The United States Geologic Survey (USGS) uses the following to represent earthquake strength:

Table 1.1 – USGS Earthquake Strength

Earthquake	Magnitude, *M*
Micro	< 3
Minor	3 – 3.9
Light	4 – 4.9
Moderate	5 – 5.9
Strong	6 – 6.9
Major	7 – 7.9
Great	> 8

Energy Release, *E*

In 1956, Gutenberg and Richter developed the following equation to approximate the earthquake energy radiated, E (in ergs), as a function of earthquake magnitude, M:

$$\text{Log10}E = 11.8 + 1.5M$$

Based on this equation, each whole number increase in Richter magnitude would represent an approximate 32-fold increase in the amount of energy radiated.

A *M*7.0 earthquake would radiate approximately 1,000 times more energy than a *M*5.0 earthquake. Theoretically, it would take approximately 1,000 *M*5.0 earthquakes to release the amount of energy of a single *M*7.0 earthquake.

1.6 Earthquake Intensity

Modified Mercalli Intensity Scale, *MMI*

The intensity of an earthquake is based on the damage to structures, damage to the ground surface, and observed effects on people and other features. Intensity is directly related to an earthquake's local ground accelerations and how long they persist (i.e., duration of strong ground motion).

The Modified Mercalli Intensity Scale is subject to human interpretation, and may be skewed if the affected structures are of unusually good versus unusually poor construction.

The Modified Mercalli Intensity Scale is a measure of the amount of shaking (and damage) at a particular site. The intensity of an earthquake will vary depending on where the site is relative to the epicenter. Intensity generally decreases with increasing distance from the epicenter, unless soil conditions (e.g., soft soil) amplify the motion.

Because earthquake intensity assessments do not depend on instruments, but on the actual observation of effects in a particular area, intensities can be assigned even to historical earthquakes.

Modified Mercalli intensities are represented with Roman numerals from I to XII. The lower numbers (*MMI* I-VI) of the intensity scale are based on the manner in which the earthquake is felt by people. The higher numbers (*MMI* VII-XII) are based on observed structural damage.

Table 3.2 - Design Spectral Response Acceleration Parameter at Short Periods (S_{DS})

S_S	Site Class				
	A	B	C	D*	E	F
0.05	0.03	0.03	0.04	0.05	0.08	*Site-Specific Ground Motion Procedure Required - ASCE 7 - Chapter 21*
0.10	0.05	0.07	0.08	0.11	0.17	
0.15	0.08	0.10	0.12	0.16	0.25	
0.20	0.11	0.13	0.16	0.21	0.33	
0.25	**0.13**	**0.17**	**0.20**	**0.27**	**0.42**	
0.30	0.16	0.20	0.24	0.31	0.47	
0.35	0.19	0.23	0.28	0.35	0.51	
0.40	0.21	0.27	0.32	0.39	0.54	
0.45	0.24	0.30	0.36	0.43	0.56	
0.50	**0.27**	**0.33**	**0.40**	**0.47**	**0.57**	
0.55	0.29	0.37	0.43	0.50	0.59	
0.60	0.32	0.40	0.46	0.53	0.60	
0.65	0.35	0.43	0.49	0.55	0.61	
0.70	0.37	0.47	0.52	0.58	0.61	
0.75	**0.40**	**0.50**	**0.55**	**0.60**	**0.60**	
0.80	0.43	0.53	0.58	0.63	0.61	
0.85	0.45	0.57	0.60	0.66	0.61	
0.90	0.48	0.60	0.62	0.68	0.61	
0.95	0.51	0.63	0.65	0.71	0.61	
1.00	**0.53**	**0.67**	**0.67**	**0.73**	**0.60**	
1.05	0.56	0.70	0.70	0.76	0.63	
1.10	0.59	0.73	0.73	0.78	0.66	
1.15	0.61	0.77	0.77	0.80	0.69	
1.20	0.64	0.80	0.80	0.82	0.72	
1.25	**0.67**	**0.83**	**0.83**	**0.83**	**0.75**	
1.30	0.69	0.87	0.87	0.87	0.78	
1.35	0.72	0.90	0.90	0.90	0.81	
1.40	0.75	0.93	0.93	0.93	0.84	
1.45	0.77	0.97	0.97	0.97	0.87	
1.50	**0.80**	**1.00**	**1.00**	**1.00**	**0.90**	
1.55	0.83	1.03	1.03	1.03	0.93	
1.60	0.85	1.07	1.07	1.07	0.96	
1.65	0.88	1.10	1.10	1.10	0.99	
1.70	0.91	1.13	1.13	1.13	1.02	
1.75	**0.93**	**1.17**	**1.17**	**1.17**	**1.05**	
1.80	0.96	1.20	1.20	1.20	1.08	
1.85	0.99	1.23	1.23	1.23	1.11	
1.90	1.01	1.27	1.27	1.27	1.14	
2.00	**1.07**	**1.33**	**1.33**	**1.33**	**1.20**	
2.10	1.12	1.40	1.40	1.40	1.26	
2.20	1.17	1.47	1.47	1.47	1.32	
2.30	1.23	1.53	1.53	1.53	1.38	
2.40	1.28	1.60	1.60	1.60	1.44	
2.50	**1.33**	**1.67**	**1.67**	**1.67**	**1.50**	
2.60	1.39	1.73	1.73	1.73	1.56	
2.70	1.44	1.80	1.80	1.80	1.62	
2.80	1.49	1.87	1.87	1.87	1.68	
2.90	1.55	1.93	1.93	1.93	1.74	
3.00	**1.60**	**2.00**	**2.00**	**2.00**	**1.80**	

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.

*Occupancy Category III and IV structures assigned to SDC = D, E or F will require *Structural Observation* per *IBC §1710.*

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 §1613.5.1*, **OR**
2. $S_{DS} < 0.167$ and $S_{D1} < 0.067$ … per *IBC Tables 1613.5.6(1) & 1613.5.6(2)*

Structures assigned to SDC = A need only comply with the requirements of *ASCE 7 – §11.7* (i.e., not *ASCE 7 – Chapter 12*).

Lateral Forces *ASCE 7 – §11.7.2*

Each structure shall be analyzed for the effects of static lateral forces applied independently in each of two orthogonal directions.

- **Base Shear, *V***

The seismic base shear, in each direction, shall be determined in accordance with the following:

$$V = 0.01 \cdot W$$

where:
W = the total dead load of the structure (D)

- **Vertical Distribution of Lateral Force, F_x**

In each direction, the static lateral forces at all levels shall be applied simultaneously. The force at each level shall be determined as follows:

$$F_x = 0.01 \cdot w_x \qquad \text{ASCE 7 (11.7-1)}$$

where:
F_x = the design lateral force applied at Level x, and
w_x = the portion of the total dead load of the structure (D) located or assigned to Level x

- **Diaphragm Design Force, F_{px}**

Floor and roof diaphragms shall be designed to resist design seismic forces in accordance with the following:

$$F_{px} = 0.01 \cdot w_{px}$$

where:
w_{px} = dead load weight of the diaphragm and the elements tributary there to at Level x

NOTE: See Chapter 8 (p. 1-95 to 1-97) – Diaphragm Design.

Load Path Connections *ASCE 7 – §11.7.3*

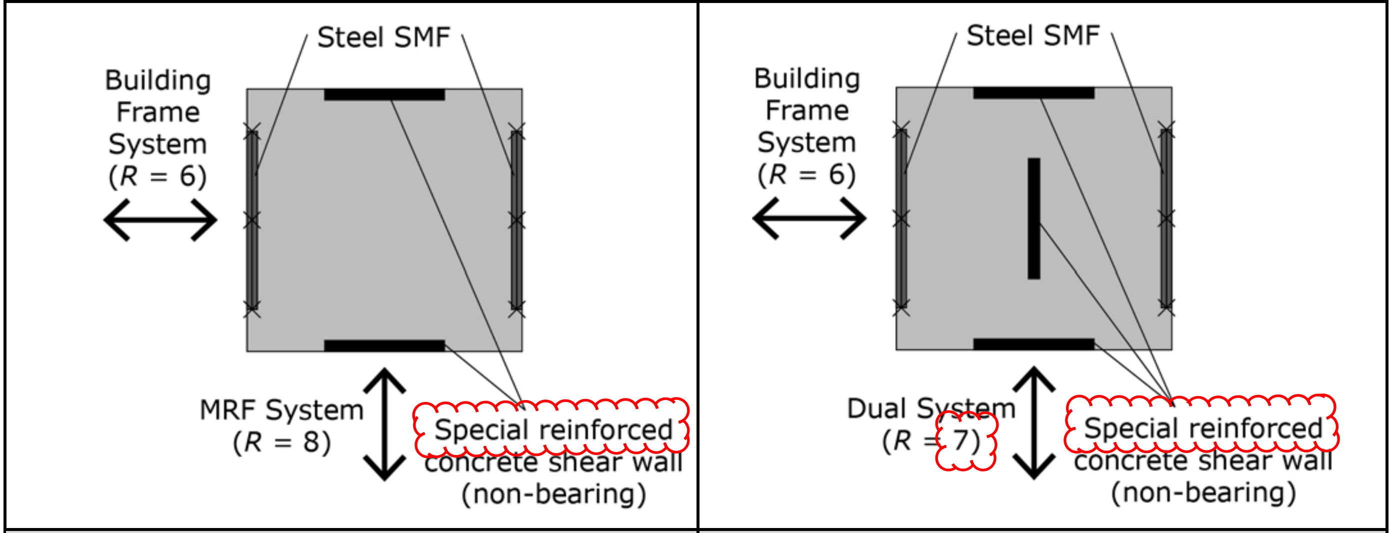
All parts of the structure … shall be interconnected to form a continuous load path to the seismic-force-resisting system (SFRS).

- ***Moment-resisting frames*** (e.g., SMF, STMF, IMF, OMF) provide resistance to lateral loads primarily by flexural (bending) action of members (e.g., beams, columns).

D. Dual System - Structural system that is essentially a combination of a Building Frame System (e.g., shear walls, CBF's, EBF's) and a Moment-Resisting Frame System (e.g., SMF's or IMF's) oriented to resist lateral loads in the same direction.

> Per *ASCE 7 – §12.2.5.1*, the total seismic force resistance is to be provided by the combination of the *moment-resisting frames* and the *shear walls* (or *braced frames*) in proportion to their rigidities.
>
> Furthermore, the *moment-resisting frames* shall be designed to independently resist at least 25 percent of the design seismic forces.

Figure 4.2 – Dual System Example



Building A - Not a Dual System

Building B – Dual System (N-S axis)

E. Shear Wall-Frame Interactive System - a structural system that uses combinations of ordinary reinforced concrete *shear walls* and ordinary reinforced concrete moment frames (OMF's). Per *ASCE 7 – Table 12.2-1*.

This type of system is not permitted (i.e., NP) in Seismic Design Categories C, D, E or F.

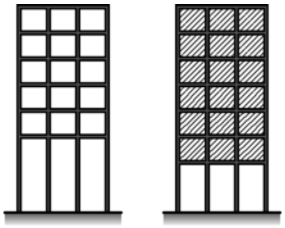
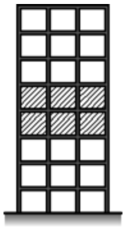
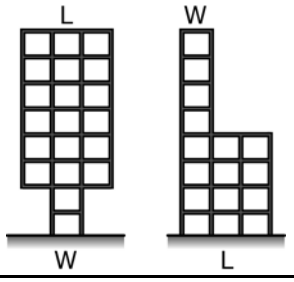
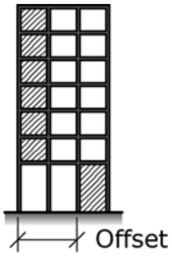
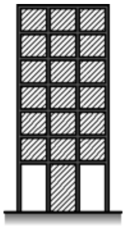
F. Cantilevered Column System - Structural system relying on cantilever column elements for lateral resistance – see Figure 4.3.

G. Steel Systems Not Specifically Detailed for Seismic Resistance - excluding cantilever column systems - per *ASCE 7 – Table 12.2-1*.

This type of system is not permitted (i.e., NP) in Seismic Design Categories D, E or F.

> **NOTE:** The structural framing system shall also comply with the system specific requirements found in *ASCE 7 – §12.2.5* (i.e., *ASCE 7 – §12.2.5.1* through *ASCE 7 – §12.2.5.10*).

