASCE 7-16* p. 147 - *Table 15.4-1* – Nonbuilding Structures Similar to Buildings Steel ordinary concentrically braced frames (steel OCBF) – R = 3¼ ←
7.13	b	p. 1-106 - Rigid Nonbuilding Structures & *ASCE 7-16* p. 149 - *§15.4.2* T = 0.05 second < 0.06 second → Rigid nonbuilding structure $V = 0.30 S_{DS} W I_e$ *ASCE 7 (15.4-5)* = 0.30(0.60)(20 kips)(1.5) = 5.4 kips ←
7.14	d	p. 1-110 to 111 - Tanks and Vessels & *ASCE 7-16* p. 153 - *§15.7.1* This section applies to all tanks, vessels, bins and silos, and similar containers storing liquids, gases, and granular solids supported at the base. ∴ I, II & III ←
7.15	d	p. 1-106 to 107 & *ASCE 7-16* p. 150 - *§15.5* Nonbuildings structures similar to buildings include pipe racks, steel storage racks, electrical power generating facilities, structural towers for tanks and vessels, and piers and wharves. ∴ All of the above ←

Problem	Answer	Reference / Solution
		δ_{max} = 0.75" > 1.4 δ_{avg} = 0.70" → *Extreme* Torsional Irregularity exists ∴ <u>II</u> ←
8.34	c	p. 1-128 & *ASCE 7-16* p. 103 - *§12.8.4.3* - Amplification of Accidental Torsional Moment N-S direction: accidental e_x = ± 5% (200′) = ± 10′ $M_{ta} = V_y$ (± 0.05 L_x) = 200 kips (± 10′) = ± 2,000 kip-ft δ_{avg} = (0.25" + 0.75") / 2 = 0.50" $A_x = (\delta_{max} / 1.2\ \delta_{avg})^2$ = [(0.75") / 1.2(0.50")]^2 = 1.56 Amplified $M_{ta} = A_x M_{ta}$ = 1.56 (± 2,000 kip-ft) = ± <u>3,120 kip-ft</u> ←
8.35	b	p. 1-58 to 59 - Effective Seismic Weight of a Level, w_x The effective seismic weight of a level includes the dead load weight of that level (plus applicable loads from *ASCE 7-16 – §12.7.2*) and the tributary weight from ALL <u>perimeter</u> exterior walls. In the case of a 1-story building, that would include the weight of the roof plus the <u>upper half</u> of ALL <u>perimeter</u> exterior wall weights: ∴ N-S load direction: $W_1 = \underline{½W_A + ½W_B + ½W_C + ½W_D + W_{roof}}$ ←
8.36	b	p. 1-58 to 59 - Effective Seismic Weight of a Level - w_x The effective seismic weight of a level includes the dead load weight of that level (plus applicable loads from *ASCE 7-16 – §12.7.2*) and the tributary weight from all <u>perimeter</u> exterior walls. In the case of a 1-story building, that would include the weight of the roof plus the <u>upper half</u> of the <u>perimeter</u> exterior wall weights: ∴ E-W load direction: $W_1 = \underline{½W_A + ½W_B + ½W_C + ½W_D + W_{roof}}$ ←
8.37	c	p. 1-113 to 114 - Diaphragm Design Force, F_{px} w_{px} = weight of the diaphragm and the elements <u>tributary</u> there to at Level x … and <u>need not include</u> the wall (or exterior panel) weights <u>parallel</u> to the direction of force that are in line with the vertical seismic force-resisting elements (e.g., shear walls, braced frames, moment frames): ∴ N-S load direction: $w_{p1} = \underline{½W_A + ½W_B + W_{roof}}$ ←