Table 4.2 – Vertical Structural Irregularities (Ref. 12 / *ASCE 7 – Table 12.3-2*)

Irregularity Type & Description	Example
1a. Stiffness - Soft Story Irregularity Defined to exist when the story lateral stiffness is < 70% of that in the story above **or** < 80% of the average story stiffness of the three stories above. ✓ *Applies to **SDC = D, E & F*** **1b. Stiffness - Extreme Soft Story Irregularity** Defined to exist when the story lateral stiffness is < 60% of that in the story above **or** < 70% of the average story stiffness of the three stories above. ✓ *Applies to **SDC = D, E & F***	
2. Weight (Mass) Irregularity Defined to exist where the effective mass of any *level* is > 150% of the effective mass of an adjacent *level*. A roof that is lighter than the floor below need not be considered. ✓ *Applies to **SDC = D, E & F***	
3. Vertical Geometric Irregularity Defined to exist where the horizontal dimension of the seismic-force-resisting system in any story is > 130% of that in an adjacent story. ✓ *Applies to **SDC = D, E & F***	
4. In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity Defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exist a reduction in stiffness of the resisting element in the story below. ✓ *Applies to **SDC = B, C, D, E & F***	
5a. Discontinuity in Lateral Strength - Weak Story Irregularity Defined to exist when the story lateral strength is < 80% of that in a story above. The story lateral strength is the total strength of all seismic-force-resisting elements sharing the story shear for the direction under consideration. ✓ *Applies to **SDC = D, E & F*** **5b. Discontinuity in Lateral Strength - Extreme Weak Story Irregularity** Defined to exist when the story lateral strength is < 65% of that in a story above. ✓ *Applies to **SDC = B, C, D, E & F***	

4.11 Vertical Distribution of Seismic Forces

Vertical Distribution of Seismic Forces, F_x *ASCE 7 – §12.8.3*

The lateral seismic force (F_x) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} \cdot V \qquad \textit{ASCE 7 (12.8-11)}$$

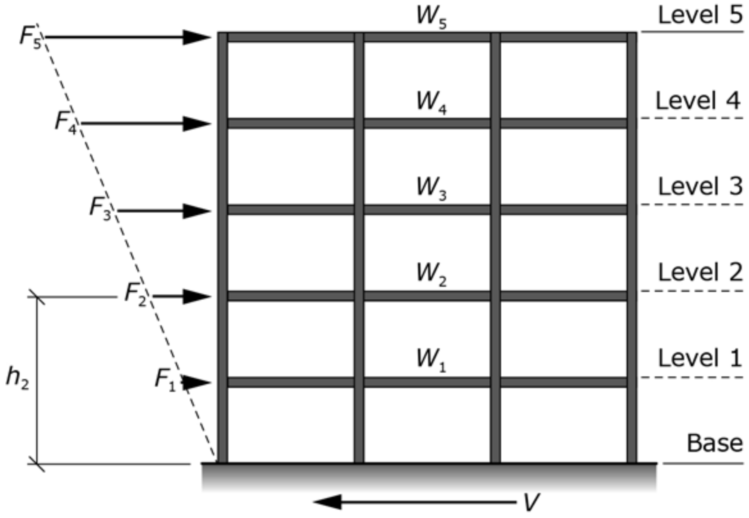
and

$$C_{vx} = \frac{w_x \cdot h_x^k}{\sum_{i=1}^{n} w_i \cdot h_i^k} \qquad \textit{ASCE 7 (12.8-12)}$$

where:

C_{vx} = vertical distribution factor
V = seismic base shear
$k = 1 \rightarrow$ for $T \leq 0.5$ second
$k = 2 \rightarrow$ for $T \geq 2.5$ seconds
$= 2 \rightarrow$ 0.5 second $< T < 2.5$ seconds … **<u>or</u>** determine k by <u>linear interpolation</u> (1 to 2) using the following equation:
$k = 0.75 + 0.5 \cdot T$

Figure 4.13 – Vertical Distribution of Force



- $\boxed{T \leq 0.5 \text{ second}}$ – $C_{vx} = \dfrac{w_x \cdot h_x}{\sum_{i=1}^{n} w_i \cdot h_i}$

- $\boxed{T \geq 2.5 \text{ seconds}}$ – $C_{vx} = \dfrac{w_x \cdot h_x^2}{\sum_{i=1}^{n} w_i \cdot h_i^2}$

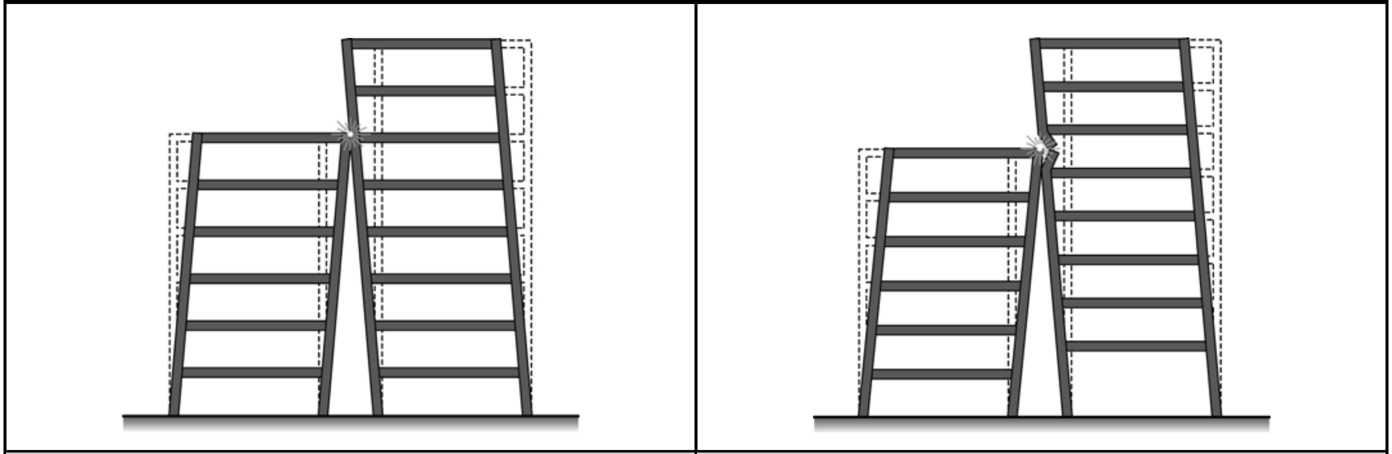
The force F_x shall be applied, at each level x, over the area of the building in accordance with the mass distribution at that level (i.e., center of mass). Structural displacements and design seismic forces shall be calculated as the effects of F_x forces applied at the appropriate levels above the base.

4.14 Building Separation *ASCE 7 – §12.12.3*

When inadequate separation is provided between structures, *pounding* may occur during an earthquake, which is the repeated collision of adjacent structures.

Pounding can be particularly dangerous with regard to adjacent buildings of differing heights and floor levels that do not align vertically. The floor of one building may collide and severely damage the vertical-load carrying columns (or walls) of the other building. The upper portion of the taller building may even collapse on top of the shorter building and propagate to the partial or total collapse of the shorter building.

Figure 4.18 – Examples of Pounding



Floors Aligned **Floors Not Aligned**

IBC 1613.6.7 modifies *ASCE 7 – §12.12.3* to provide the requirements for the minimum distance for building separation.

Separations shall allow for the maximum inelastic response displacement (δ_M), which shall be determined at critical locations (considering both translational and torsional displacements) using the following equation:

$$\delta_M = \frac{C_d \cdot \delta_{\max}}{I}$$ *IBC (16-44)*

where:

C_d = the deflection amplification factor per *ASCE 7 – Table 12.2-1*
δ_{max} = Maximum displacement defined in *ASCE 7 – §12.8.4.3* (i.e., due to F_x forces)
I = Importance Factor per *ASCE 7 – §11.5.1*

Adjacent Buildings on the Same Property, δ_{MT}

Adjacent buildings on the same property shall be separated by a distance not less than δ_{MT} using the following equation:

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}$$ *IBC (16-45)*

where:

δ_{M1} = Maximum inelastic displacement of adjacent Structure 1
δ_{M2} = Maximum inelastic displacement of adjacent Structure 2

Horizontal Seismic Load Effect with Overstrength Factor, E_{mh} *ASCE 7 – §12.4.3.1*

The horizontal seismic load effect with overstrength factor (E_{mh}) shall be determined in accordance with the following:

➢ $E_{mh} = \pm\Omega_0 \cdot Q_E$ *ASCE 7 (12.4-7)*

where:

Q_E = effects of horizontal seismic forces from the seismic base shear V (per *ASCE 7 – §12.8.1*) or the seismic lateral force F_p (per *ASCE 7 – §13.3.1*). See *ASCE 7 – §12.5.3* & *ASCE 7 – §12.5.4* for consideration of orthogonal effects)

Ω_0 = Overstrength Factor … per *ASCE 7 – Table 12.2-1* ~~*or 12.14-1*~~

Exception: E_{mh} need not exceed the maximum force that can develop in the element as determined by … see *ASCE 7 – §12.4.3.1*

5.2 Load Combinations *IBC §1605*

General *IBC §1605.1*

Buildings (and other structures) and portions thereof shall be designed to resist the load combinations specified in:

- *IBC §1605.2* (Strength Design or Load & Resistance Factor Design – SD/LRFD) **or**
- *IBC §1605.3* (Allowable Stress Design – ASD), **and**
- *IBC Chapters 18* through *23*, **and**
- The load combinations with overstrength factor (Ω_0) specified in *ASCE 7 – §12.3.4.2* where require by *ASCE 7 – §12.2.5.2*, *§12.3.3.3* and/or *ASCE 7 – §12.10.2.1*

NOTE: When using the Simplified Procedure of *ASCE 7 – §12.14*, the load combinations with overstrength factor of *ASCE 7 – §12.14.3.2* shall be used (i.e., $\Omega_0 = 2.5$ assumed).

Load combinations are a way of considering the maximum (or minimum) forces on a structural element using principles of superposition.

The load combinations consider combined effects of gravity loads (e.g., dead load, floor live load, roof live load, rain load, snow load) and other load effects as a result of earthquake, wind, flood, earth pressure, fluid pressure, etc.

Notations –

D = dead load

E = seismic (i.e., earthquake) load effect

E_m = maximum seismic load effect of horizontal and vertical seismic forces per *ASCE 7 – §12.4.3*

F = load due to fluids with well-defined pressures and maximum heights

F_a = flood load

H = load due to earth pressure, ground water pressure or pressure of bulk materials

L = live load (except roof live load) … including any permitted live load reduction

L_r = roof live load … including any permitted live load reduction

R = rain load

But wood members are very weak at resisting tension stresses applied perpendicular to the grain of the wood member (i.e., transverse to the length of the wood member).

Cross-grain tension refers to tension forces that result in tension stresses applied perpendicular to the grain of the wood member.

Cross-grain bending refers to bending moments that result in flexural tension stresses applied perpendicular to the grain of the wood member.

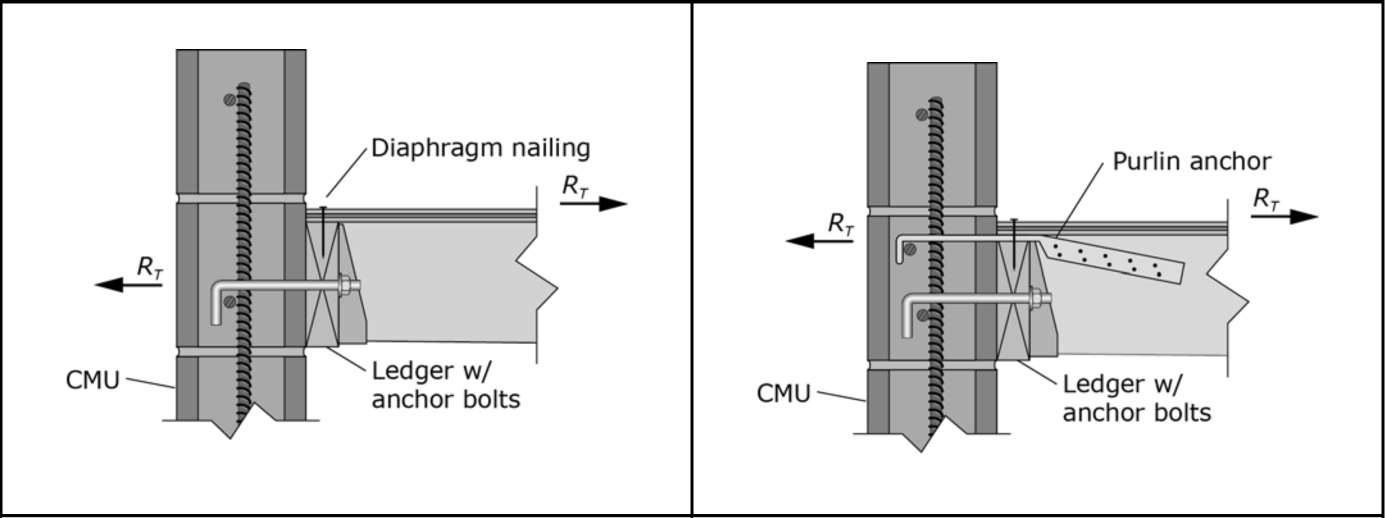
Figure 6.7a below demonstrates an unacceptable condition, since there is no positive direct connection of the structural wall to the wood diaphragm. The wall anchorage load path would be as follows:

1. The wood ledger anchor bolts will resist the wall anchorage force in tension,
2. The anchor bolt nut & washer will transfer the anchorage force to the middle of the ledger through bearing on the face of the wood ledger,
3. The roof sheathing nailing at the top of the ledger will attempt to transfer the wall anchorage force (in shear) into the main roof diaphragm.

Since the applied force at the middle of the ledger is eccentric to the resisting force at the top of the ledger, a bending moment will result in flexural tension stresses applied perpendicular to the grain of the wood ledger … or *cross-grain bending* which is not allowed per *ASCE 7 - §12.11.2.2.3.*

Figure 6.7b below demonstrates an acceptable condition using purlin anchors to provide a positive direct connection of the wall to the roof framing members (i.e., not relying on cross-grain bending).

Figure 6.7 – CMU Parapet Wall Anchorage to Wood Diaphragm



a. Not Acceptable

b. Acceptable

Figure 6.8 provides some examples of purlin anchors that might be used for masonry or concrete parapet wall anchorage to wood framed roofs. Again, the "DF/SP Allowable Loads" noted would be for Douglas-Fir (or Southern Pine) framing members. And since "allowable loads" are noted, the ASD load combinations of *IBC §1605.3.1* or *§1605.3.2* would apply … resulting in a calculated reduction of the strength design (SD) anchorage force by multiplying by 0.7 (or dividing by 1.4) to convert to an allowable stress design (ASD) anchorage force. BUT, being a steel element, the steel purlin anchor will require a 1.4 increase in the anchorage force (i.e., $R_T = (0.7)\cdot 1.4\cdot F_p$ for ASD anchorage force of steel purlin anchors) per *ASCE 7 – §12.11.2.2.2.*

Structural Analysis Procedure *ASCE 7 – §15.1.3*

- **Nonbuilding Structures Similar to Buildings -** structural analysis procedures shall be selected in accordance with *ASCE 7 – §12.6*.
- **Nonbuilding Structures NOT Similar to Buildings -** structural analysis procedure shall be one of the following:
    1. equivalent lateral force (ELF) procedure in accordance with *ASCE 7 – §12.8,*
    2. modal analysis procedure in accordance with *ASCE 7 – §12.9,*
    3. linear response history analysis procedure in accordance with *ASCE 7 – §16.1,*
    4. nonlinear response history analysis procedure in accordance with *ASCE 7 – §16.2*, or
    5. procedure prescribed in the specific reference document

Nonbuilding Structures Supported by Other Structures *ASCE 7 – §15.3*

Where nonbuilding structures NOT similar to buildings (i.e., identified in *ASCE 7 – Table 15.4-2*) are supported by other structures, and the nonbuilding structures are not part of the primary seismic-force-resisting system (SFRS), one of the following methods shall be used:

- **< 25% Combined Weight Condition** ***ASCE 7 – §15.3.1***

  Where the weight of the nonbuilding structure is less than 25 percent of the combined weight of the nonbuilding structure and supporting structure (i.e., $W_p < 25\% \cdot W$) … the design seismic forces of the nonbuilding structure shall be determined in accordance with *ASCE 7 – Chapter 13* (i.e., as a *nonstructural component*) where the values of R_p and a_p shall be determined per *ASCE 7 – §13.1.5*.

  The supporting structure shall be designed in accordance with the requirements of *ASCE 7 – Chapter 12* or *ASCE 7 – §15.5* as appropriate with the weight of the nonbuilding structure considered in the determination of the effective seismic weight (i.e., W_p included in W).

- **≥ 25% Combined Weight Condition** ***ASCE 7 – §15.3.2***

  Where the weight of the nonbuilding structure is equal to or greater than 25 percent of the combined weight of the nonbuilding structure and supporting structure (i.e., $W_p \geq 25\% \cdot W$) … an analysis combining the structural characteristics of both the nonbuilding structure and the supporting structures shall be performed per *ASCE 7 – §15.3.2, items 1 & 2*.

Architectural, Mechanical & Electrical Components *ASCE 7 – §15.3.3*

Architectural, mechanical, and electrical components supported by nonbuilding structures shall be designed as *nonstructural components* per *ASCE 7 – Chapter 13*.

7.2 Structural Design Requirements *ASCE 7 – §15.4*

Design Basis *ASCE 7 – §15.4.1*

Nonbuilding structures having specific seismic design criteria established in reference documents (i.e., noted in *ASCE 7 – Chapter 23*) shall be designed using the appropriate design standards as amended by *ASCE 7 – Chapter 15*.

Where reference documents are not cited … nonbuilding structures shall be designed in compliance with *ASCE 7 – §15.5* (i.e., nonbuilding structures similar to buildings) and *ASCE 7 – §15.6* (i.e., nonbuilding structures NOT similar to buildings) to resist minimum seismic lateral forces that are not less than the requirements of *ASCE 7 – §12.8* with the additions and exceptions of *ASCE 7 – §15.4.1, items 1* to *9*.

Importance Factor, *I* ASCE 7 – §15.4.1.1

The *Importance Factor* (*I*) and *Occupancy Category* for nonbuilding structures are based on the relative hazard of the contents and the function of the nonbuilding structure.

The *Occupancy Category* for a nonbuilding structure is determined from *IBC Table 1604.5.*

The *Importance Factor* (*I*) shall be the largest value determined by the following:

1. Applicable reference document listed in *ASCE 7 – Chapter 23*,
2. The largest value as selected from *ASCE 7 – Table 11.5-1*, or
3. As specified elsewhere in *ASCE 7 – Chapter 15*.

Effective Seismic Weight, *W* ASCE 7 – §15.4.3

The effective seismic weight, or operating weight, (*W*) shall include:

- ✓ All dead loads (*D*) as defined for buildings per *ASCE 7 – §12.7.2,* and
- ✓ All normal operating contents (e.g., tanks, vessels, bins, hoppers, and piping)

The effective seismic weight (*W*) shall include snow and ice loads where these loads constitute ≥ 25%·*W* … or where required by the building official (based on local environmental characteristics).

Fundamental Period, *T* ASCE 7 – §15.4.4

The fundamental period of the nonbuilding structure (*T*) shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis as per *ASCE 7 – §12.8.2*.

Alternatively, the fundamental period may be determined using the following equation:

$$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n} w_i \cdot \delta_i^2}{g \cdot \sum_{i=1}^{n} f_i \cdot \delta_i}} \qquad \textit{ASCE 7 (15.4-6)}$$

where:

w_i = effective seismic weight of Level *i*
f_i = lateral force at Level *i*
δ_i = elastic deflection at Level *i*, relative to the base
g = acceleration due to gravity (32.2 ft/sec^2 or 386.4 in/sec^2)

For a single-degree-of-freedom (SDOF) nonbuilding structure, this equation becomes:

$$T = 2\pi\sqrt{\frac{W}{K \cdot g}}$$

where:

W = effective seismic weight (i.e., operating weight)
K = stiffness of the nonbuilding structure
g = acceleration due to gravity (32.2 ft/sec^2 or 386.4 in/sec^2)

NOTE: The approximate fundamental period (T_a) of *ASCE 7 – §12.8.2.1* is not permitted to be used for nonbuilding structures.

Figure 8.8 below demonstrates the comparison that is often made between the analysis of a uniformly loaded flexible diaphragm (on the left) and a uniformly loaded simply supported beam (on the right). For the design of flexible diaphragms, the shear diagram can be used to determine the maximum unit shear at the end supports (e.g., shear walls). The moment diagram can be used to determine the maximum chord force, or the chord force at a specific point on the chord boundary member (see p. 1-103).

Figure 8.8 – Flexible Diaphragm Loading (Ref. 17)

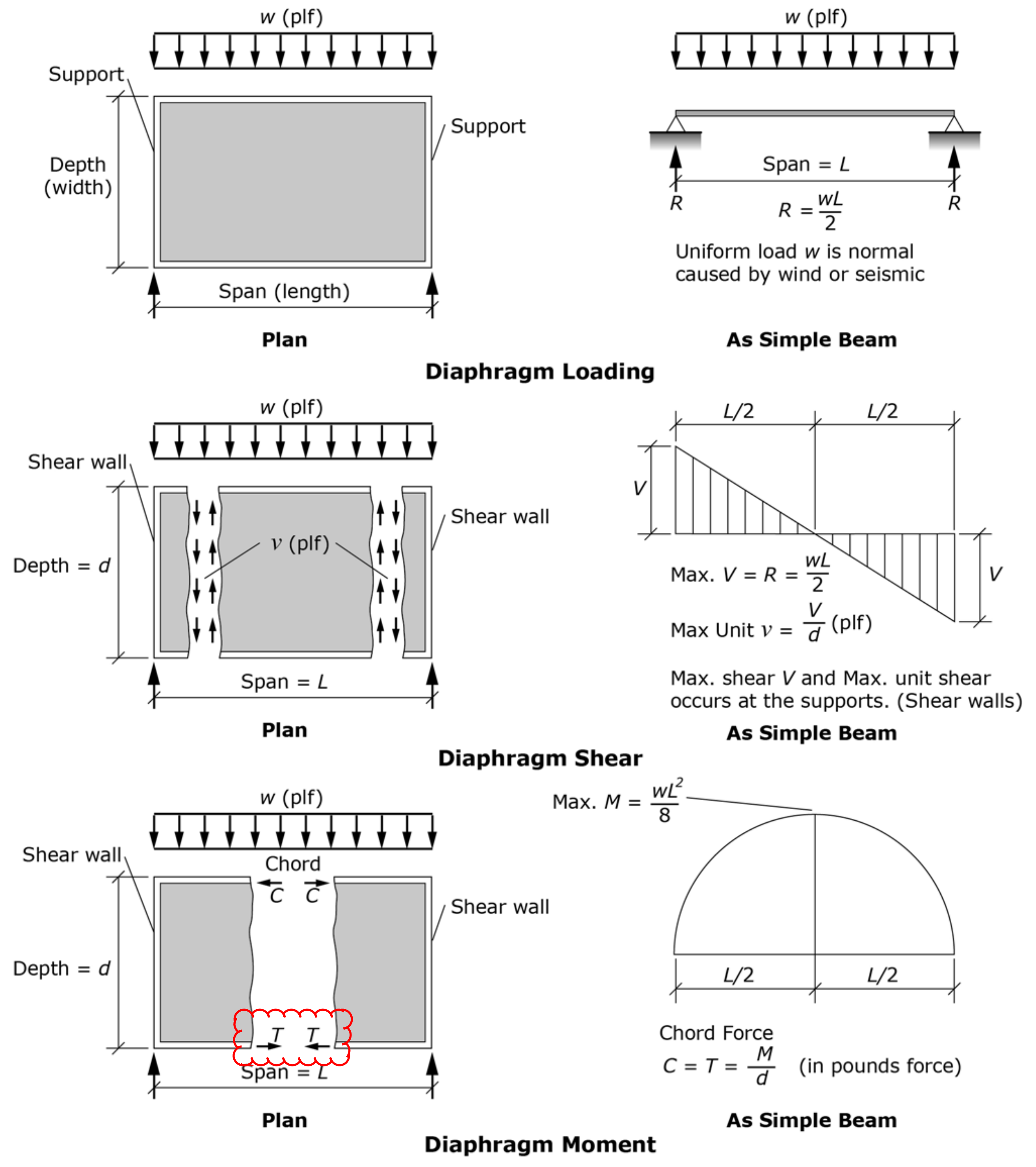
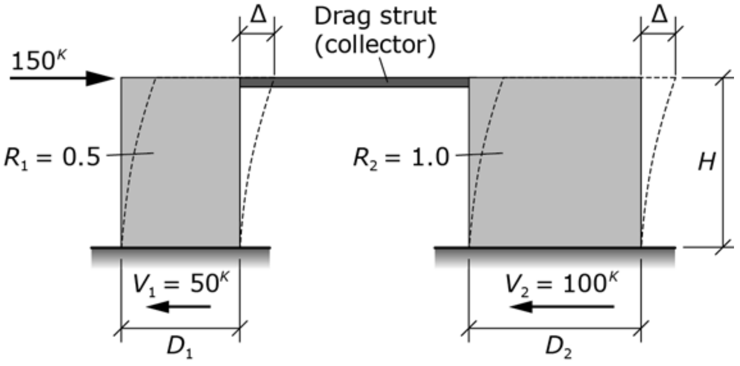


Figure 8.13 – Cantilever Shear Wall Relative Rigidity



$$V_1 = F\left(\frac{R_1}{R_1 + R_2}\right) \qquad V_2 = F\left(\frac{R_2}{R_1 + R_2}\right)$$

Fixed Shear Wall – Deflection

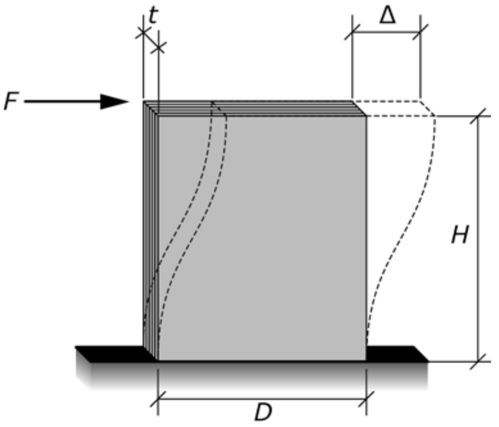
Figure 8.14 – Fixed Shear Wall / Pier

$$\Delta_{Total} = \Delta_{Flexure} + \Delta_{Shear}$$

$$\Delta_F = \frac{F \cdot H^3}{12EI} + \frac{1.2F \cdot H}{AG}$$

where:

F = force at top of wall
H = height of wall to force, F
E = modulus of elasticity
G = shear modulus
A = area = $t \cdot D$
I = moment of inertia = $t \cdot D^3/12$

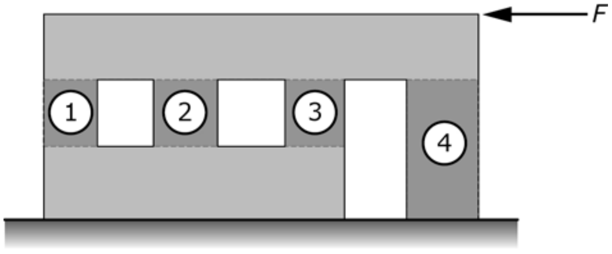


Shear Wall with Openings

Method A: Most Simple

Use this method to determine the force to a particular pier when the lateral force (F) to the total wall is known, such as for a flexible diaphragm building.