8.38	d	p. 1-113 to 114 - Diaphragm Design Force, F_{px} w_{px} = weight of the diaphragm and the elements <u>tributary</u> there to at Level x … and <u>need not include</u> the wall (or exterior panel) weights <u>parallel</u> to the direction of force that are in line with the vertical seismic force-resisting elements (e.g., shear walls, braced frames, moment frames): ∴ E-W Load direction: $w_{p1} = \underline{½W_C + ½W_D + W_{roof}}$ ←
8.39	b	p. 1-74 to 75 - Seismic Base Shear, V & *ASCE 7-16* p. 118 - *§12.14.8.1* $V = \frac{FS_{DS}}{R}W$ *ASCE 7 (12.14-12)* One-story building – F = 1.0 ∴ $V = (1.0\ S_{DS} / R)\ W$ ←

Problem	Answer	Reference / Solution
9.8	b	p. 1-122 - Drag Force Maximum drag force occurs on right (i.e., East) wall line at 20′ from South end of collector (i.e., South end of 30′ shear wall) – From Problem 9.7, ASD roof $v = 240$ plf for ASD, max F_d = roof v(20′) = (240 plf)(20′) = 4,800 lbf = 4.8 kips ←
9.9	b	p. 1-155 to 158 - Shear Wall Overturning / Hold-Downs $\rho = 1.0$ (given) $V_1 = V_2 = V/2 = 33.6 \text{ kips}/2 = 16.8 \text{ kips}$ $T = \dfrac{-0.7\rho(V_1\,h)}{b} = \dfrac{-0.7(1.0)(16.8)(10')}{30'} = -3.92 \text{ kips}$ ∴ for ASD, uplift T = 4.0 kips ←
9.10	b	p. 1-144 - Wood Structural Panel Diaphragms $V = C_S W = 0.196\,W$ (given) For a single-story building – $w_s = f_{p1} = F_{p1}/L = C_S w_{p1}$ E-W direction: w_s = 0.196 [(25 psf)(75′) + (15 psf)(12′/2) 4 walls] = 438 plf $V_{max} = w_s L/2$ = (438 plf)(40′) / 2 = 8,760 lbf for ASD, roof $v = (0.7\,V_{max})/d$ = 0.7 (8,760 lbf) / 75′ = 82 plf ≈ 80 plf ←
9.11	c	p. 1-152 - Table 9.5 & *SDPWS-2021* p. 40 - *Table 4.3A* 3/8″ rated sheathing w/ 8d common @ 2″ o.c. → v_n = 1485 plf 15/32″ Structural I w/ 10d common @ 6″ o.c. → v_n = 950 plf **NG!** 15/32″ Structural I w/ 10d common @ 4″ o.c. → v_n = 1430 plf **NG!** 15/32″ Structural I w/ 10d common @ 3″ o.c. → v_n = 1860 plf **OK** 15/32″ Structural I w/ 10d common @ 2″ o.c. → v_n = 2435 plf **OK** ∴ use 15/32″ Structural I w/ 10d common @ 3″ o.c. = 1860 plf > 1485 plf ←
9.12	d	p. 1-145/146 - Table 9.2 & 9.3, *SDPWS-2021* p. 25/26 - *Table 4.2A* & *4.2C* Load parallel to continuous panel joints = CASE 3 (weak direction) 15/32″ sheathing w/ 8d @ 6″ o.c. unblocked → Table 9.3 (*4.2C*) → v_n = 505 plf → ASD $v_s = v_n/2.8$ = (505 plf) / 2.8 = 180 plf < 275 plf **NG!** 15/32″ sheathing w/ 10d @ 6″ o.c. unblocked → Table 9.3 (*4.2C*) → v_n = 530 plf → ASD $v_s = v_n/2.8$ = (530 plf) / 2.8 = 189 plf < 275 plf **NG!** 15/32″ sheathing w/ 8d @ 6″ o.c. blocked → Table 9.2 (*4.2A*) → v_n = 755 plf → ASD $v_s = v_n/2.8$ = (755 plf) / 2.8 = 270 plf < 275 plf **NG!** 15/32″ sheathing w/ 10d @ 6″ o.c. blocked → Table 9.2 (*4.2A*) → v_n = 810 plf → ASD $v_s = v_n/2.8$ = (810 plf) / 2.8 = 289 plf > 275 plf **OK** ∴ use 15/32″ sheathing w/ 10d common @ 6″ o.c. ←
9.13	d	p. 1-118 - Flexible Diaphragm Analysis $V = C_S W$ and $C_S = 0.20$ (given) For a single-story building – $w_s = f_{p1} = F_{p1}/L = C_S w_{p1}$ *(continued)*