Figure 8.15 – Wall Pier Elevation



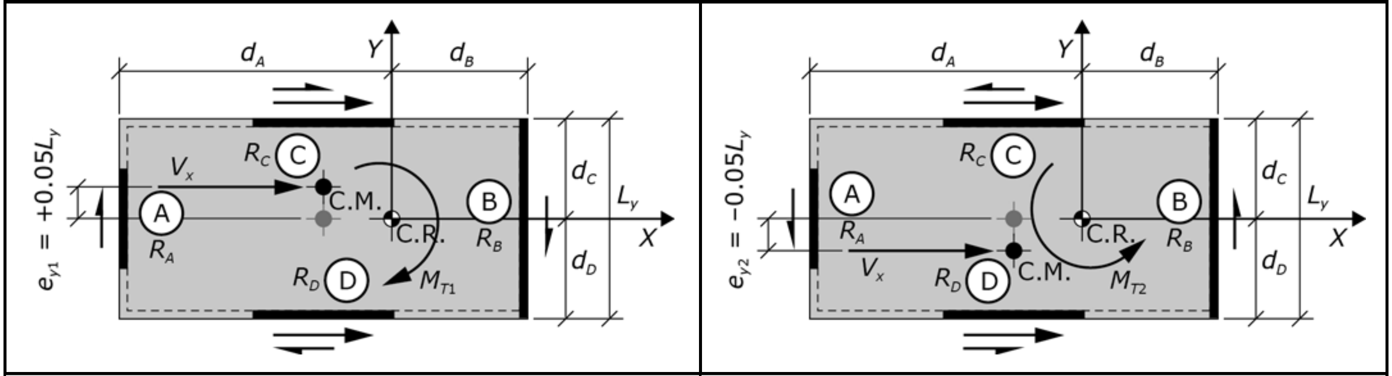
Determine the "Fixed" Rigidity (R_F) of each of the individual piers using their respective H/D ratios and Table D2 - Relative Rigidity of Fixed Shear Walls / Piers (Appendix D, p. 5-19).

$$\text{Force to Pier 1, } F_1 = F\left(\frac{R_{F1}}{R_{F1} + R_{F2} + R_{F3} + R_{F4}}\right)$$

Method B: More Tedious (see Ref. 9 – Lindeburg, 9th edition, Section 7-5, p. 110). Use this method when the total wall rigidity is needed for use in determining the lateral force to the wall, such as for rigid diaphragm buildings (e.g., center of rigidity, torsion).

Torsion: (*X*-Direction)

Figure 8.17 – Rigid Diaphragm with Torsion (*X*-Direction)



e_{y1} = calc. e_y + 0.05·L_y | **e_{y2} = calc. e_y – 0.05·L_y**

Inherent Torsional Moment - $M_t = V_x \cdot e_y$

Accidental Eccentricity - $e_y = \pm 0.05 \cdot L_y$

Design e_{y1} = calc. $e_y + 0.05 \cdot L_y$
Design e_{y2} = calc. $e_y - 0.05 \cdot L_y$

Accidental Torsional Moment - $M_{ta} = V_x \cdot (\pm 0.05 \cdot L_y)$

Design Torsional Moments -

$$M_{T1} = M_t + M_{ta} = V_x \cdot (e_y + 0.05 \cdot L_y)$$
$$M_{T2} = M_t - M_{ta} = V_x \cdot (e_y - 0.05 \cdot L_y)$$

Total Force to a Resisting Element, *F*

F = Direct Shear + Torsional Shear

$$\boxed{F = V_X \cdot \frac{R_X}{\sum R_X} + \frac{M_T \cdot R \cdot d}{\sum R \cdot d^2}}$$

Where:

$$\sum R \cdot d^2 = R_A \cdot d_A^2 + R_B \cdot d_B^2 + R_C \cdot d_C^2 + R_D \cdot d_D^2$$

NOTE: By observation of Figure 8.17, the following can be concluded –

- M_{T1} and M_{T2} are equal but opposite in sign (i.e., calculated $e_y = 0$)
- Torsional shears from M_{T1} are equal but opposite in sign to the torsional shears from M_{T2}
- e_{y1} will govern the design of shear wall C (i.e., maximum additive torsional shear)
- e_{y2} will govern the design of shear wall D (i.e., maximum additive torsional shear)
- Neither eccentricity will govern the design of shear walls A & B since the force direction is not parallel to these shear walls (i.e., no direct shear)

Chapter 9

IBC Chapter 23 – Wood

9.1 General *IBC §2301*

Scope *IBC §2301.1*

The provisions of *IBC Chapter 23* shall govern the materials, design, construction and quality of wood members and their fasteners.

General Design Requirements *IBC §2301.2*

The design of structural elements or systems constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

- *Allowable Stress Design* (ASD) – per *IBC §2304*, *§2305* and *§2306*
- *Load and Resistance Factor Design* (LRFD) – per *IBC §2304*, *§2305* and *§2307*
- *Conventional Light-Frame Construction* – per *IBC §2304* and *§2308*

  Exception: Buildings designed in accordance with the provisions of the *AF&PA Wood Frame Construction Manual (WFCM)* shall be deemed to meet the requirements of the provisions of *IBC §2308*

- *ICC 400* – for design and construction of log structures

9.2 Lateral-Force-Resisting Systems *IBC §2305*

General *IBC §2305.1*

Structures using wood shear walls and wood diaphragms to resist wind, seismic and other lateral loads shall be designed and constructed in accordance with *AF&PA Special Design Provisions for Wind and Seismic* (*SDPWS*) and the provisions of *IBC §2305* (General), *§2306* (ASD), and *§2307* (LRFD).

Design Requirements *SDPWS §4.1.1*

A continuous load path (or paths) with adequate strength and stiffness shall be provided to transfer all forces from their point of application to the final point of resistance.

Boundary Elements *SDPWS §4.1.4*

- ✓ Shear wall and diaphragm boundary elements shall be provided to transmit the design tension and compression forces.
- ✓ Diaphragm and shear wall sheathing shall not be used to splice boundary elements.
- ✓ Diaphragm chords and collectors shall be placed in, or in contact with, the plane of the diaphragm framing unless …

Toe-Nailed Connections *SDPWS §4.1.7*

In SDC = D, E & F – the capacity of toe-nailed connections shall not be used when calculating lateral load resistance to transfer seismic lateral forces > 150 plf for ASD (> 205 plf for LRFD) from diaphragms to shear walls, collectors (or other elements), or from shear walls to other elements.

9. **Controlled low-strength material (CLSM)** – *IBC §1803.5.9*
10. **Alternate setback and clearance*** – *IBC §1803.5.10*
11. **Seismic Design Category C, D, E & F*** – *IBC §1803.5.11*

    An investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions:

    - Slope instability
    - Liquefaction
    - Differential settlement ~~slope instability~~
    - Surface displacement due to faulting or lateral spreading

***<u>Exception</u>:** The *building official* shall be permitted to waive the requirement for a geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in *IBC §1803.5.1* through *§1803.5.6*, *§1803.5.10*, and *§1803.5.11*.

12. **Seismic Design Category D, E & F** – *IBC §1803.5.12*

    The geotechnical investigation required by *IBC §1803.5.11* shall also include:

    - ✓ Determination of lateral pressures on foundation walls and <u>retaining walls</u> due to earthquake motions.
    - ✓ Potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes and source characteristics consistent with the design earthquake ground motions …
    - ✓ An assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls and flotation of buried structures.
    - ✓ Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.

Reporting *IBC §1803.6*

Where geotechnical investigations are required, a written report of the investigations shall be submitted to the *building official* by the owner or authorized agent at the time of *permit* application. See *IBC §1803.6* for required information in the report.

Foundations *IBC §1808*

General *IBC – §1808.1*

Foundations shall be designed and constructed in accordance with *IBC §1808.2* through *§1808.9*. Shallow foundations shall also satisfy the requirements of *IBC §1809.* Deep foundations shall also satisfy the requirements *IBC §1810.*

Design Loads *IBC – §1808.3*

Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in *IBC §1605.2* (i.e., SD/LRFD) or *§1605.3* (i.e., ASD). The dead load is permitted to include the weight

Chapter 14

Geotechnical Issues & Lifelines

Geotechnical hazards that can result from earthquake ground motions include:

- Liquefaction-induced ground failures,
- Earthquake-induced landslides
- Surface fault rupture

14.1 Liquefaction

Liquefaction is essentially the temporary transformation of a solid material (with grain-to-grain contact) into a fluid like material.

Earthquake ground motions can cause an increase in pore water pressure, which results in a decrease in the soils effective stress. A reduction in effective stress corresponds to a decrease in shear strength. Since the bearing capacity of a soil is a function of the shear strength, the bearing capacity decreases in proportion to the reduction in shear strength. When the shear strength approaches zero, the soil may flow like a fluid.

Earthquake induced soil liquefaction can result in:

- ✓ Loss of soil bearing capacity
- ✓ Soil settlement
- ✓ Lateral spreading
- ✓ Flow slides on soil slopes

Soils most susceptible to liquefaction are saturated, relatively cohesionless/clay-free sands and silts at or below groundwater, and occasionally loose gravels below the water table deposited by rivers. Other factors include soil density, gradation, confining pressure, and the geologic history of the soil deposit.

- Dense sands are less susceptible to liquefaction than loose sands.
- Well-graded sands are less susceptible to liquefaction than uniform sands (i.e., more stable interlocking of grains).
- Sands below a depth of approximately 50 feet are less susceptible to liquefaction (i.e., due to confining pressure).
- Geologically old sand deposits are less susceptible to liquefaction than recent sand deposits (i.e., possibly due to previous earthquake induced settlements & densification).

Extensive damage may occur to structures supported on liquefiable soils due to the loss of bearing capacity and large settlements. Damage might be avoided by supporting the structure on piles or drilled piers that pass through the saturated sand layer that are supported by sound material below (i.e., bedrock). Regardless, liquefaction may still result in settlement of the saturated sand layer below the structure.

Even without the presence of structures on liquefiable soils, *lateral spreading* alone can cause extensive damage to roadways and pipelines (above or below ground).

Solution:

A.) N-S DIRECTION: $L = 100'$, $d = 40'$, CASE 3

$L/d = 100'/40' = 2.5{:}1 < 3{:}1$ max. per *SDPWS Table 4.2.4* (see Table 9.1, p. 1-118) for an unblocked diaphragm → **OK**

1. Unit Roof Shear on lines A & B, v_r

$V_A = V_B = w_s \cdot L / 2 = (300\text{ plf})(100'/2) = \underline{15{,}000\text{ lbs}}$ (SD/LRFD force level)

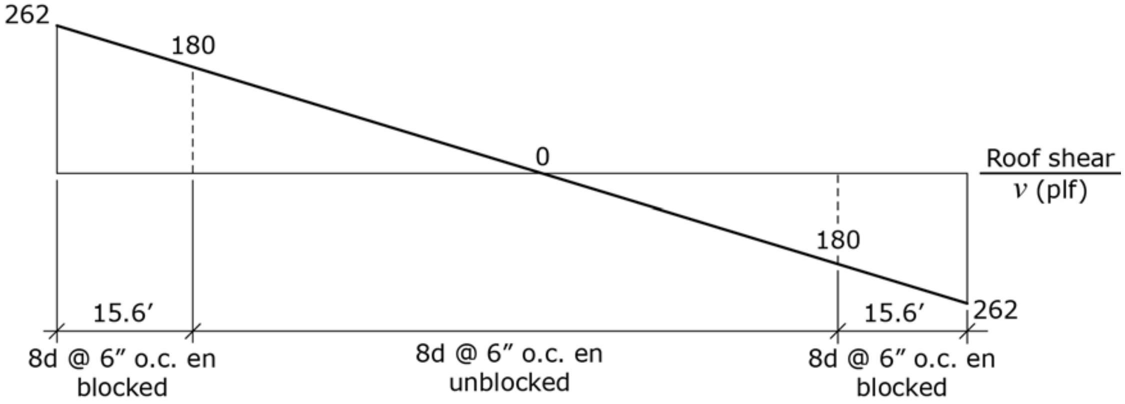
roof $v_A = v_B = 0.7 \cdot V_A / d = 0.7\ (15{,}000\text{ lbs}) / 40' = \boxed{262\text{ plf}}$ (ASD force level)

2. Roof Diaphragm Nailing

roof $v_A = 262$ plf → *IBC Table 2306.2.1(1)* →
15/32" WSP sheathing w/ 8d common (2½″ x 0.131″) @ 6" o.c. edges, unblocked (CASE 3)
allowable unit diaphragm shear = 180 plf < 262 plf → **NG!**

Use 15/32″ WSP sheathing w/ 8d common @ 6" o.c. boundary and edge nailing, 12" o.c. field nailing. Blocked (CASE 3) allowable unit diaphragm shear = 270 plf > 262 plf → **OK**

NOTE: Blocking of the roof diaphragm may be terminated when the calculated unit roof shear drops below the allowable unit shear for an unblocked diaphragm (e.g., 180 plf). This occurs approximately 15.6′ from lines A & B.



3. Maximum Chord Force on lines 1 & 2, *CF*

max. $M = w_s \cdot L^2 / 8 = (300\text{ plf})(100')^2 / 8 = 375{,}000$ lb-ft (SD/LRFD force level)

max. $CF = 0.7 \cdot M / d = 0.7\ (375{,}000\text{ lb-ft}) / (40') = \boxed{6{,}560\text{ lbs}}$ (ASD force level)

4. Unit Wall Shear & Nailing on lines A & B, v_w

- Wall Line A: total shear wall length, $\Sigma b = 20'$

$h/b = 12'/20' = 0.60{:}1 \ll 2{:}1$ max. per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →
∴ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary

wall $v_A = \rho\,(0.7 \cdot V_A) / \Sigma b = (1.00)(0.7)(15{,}000\text{ lbs}) / (20') = \boxed{525\text{ plf}}$ (ASD force level)

wall $v_A = 525$ plf → *IBC Table 2306.3* →

Wall Line A - use 15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing & 12″ o.c. field nailing … allowable unit wall shear = 550 plf > 525 plf → **OK**
(3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnote i*)

- Wall Line B: total shear wall length, $\Sigma b = 40'$

$h/b = 12'/40' = 0.30{:}1 \ll 2{:}1$ max. per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →
$\therefore$ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary

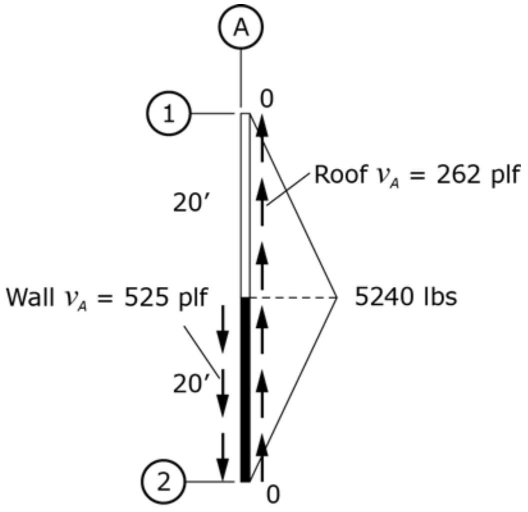
wall $v_B = \rho\,(0.7\cdot V_B) / \Sigma b = (1.00)(0.7)(15{,}000\text{ lbs}) / (40') = \boxed{262\text{ plf}}$ (ASD force level)
wall $v_B = 262$ plf → *IBC Table 2306.3* →

Wall Line B - use 15/32″ WSP Structural I sheathing w/ 8d common @ 6″ o.c. edge nailing & 12″ o.c. field nailing … allowable unit wall shear = 280 plf > 262 plf → **OK**

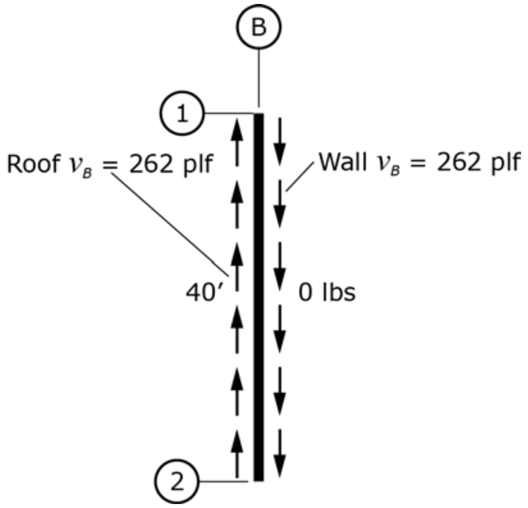
5. Drag Force Diagram on lines A & B, F_d

roof $v_A = v_B = 262$ plf (ASD force level)

- Wall Line A: $F_d = (262\text{ plf})(20') = 5{,}240$ lbs (ASD force level)
- Wall Line B: $F_d = 0$ lbs



Drag Force – Line A



Drag Force – Line B

B.) E-W DIRECTION: $L = 40'$, $d = 100'$, CASE 1

$L/d = 40'/100' = 0.4{:}1 < 3{:}1$ max. per *SDPWS Table 4.2.4* (see Table 9.1, p. 1-118) for an unblocked diaphragm → **OK**

1. Unit Roof Shear on lines 1 & 2, v_r

$V_1 = V_2 = w_s\cdot L / 2 = (490\text{ plf})(40'/2) = 9{,}800$ lbs (SD/LRFD force level)

roof $v_1 = v_2 = (0.7\cdot V_1) / d = (0.7)(9{,}800\text{ lbs}) / 100' = \boxed{69\text{ plf}}$ (ASD force level)

2. Roof Diaphragm Nailing

roof $v_1 = 69$ plf → *IBC Table 2306.2.1(1)* →
15/32" WSP sheathing w/ 8d common (2½″ x 0.131″) @ 6″ o.c. edges, unblocked (CASE 1)
allowable unit diaphragm shear = 240 plf >> 69 plf → **OK**

BUT … N-S direction required a blocked diaphragm (15.6′ minimum from lines A & B), which governs the diaphragm design! Therefore, provide diaphragm nailing per N-S Direction (part A).

3. Maximum Chord Force on lines A & B, *CF*

max. $M = w_s \cdot L^2 / 8 = (490 \text{ plf})(40')^2 / 8 = 98{,}000$ lb-ft (SD/LRFD force level)

max. $CF = 0.7 \cdot M / d = (0.7)(98{,}000 \text{ lb-ft}) / (100') = \boxed{690 \text{ lbs}}$ (ASD force level)

4. Unit Wall Shear & Nailing on lines 1 & 2, v_w

- Wall Line 1: total shear wall length, $\Sigma b = 7' + 13' = 20'$

Maximum $h/b = 12'/7' = 1.71{:}1 < 2{:}1$ max. per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →

∴ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary

wall $v_1 = \rho\,(0.7 \cdot V_1) / \Sigma b = (1.00)(0.7)(9{,}800 \text{ lbs}) / (20') = \boxed{343 \text{ plf}}$ (ASD force level)

wall $v_1 = 343$ plf → *IBC Table 2306.3* →

Wall Line 1 - use 15/32″ WSP Structural I sheathing w/ 8d common @ 4″ o.c. edge nailing & 12″ o.c. field nailing for both walls … allowable unit wall shear = 430 plf > 343 plf → **OK**
(3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

- Wall Line 2: total shear wall length, $\Sigma b = 10' + 5' = 15'$

Minimum $h/b = 12'/10' = 1.2{:}1 < 2{:}1$ maximum per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →

∴ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary for 10′ shear wall

Maximum $h/b = 12'/5' = 2.4{:}1 > 2{:}1$ maximum per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →

∴ A reduction of unit wall shears in *IBC Table 2306.3* **is** necessary for 5′ shear wall …
where the reduction factor = $2b/h = 2(5')/(12') = 0.83$

wall $v_2 = \rho\,(0.7 \cdot V_2) / \Sigma b = (1.00)(0.7)(9{,}800 \text{ lbs}) / (15') = \boxed{457 \text{ plf}}$ (ASD force level)

wall $v_2 = 457$ plf → *IBC Table 2306.3* →

15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing.

10′ shear wall - allowable unit wall shear = 550 plf > 457 plf → **OK** (w/ no reduction)

Wall Line 2 – 10′ shear wall - use 15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing … allowable unit wall shear = 550 plf > 457 plf → **OK**
(3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

15/32″ WSP sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing.

5′ shear wall - allowable unit wall shear = (0.83)(550 plf) = 456 plf ≈ 457 plf → **OK**

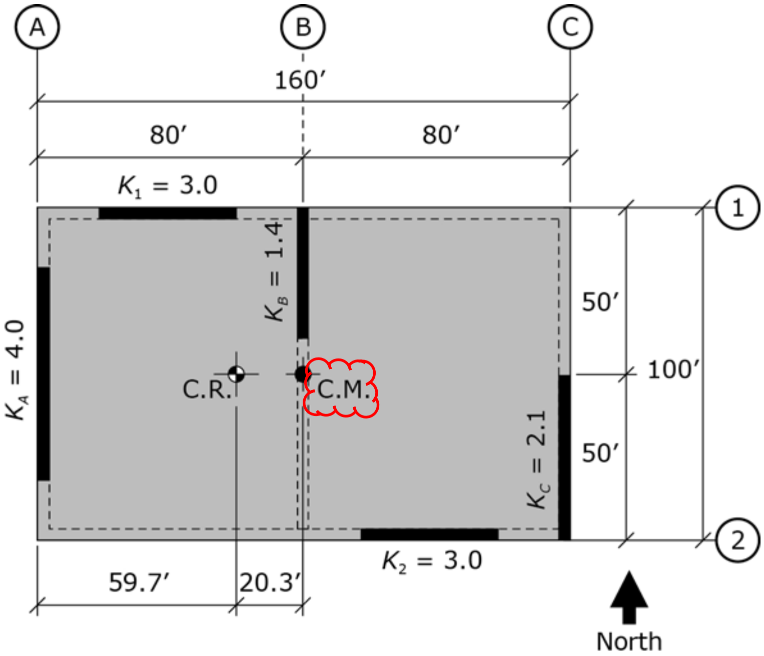
Wall Line 2 – 5′ shear wall - use 15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing … allowable unit wall shear = 0.83(550 plf) = 456 plf ≈ 457 plf → **OK**
(3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

NOTE: – the allowable unit wall shear reduction factor 2*b*/*h* per *SDPWS Table 4.3.4, footnote 1* can easily result in separate wood structural panel shear walls on the same wall line with different required edge nail spacing … as nearly occurred in this example on Wall Line 2.

Problem #19

Given:

- Single-story building w/ special steel concentrically braced frames
- Concrete (rigid) roof diaphragm
- Relative stiffness of frames shown on plan
- Calculated Center of Mass (*CM*) and Center of Rigidity/Stiffness (*CR*) shown per Plan
- Seismic base shear, N-S & E-W Directions, $V = 300$ kips



Plan

Find:

A.) N-S DIRECTION:

1. Accidental eccentricity (e_x), accidental & inherent torsional moments (M_{ta} & M_t)
2. Design eccentricities (e_{x1} & e_{x2}) and design torsional moments (M_{T1} & M_{T2})
3. Building plan sketch showing direct & torsional shears for each design eccentricity
4. Total design force to frames A, B & C

B.) E-W DIRECTION:

1. Accidental eccentricity (e_y), accidental & inherent torsional moments (M_{ta} & M_t)
2. Design eccentricities (e_{y1} & e_{y2}) and design torsional moments (M_{T1} & M_{T2})
3. Building plan sketch showing direct & torsional shears for each design eccentricity
4. Total design force to frames 1 & 2

B.) Center of Rigidity, *CR*

Shear Wall Rigidities: (assume cantilever walls, Table D1 - Relative Rigidity of Cantilever Shear Walls / Piers, Appendix D, p. 5-18)

Wall A : $H/D = 15'/30' = 0.50$ → Table D1 (p. 5-18) → $R_A = \underline{5.0}$

Wall B : $H/D = 15'/20' = 0.75$ → Table D1 (p. 5-18) → $R_B = \underline{2.54}$

$\sum R_Y = R_A + R_B = 5.0 + 2.54 = \underline{7.54}$

Wall C : $H/D = 15'/40' = 0.375$ → Table D1 (p. 5-18) → $R_C = \underline{7.49}$

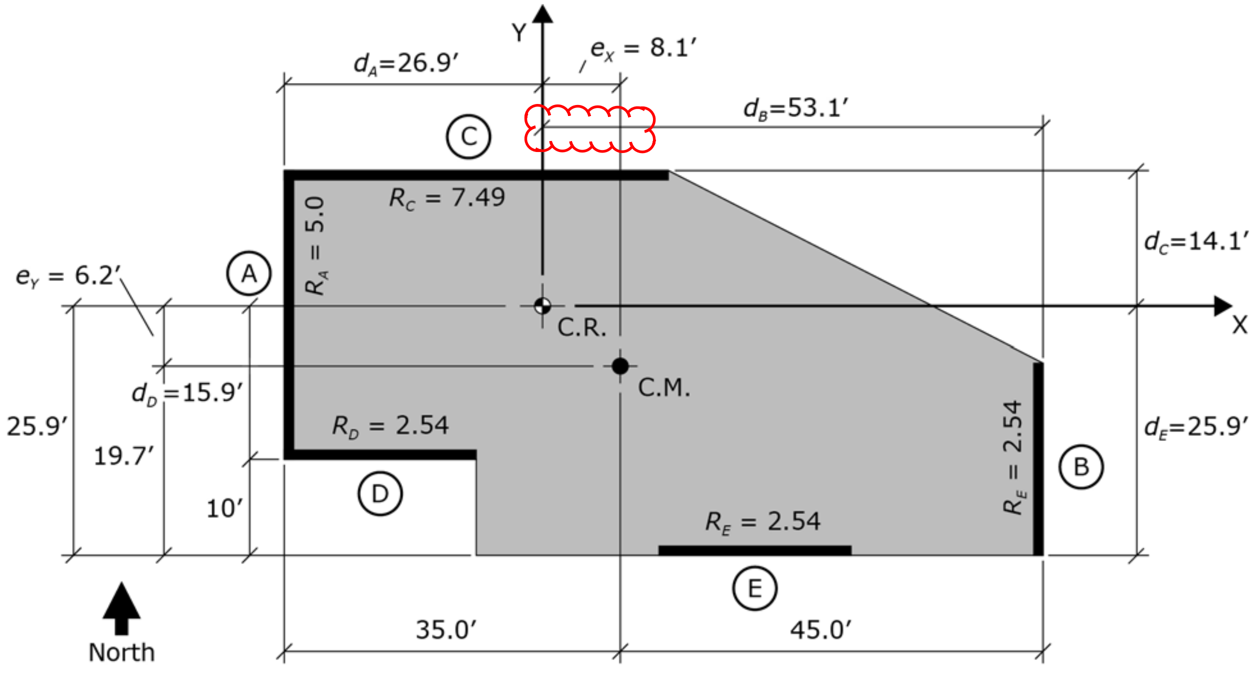
Wall D : $R_D = R_B = \underline{2.54}$

Wall E : $R_E = R_D = \underline{2.54}$

$\sum R_X = R_C + R_D + R_E = 7.49 + 2.54 + 2.54 = \underline{12.57}$

$$\overline{X}_{CR} = \frac{\sum R_Y \cdot \overline{x}}{\sum R_Y} = [5.0\,(0') + 2.54\,(80')]\,/\,(7.54) = \boxed{26.9 \text{ feet}}$$

$$\overline{Y}_{CR} = \frac{\sum R_X \cdot \overline{y}}{\sum R_X} = [7.49\,(40') + 2.54\,(10') + 2.54\,(0')]\,/\,(12.57) = \boxed{25.9 \text{ feet}}$$



$$\text{Maximum } F_B = \left(V_Y \cdot \frac{R_B}{\sum R_Y}\right) + \left(\frac{M_{T1} \cdot R_B \cdot d_B}{\sum R \cdot d^2}\right)$$

$= [73.4 \text{ kips } (2.54) / (7.54)] + [888.1 \text{ kip-ft } (2.54)(53.1 \text{ ft}) / (14{,}615 \text{ ft}^2)]$

$= 24.7 \text{ kips} + 8.2 \text{ kips} = \boxed{32.9 \text{ kips}}$

D.) <u>E-W DIRECTION</u> – Design Force to Shear Walls C, D & E

$V = 73.4$ kips

Accidental eccentricity, $e_y = \pm\, 0.05 \cdot L_\perp = \pm\, 0.05\,(40') = \pm\, 2.0'$
Accidental torsional moment, $M_{ta} = V \cdot (\pm\, 0.05 \cdot L_\perp) = 73.4 \text{ kips } (\pm\, 2.0') = \underline{\pm 146.8 \text{ kip-ft}}$

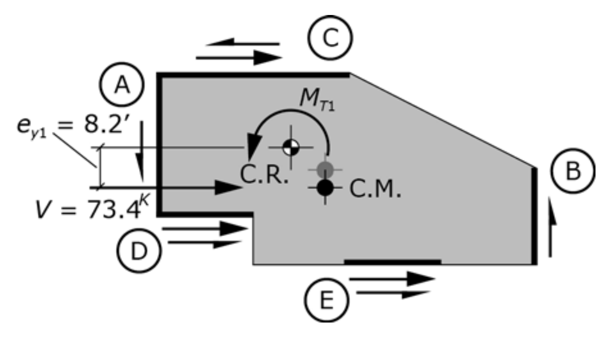
Calculated $e_y = 6.2'$ (from plan)
Inherent torsional moment, $M_t = V \cdot e_y = 73.4 \text{ kips } (6.2') = \underline{+455.1 \text{ kip-ft}}$

Design $e_y = e_y \pm 0.05 \cdot L_\perp = 6.2' \pm 2.0'$

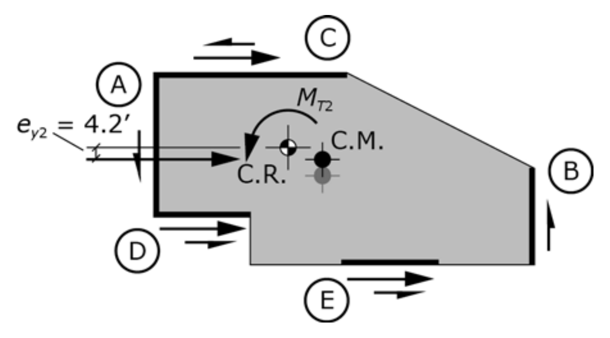
$e_{y1} = 6.2' + 2.0' = \underline{+8.2 \text{ feet}}$
$e_{y2} = 6.2' - 2.0' = \underline{+4.2 \text{ feet}}$

$M_{T1} = V \cdot (e_y + 0.05 \cdot L_\perp)$
$= M_t + M_{ta} = 455.1 + 146.8 = \underline{+601.9 \text{ kip-ft}}$

$M_{T2} = V \cdot (e_y - 0.05 \cdot L_\perp)$
$= M_t - M_{ta} = 455.1 - 146.8 = \underline{+308.3 \text{ kip-ft}}$



e_{y1} = calc. e_y + 0.05·L_y



e_{y2} = calc. e_y – 0.05·L_y

<u>**NOTE**</u>**:** By observation, e_{y1} will govern the design of shear walls D & E (i.e., <u>maximum</u> additive torsional shear) <u>and</u> e_{y2} will govern the design of shear wall C (i.e., <u>minimum</u> subtractive torsional shear).

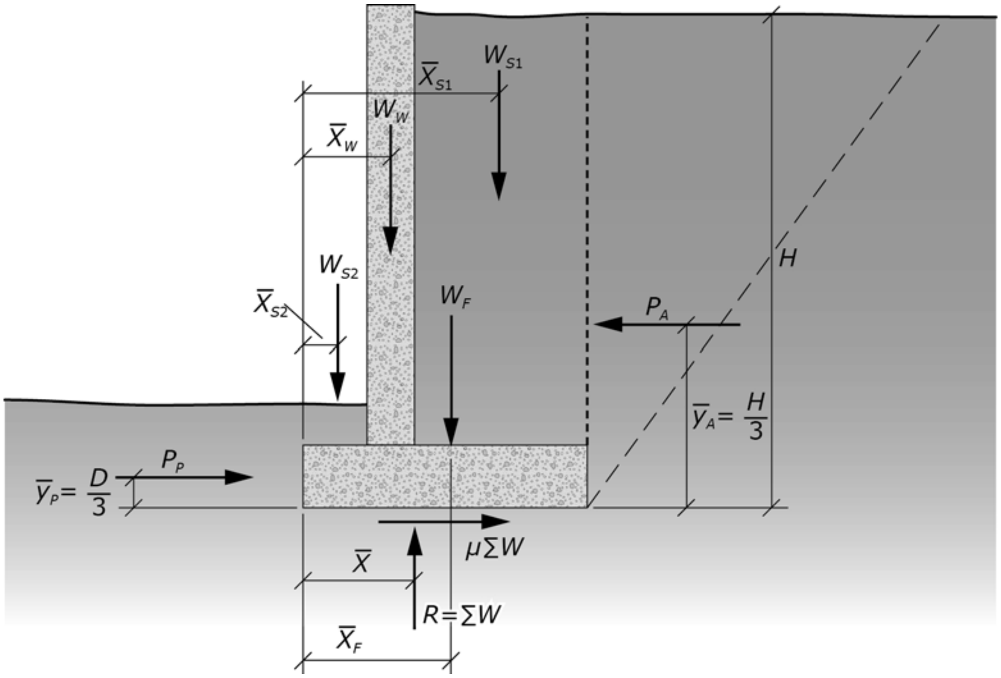
Neither eccentricity will govern the design of shear walls A & B since the force direction is not parallel to these walls (i.e., no direct shear).

$\sum R_X = R_C + R_D + R_E = 7.49 + 2.54 + 2.54 = \underline{12.57}$

$\sum R \cdot d^2 = \underline{14{,}615 \text{ ft}^2}$ … from Part C

<u>Solution</u>:

A.) STATIC CONDITION, K_A:



Static Active Soil Pressure

<u>Total static active force</u>, P_A	<u>Total static passive (resisting) force</u>, P_P
$P_A = ½\ K_A \cdot \gamma \cdot H^2$ $= ½\ (0.318)(110\text{ pcf})(11.25')^2 = \underline{2{,}214\text{ lbs/ft}}$	$P_P = ½\ K_P \cdot \gamma \cdot D^2$ $= ½\ (3.18)(110\text{ pcf})(2.25')^2 = \underline{885\text{ lbs/ft}}$
resultant height, $\bar{y}_A = H/3 = (11.25')/3 = 3.75'$	resultant height, $\bar{y}_P = D/3 = (2.25')/3 = 0.75'$

Weights

Soil over heel, $W_{S1} = (10')(4')(110\text{ pcf}) = 4{,}400\text{ lbs/ft}$

Soil over toe, $W_{S2} = (1')(1.5')(110\text{ pcf}) = 165\text{ lbs/ft}$

Concrete stem wall, $W_W = (10')(1')(150\text{ pcf}) = 1{,}500\text{ lbs/ft}$

Concrete footing, $W_F = (1.25')(6.5')(150\text{ pcf}) = 1{,}219\text{ lbs/ft}$

Resultant weight, $R = \Sigma W = 4{,}400 + 165 + 1{,}500 + 1{,}219 = \underline{7{,}284\text{ lbs/ft}}$

1. Sliding Factor of Safety

Sliding force, $F_S = P_A = \underline{2{,}214\text{ lbs/ft}}$

Resisting force, F_R = passive force + friction force

$$= P_P + \mu \cdot \Sigma W$$
$$= 885\text{ lbs/ft} + 0.4\ (7{,}284\text{ lbs/ft}) = \underline{3{,}799\text{ lbs/ft}}$$

Sliding factor of safety, $FS = \dfrac{F_R}{F_S} = \dfrac{3{,}799}{2{,}214} = \boxed{1.72} > 1.5$ minimum per *IBC §1807.2.3* **<u>OK</u>**

2. Overturning Factor of Safety

Overturning moment, $OTM = P_A \cdot \bar{y}_A$

$$= (2{,}214 \text{ lbs/ft})(11.25'/3) = \underline{8{,}302 \text{ lb·ft/ft}}$$

Resisting moment,

$$RM = P_P \cdot \bar{y}_P + W_{S1} \cdot \bar{x}_{S1} + W_{S2} \cdot \bar{x}_{S2} + W_W \cdot \bar{x}_W + W_F \cdot \bar{x}_F$$

$$= 885 \text{ lbs/ft } (2.25')/3 + 4{,}400\ (4.5') + 165\ (0.75') + 1{,}500\ (2') + 1{,}219\ (3.25')$$

$$= \underline{27{,}550 \text{ lb·ft/ft}}$$

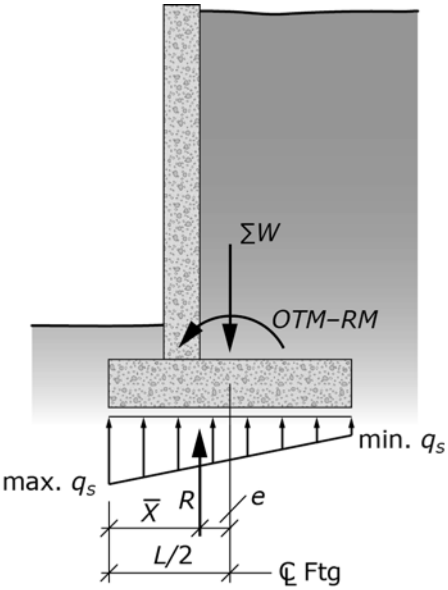
Overturning factor of safety, $FS = \dfrac{RM}{OTM} = \dfrac{27{,}550}{8{,}302} = \boxed{3.32} > 1.5$ minimum per *IBC* §1807.2.3 **OK**

3. Maximum Soil Bearing Pressure

$$\bar{x} = \frac{RM - OTM}{R} = \frac{(27{,}550 - 8{,}302)}{7{,}284} = 2.64'$$

eccentricity from centerline of footing,

$$e = L/2 - \bar{x} = (6.5')/2 - 2.64' = \underline{0.61'}$$



Soil Bearing Pressure

if $e < L/6$ (i.e., R is within middle 1/3 of footing) → the soil pressure distribution is trapezoidal

if $e \geq L/6$ (i.e., R is outside of middle 1/3 of footing) → the soil pressure distribution is triangular

$e = 0.61' < L/6 = 6.5'/6 = 1.08'$ → ∴ soil pressure distribution is trapezoidal

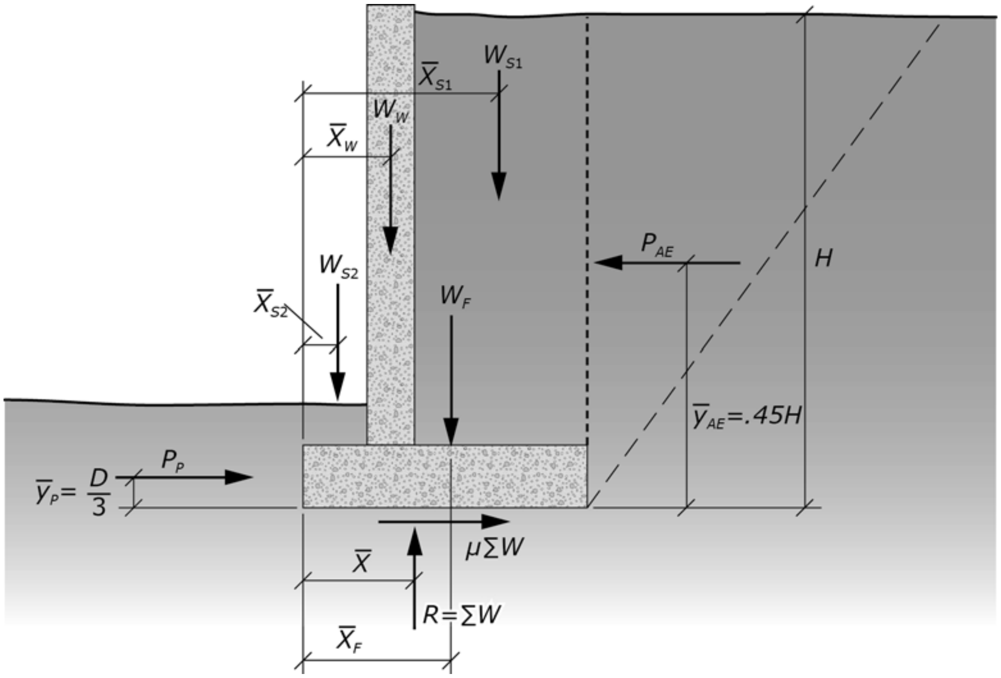
for a trapezoidal soil pressure distribution,

maximum soil pressure, $q_s = \dfrac{R}{L}\left(1 + \dfrac{6 \cdot e}{L}\right)$

minimum soil pressure, $q_s = \dfrac{R}{L}\left(1 - \dfrac{6 \cdot e}{L}\right)$

therefore, max. $q_s = \dfrac{7{,}284}{6.5'}\left(1 + \dfrac{6 \cdot (0.61')}{6.5'}\right) = \boxed{1{,}750 \text{ psf/ft}} < 3{,}000$ psf allowable $(D + L)$ **OK**

B.) STATIC PLUS SEISMIC CONDITION, K_{AE}:



Static plus Seismic Active Soil Pressure

Total static plus seismic active force, P_{AE} $P_{AE} = \frac{1}{2}\, K_{AE} \cdot \gamma \cdot H^2$ $= \frac{1}{2}\,(0.538)(110\text{ pcf})(11.25')^2 = \underline{3{,}745\text{ lbs/ft}}$ resultant height, $\overline{y}_{AE} = 0.45 \cdot H$ $= 0.45\,(11.25') = 5.06'$	Total static passive (resisting) force, P_P (from part A) $P_P = \underline{885\text{ lbs/ft}}$ resultant height, $\overline{y}_P = D/3 = 0.75'$

Weights (from part A)

Resultant weight, $R = \Sigma W = \underline{7{,}284\text{ lbs/ft}}$

1. Sliding Factor of Safety

Sliding force, $F_S = P_{AE} = \underline{3{,}745\text{ lbs/ft}}$

Resisting force, F_R = passive force + friction force

$$= P_P + \mu \cdot \Sigma W$$
$$= 885\text{ lbs/ft} + 0.4\,(7{,}284\text{ lbs/ft}) = \underline{3{,}799\text{ lbs/ft}}$$

Sliding factor of safety, $FS = \dfrac{F_R}{F_S} = \dfrac{3{,}799}{3{,}745} = \boxed{1.01} < 1.5 / 1.33 = 1.1$ **NG!**

NOTE: *2009 IBC §1605.3.2 & §1806.1* (and most Geotechnical reports) allow a one-third increase in allowable stress for all load combinations that include short-term loads such as earthquake (or wind). Although not specifically addressed in the *IBC*, many designers allow a reduced factor for safety (for sliding and overturning) when considering these short-term loads … i.e., short term $FS = 1.5 / 1.33 = 1.1$

3.43 Which of the following occupancies types would never be assigned to Seismic Design Category F (SDC = F)?

a. Hospital
b. Single-family residence
c. County jail
d. Both b & c

3.44 A 5-story building with offices in the upper four stories and a fire station in the first story, would be assigned to what Occupancy Category per *IBC Table 1604.5*?

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

3.45 What would be the most appropriate spectral acceleration response parameters (S_S & S_1) for a building project proposed at 36º00′00″ Latitude and -120º00′00″ Longitude?

a. $S_S = 1.75$ & $S_1 = 0.80$
b. $S_S = 1.75$ & $S_1 = 0.60$
c. $S_S = 0.95$ & $S_1 = 0.35$
d. $S_S = 0.75$ & $S_1 = 0.35$

3.46 What would be the most appropriate spectral acceleration response parameters (S_S & S_1) for a building project proposed at 39º00′00″ Latitude and -123º00′00″ Longitude?

a. $S_S = 2.00$ & $S_1 = 0.70$
b. $S_S = 1.55$ & $S_1 = 0.70$
c. $S_S = 1.25$ & $S_1 = 0.55$
d. $S_S = 1.25$ & $S_1 = 0.45$

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

a. Site Class A
b. Site Class B
c. Site Class C
d. Site Class D

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?*

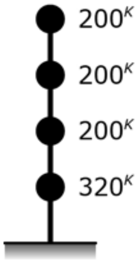
a. $F_a = 1.0$ & $F_v = 1.0$
b. $F_a = 1.2$ & $F_v = 1.8$
c. $F_a = 1.3$ & $F_v = 1.9$
d. $F_a = 1.4$ & $F_v = 2.0$

4.68 A structural analysis has been performed on a two-story apartment building (with parking garage in the first-story). The lateral story strength of the first and second stories were determined to be 57 kips and 76 kips respectively. The story stiffness of the first and second stories was determined to be 14 kips/inch and 19.5 kips/inch respectively. Which of the following vertical irregularities are present in this structure?

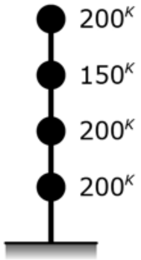
I. Stiffness – Soft Story
II. Discontinuity in Lateral Strength – Weak Story

a. I
b. II
c. I & II
d. None of the above

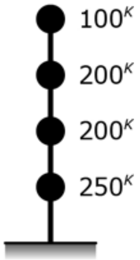
4.69 Which of the following structures are considered to have a Weight (Mass) Irregularity?



I.



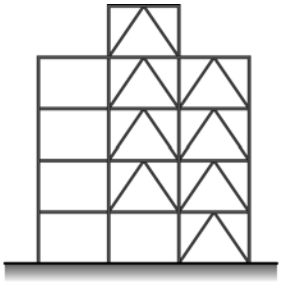
II.



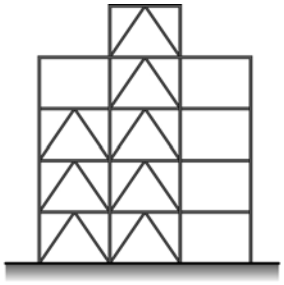
III.

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

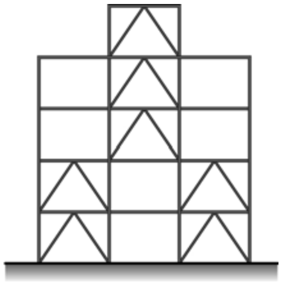
4.70 Which of the following braced frame structures is likely to have a Stiffness-Soft Story Irregularity?



I.



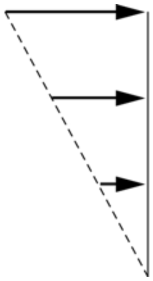
II.



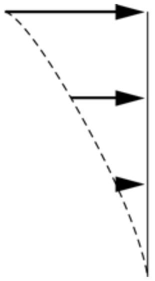
III.

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

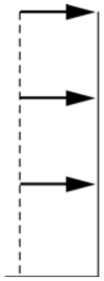
Using the figures below, answer questions 4.79 through 4.82



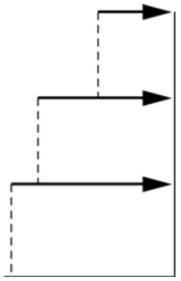
a.



b.



c.



d.

4.79 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure vertical distribution of seismic forces (F_x) for structures with a period (T) less than or equal to 0.5 seconds?

4.80 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure story shear distribution (V_x) for structures with a period (T) less than or equal to 0.5 seconds?

4.81 Which figure best represents the *Simplified Design* procedure vertical distribution of seismic forces (F_x)?

4.82 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure vertical distribution of seismic forces (F_x) for structures with a period (T) greater than or equal to 2.5 seconds?

4.83 Given a 15-story Office Bldg w/ steel moment frames, which site class is likely to result in the largest seismic forces?

a. Site Class B (rock)
b. Site Class C (dense soil)
c. Site Class D (stiff soil)
d. Site Class E (soft soil)

4.84 Given two structures with the same R, I, $S_{DS} = 0.73$ & $S_{D1} = 0.30$. Structure A has a period (T_A) of 0.35 second. Structure B has an effective seismic weight of 3 times that of Structure A (i.e., $W_B = 3 \cdot W_A$). What would be the period of Structure B such that the Base Shear (V) of the two structures would be equal?

a. 0.35 second
b. 0.65 second
c. 1.25 seconds
d. 2.15 seconds

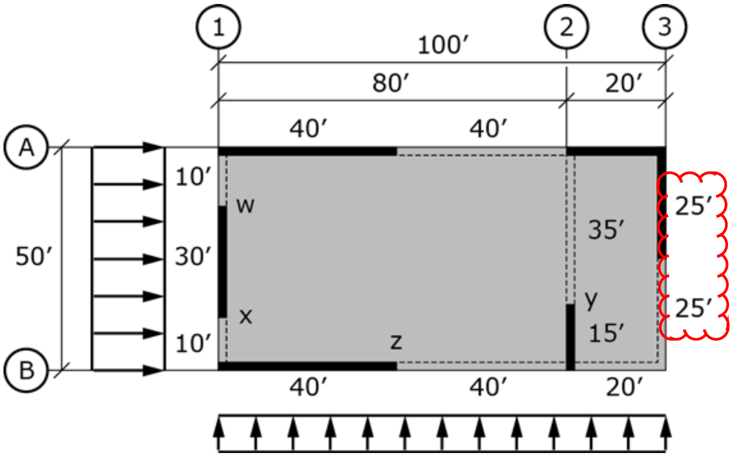
8.43 Assuming flexible diaphragms at all levels, what should be the force used to design the shear walls in the 2^{nd} story?

a. Vertical distribution force at level 2, F_2
b. Diaphragm design force at level 2, F_{p2}
c. 2^{nd} story shear, V_2
d. Base shear, V

8.44 Assuming flexible diaphragms at all levels, what should be the force used to design the chords and collectors at level 2?

a. Vertical distribution force at level 2, F_2
b. Diaphragm design force at level 2, F_{p2}
c. 2^{nd} story shear, V_2
d. Base shear, V

Given the single story building below with a flexible roof diaphragm and a base shear, V = 15 kips, answer questions 8.45 through 8.59



Plan

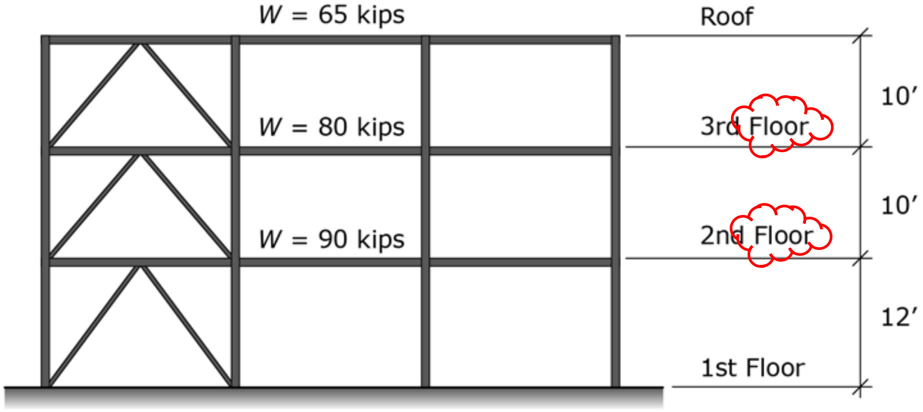
8.45 What is the lateral force in the shear wall on line 1?

a. 6.0 kips
b. 7.5 kips
c. 9.0 kips
d. 12.0 kips

8.46 What is the lateral force in the shear wall on line 2?

a. 6.0 kips
b. 7.5 kips
c. 9.0 kips
d. 12.0 kips

Given the three story office building below with a base shear of 24 kips, approximate fundamental period of 0.25 second, $S_{DS} = 0.60$ & $S_{D1} = 0.25$, answer questions 8.60 through 8.62



Elevation

8.60 What is the lateral force at the 2nd floor level using *ASCE 7-05 §12.8.3*?

a. 5.3 kips
b. 8.6 kips
c. 10.2 kips
d. 18.7 kips

8.61 What is the 2nd story shear?

a. 10.2 kips
b. 18.7 kips
c. 24.0 kips
d. None of the above

8.62 What is the diaphragm design force at the 3rd floor level (assume all $w_{px} = w_x$)?

a. 5.3 kips
b. 8.6 kips
c. 10.3 kips
d. 18.7 kips

9.1 For wood structural panel horizontal diaphragms, what is the minimum sheet dimension at boundaries with blocking omitted?

a. 12"
b. 18"
c. 24"
d. 48"

9.2 What is the maximum length-width (i.e., span-depth) ratio for an unblocked wood structural panel horizontal diaphragm?

a. 2:1
b. 3:1
c. 3½:1
d. 4:1

9.7 What is the maximum unit roof shear at Allowable Stress Design (ASD) force level?

a. 240 plf
b. 400 plf
c. 480 plf
d. 800 plf

9.8 What is the maximum drag force at Allowable Stress Design (ASD) force level?

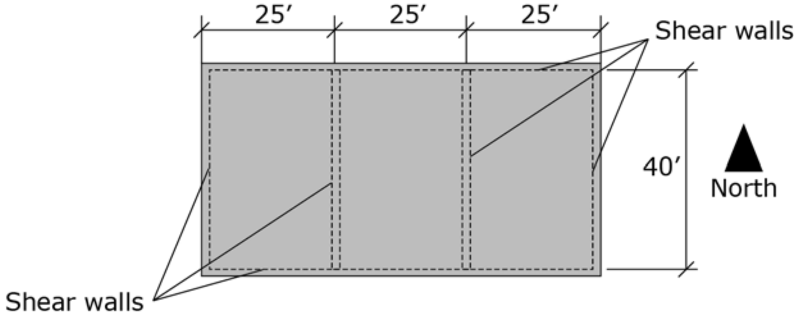
a. 2.4 kips
b. 4.8 kips
c. 6.0 kips
d. 9.6 kips

9.9 What is the hold-down force at Allowable Stress Design (ASD) force level assuming a 10 foot wall height, 30 foot shear wall width, $\rho = 1.0$, and neglecting the wall (and tributary roof) weight?

a. 2.0 kips
b. 4.0 kips
c. 8.0 kips
d. 12.0 kips

9.10 Given a seismic base shear $V = C_S \cdot W = 0.196 \cdot W$ at Strength Design (SD) force level. For the single story flexible roof diaphragm plan below, find the unit diaphragm shear at Allowable Stress Design (ASD) force level, for East-West loads. Roof DL = 25 psf, wall DL = 15 psf & 12 foot wall heights.

a. 50 plf
b. 80 plf
c. 155 plf
d. 170 plf



Plan

9.11 The contractor of a one-story wood frame commercial building project is proposing to substitute 15/32″ Structural I wood structural panel sheathing for the shear walls. The approved plans call for 3/8″ rated wood structural panel sheathing with 8d common (2½″ x 0.131″) at 2″ o.c. edge nailing. As the project engineer, which of the following nail size and edge nail spacing would provide the lowest acceptable allowable unit shear value?

a. 10d common at 6″ o.c.
b. 10d common at 4″ o.c.
c. 10d common at 3″ o.c.
d. 10d common at 2″ o.c.

9.21 What is the purpose of the subdiaphragms in a building with reinforced masonry (or concrete) shear walls?

a. Transfer out-of-plane wall anchorage forces into the roof diaphragm
b. Transfer in-plane diaphragm unit shears into the shear walls
c. Transfer in-plane diaphragm unit shears into the collector
d. All of the above

9.22 What is the maximum length-width (i.e., span-depth) ratio for a blocked wood structural panel horizontal diaphragm?

a. 2:1
b. 3:1
c. 3.5:1
d. 4:1

9.23 What is the maximum length-to-width (i.e., span-depth) ratio for a wood structural panel subdiaphragm?

a. 2:1
b. 2.5:1
c. 3:1
d. 4:1

9.24 What is the maximum height-width ratio of a blocked wood structural panel shear wall where the unit shear values of *IBC Table 2306.3* may be used without any adjustment (i.e., need not be reduced) when resisting seismic forces?

a. 3.5:1
b. 3:1
c. 2.5:1
d. 2:1

9.25 Given a wood structural panel shear wall with a height-width ratio of 3:1, what reduction factor would need to be applied to the unit shear values of *IBC Table 2306.3* when resisting seismic forces?

a. 0.82
b. 0.75
c. 0.67
d. 0.33

Given a two-story Bearing Wall System building with special reinforced masonry shear walls, assigned to Seismic Design Category D (SDC = D), and with blocked wood structural panel (flexible) diaphragms at the second floor and roof levels. Answer questions 9.26 through 9.27 below.

9.26 What *Response Modification Coefficient* (*R*) is appropriate for determining the seismic base shear?

a. 2
b. 3½
c. 5
d. 5½

14.5 A City's municipal water supply pipe is proposed to cross an active strike-slip fault. Which of the following combinations is least likely to result in damage to this lifeline when subjected to a localized fault rupture?

a. Pipe above ground oriented at 45 degrees to the fault
b. Pipe below ground oriented at 45 degrees to the fault
c. Pipe above ground oriented at 90 degrees to the fault
d. Pipe below ground oriented at 90 degrees to the fault

14.6 In order for liquefaction to occur during an earthquake, which of the following conditions are required to be present?

a. High groundwater table
b. Granular soils (e.g., sand, silty sand, sandy silt, etc.)
c. Low density in the granular soils
d. All of the above

15.1 Which of the following projects can a California licensed Civil Engineer design and be in responsible charge?

I. Concrete culvert under a freeway
II. Public school building
III. Vehicle bridge

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

15.2 A California licensed Civil Engineer has experience in bridge design only. Which of the following is she/he able to design?

I. A hospital under the supervision of a licensed Civil Engineer with hospital design experience
II. A building under the supervision of a licensed Structural Engineer
III. A vehicle bridge between two buildings

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

15.3 Given a two-story wood frame single family dwelling entirely of "conventional construction", who is allowed to prepare the plans (and specifications)?

I. Architect
II. Civil Engineer
III. Non-registered person

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

Problem	Answer	Reference / Solution
1.1	d	Earthquake design applies to all structures such as buildings, highway bridges, railroad bridges, dams, etc. The *International Building Code* & *ASCE 7-05* apply to buildings, "nonbuilding" structures, etc. ∴ All structures ←
1.2	b	p. 1-1, Figure 1.1 The *epicenter* is the point on the Earth's surface directly above the hypocenter. ∴ epicenter ←
1.3	b	p. 1-1, Figure 1.1 The place in the Earth's crust where this energy release occurs is known as the *hypocenter* (or focus). ∴ hypocenter ←
1.4	b	p. 1-1, Nature of earthquakes Often times, a major earthquake is preceded by smaller earthquakes known as *foreshocks* … ∴ foreshocks ←
1.5	a	p. 1-2, Fault types Fault movement may occur suddenly, or as slow continuous (or intermittent) movement without noticeable earthquakes known as *fault creep*. ∴ fault creep ←
1.6	d	p. 1-2, Fault types The San Andreas fault is a right-lateral fault more than 600 miles long. ∴ right-lateral ←
1.7	c	p. 1-2, Seismic sea waves *Seismic sea waves* (or *Tsunami's*) occur when a vertical fault movement occurs on the ocean floor (i.e., normal fault or reverse fault). ∴ I & II ←
1.8	d	p. 1-3, Seismic waves & Figure 1.3 Shear waves (*S-waves*) are most effective in damaging structures near the epicenter … and therefore, most responsible for the strong ground motion portion of an earthquake. ∴ Shear waves ←
1.9	b	Ground accelerations are the cause of seismic forces in a structure. ∴ ground acceleration ←
1.10	c	p. 1-5, Earthquake intensity Modified Mercalli (Intensity) scale ←

Problem	Answer	Reference / Solution
3.40	c	1-29 & *2009 IBC* p. 343, *§1613.5.6 & Tables 1613.5.6(1) & 1613.5.6(2)* Occupancy Category II – *IBC Table 1604.5* (p. 307) for apartment building $S_1 = 0.20 < 0.75$ → ∴ use *Tables 1613.5.6(1) & (2)* to determine SDC $S_{DS} = 0.41$ & OC = II → *Table 1613.5.6(1)* → SDC = C $S_{D1} = 0.20$ & OC = II → *Table 1613.5.6(2)* → SDC = D ← governs ∴ use Seismic Design Category D, SDC = D ←
3.41	d	1-26 & *ASCE 7-05* p. 128, *Tables 12.2-1 & 12.6-1* SDC determines the permissible lateral analysis procedure, building height limit, and seismic detailing requirements of the SFRS ∴ all of the above ←
3.42	c	1-26, Table 3.1 Seismic Design Category C = Moderate seismic hazard level ←
3.43	d	p. 1-29 & *2009 IBC* p. 343, *§1613.5.6* a. Hospital (Group I-2) → *IBC Table 1604.5* → OC = IV b. Single-family residence (Group R-3) → *IBC Table 1604.5* → OC = II c. County jail (Group I-3) → *IBC Table 1604.5* → OC = III Seismic Design Category F applies only to Occupancy Category IV structures (i.e., essential facilities, etc.). SDC = F does not apply to Occupancy Category I, II or III structures … ∴ Both b & c ←
3.44	d	p. 1-20 & *2009 IBC* p. 306 & 307, *§1604.5.1 & Table 1604.5* Where a building or structure is occupied by two or more occupancies not included in the same *Occupancy Category*, it shall be assigned the classification of the highest *Occupancy Category* corresponding to the various occupancies. Office building → *IBC Table 1604.5* → OC = II Fire station → *IBC Table 1604.5* → OC = IV (governs) ∴ use Occupancy Category IV ←
3.45	b	*2009 IBC* p. 353 & 355, *Figures 1613.5(3) & 1613.5(4)* At 36°00′00″ Latitude and -120°00′00″ Longitude … *Figure 1613.5(3)* → $S_S \approx 1.75$ *Figure 1613.5(4)* → $S_1 \approx 0.60$
3.46	b	*2009 IBC* p. 352 & 354, *Figures 1613.5(3) & 1613.5(4)* At 39°00′00″ Latitude and -123°00′00″ Longitude … *Figure 1613.5(3)* → $S_S \approx 1.55$ *Figure 1613.5(4)* → $S_1 \approx 0.70$
3.47	b	*2009 IBC* p. 348 to 365, *Figures 1613.5(1)* to *1613.5(14)* Site Class B ←

Problem	Answer	Reference / Solution
		Or using Table C1 (p. 5-16) → Dual Systems & $h_n = 35'$ → $T_a = 0.29$ sec ∴ T_a = <u>0.29 second</u> ←
4.23	a	p. 1-43 & *ASCE 7-05* p. 128, *Table 12.6-1* $3.5 \cdot T_S = 3.5 \cdot (0.6) = 2.10$ seconds SDC = D & $T = 2.4$ seconds $> 3.5 \cdot T_S$ → *Table 12.6-1* → ELF procedure is NP (not permitted). All others allow use of the ELF procedure. ∴ <u>A regular structure with $T = 2.4$ seconds</u> ←
4.24	c	*ASCE 7-05* p. 120, *Table 12.2-1, item B.3* Steel SCBF's, R = <u>6</u> ←
4.25	c	*ASCE 7-05* p. 120, *Table 12.2-1, item B.3* Ω_0 = <u>2</u> ←
4.26	b	*ASCE 7-05* p. 120, *Table 12.2-1, item B.3* H = <u>160 feet</u> ←
4.27	d	p. 1-49 & *ASCE 7-05* p. 129, *§12.8.2.1* $T_a = C_t (h_n)^x$ *ASCE 7 (12.8-7)* $C_t = 0.02$ & $x = 0.75$ → *ASCE 7* p. 129, *Table 12.8-2* (all others) Steel SCBF $T_a = 0.02\ (140')^{0.75} = 0.81$ sec Or using Table C1 (p. 5-16) → CBF & $h_n = 140'$ → $T_a = 0.81$ sec ∴ T_a = <u>0.8 second</u> ←
4.28	e	p. 1-37 & *ASCE 7-05* p. 120 & 121, *Table 12.2-1* R is proportional to ductility (i.e., larger R = more ductile) Steel SCBF → *Table 12.2-1, item B.3 - R* = 6 Light-framed WSP shear walls → *Table 12.2-1, item A.13/B.23 - R* = 6½ / 7 Reinforced concrete SMF → *Table 12.2-1, item C.5 - R* = 8 Steel EBF w/ steel SMF (Dual System) → *Table 12.2-1, item D.1 - R* = 8 ∴ <u>Both c & d</u> ←
4.29	d	p. 1-37, ductility is <u>not</u> related to flexibility or stiffness. ∴ <u>None of the above</u> ←
4.30	a	p. 1-37, Figure 4.4 $K = V / \Delta$ → Largest K = "steepest" elastic curve (i.e., straightline portion) ∴ Shear Wall A is the <u>most</u> stiff ←
4.31	d	p. 1-37, Figure 4.4 $K = V / \Delta$ → Lowest K = "flattest" elastic curve (i.e., straightline portion) ∴ Shear Wall D is the <u>least</u> stiff ←
4.32	b	p. 1-37, Figure 4.4 ∴ Shear Wall B would be the <u>most</u> ductile ←

Problem	Answer	Reference / Solution
4.86	d	p. 1-57, Building Separation & *IBC* p. 345, *§1613.6.7* From Problem 4.85 - Structure 1: At Level 7 (roof) - $\delta_{M1} = C_d \cdot \delta_{max} / I = (5\tfrac{1}{2})(3.5'') / (1.00) = 19.25''$ Where a structure adjoins a property line (not common to a public way), the structure shall be set back from the property line by not less than δ_M Structure 1 setback from property line, $\delta_{M1} = 19.25''$ ∴ 19 inches ←
4.87	b	p. 1-57, Building Separation & *IBC* p. 345, *§1613.6.7* From Problem 4.85 - Structure 2: At Level 4 (roof) - $\delta_{M2} = C_d \cdot \delta_{max} / I = (4)(1.4'') / (1.00) = 5.60''$ Where a structure adjoins a property line (not common to a public way), the structure shall be set back from the property line by not less than δ_M Structure 2 setback from property line, $\delta_{M2} = 5.60''$ ∴ 6 inches ←
4.88	a	p. 1-10, Natural Period $T = 2\pi\sqrt{W / K \cdot g}$ Increase in stiffness (K) will result in a decrease in structure period (T). While a decrease in period can result in an increase in base shear (V) and lateral forces at each level (F_x), the increase in overall stiffness should more than compensate the increased force level and result in a decrease in story drifts. ∴ Decrease in period and decrease in story drifts ←
4.89	a	p. 1-36 & 37, Response Modification Coefficient - *R* The *R* coefficient is representative of the inherent overstrength and global ductility of a seismic-force-resisting system (SFRS). ∴ The higher *R* structure has greater ductility ←
4.90	b	p. 1-47 & *ASCE 7-05* p. 129, *§12.8.1.1* Office Building = Occupancy Category II – *IBC* p. 307, *Table 1604.5* $I = 1.0$ – *ASCE 7-05* p. 116, *Table 11.5-1* $R = 5$ – *ASCE 7-05* p. 120, *Table 12.2-1, item A.7* $T_s = S_{D1}/S_{DS} = (0.30) / (0.75) = 0.40$ second $T_a = 0.21$ second $< T_s \rightarrow \therefore$ *ASCE 7 (12.8-2)* governs for C_s $C_S = \dfrac{S_{DS}}{(R/I)} = \dfrac{0.75}{(5/1.0)} = 0.15$ *ASCE 7 (12.8-2)* $V = C_s \cdot W$ *ASCE 7 (12.8-1)* $= 0.150 \cdot W$ ←

Problem	Answer	Reference / Solution
		Diaphragm 2 - maximum $CF_A = w_s \cdot L_2^2 / 8d = (0.15\text{ klf})(20')^2 / 8\,(50') = 0.15$ kips ∴ maximum CF_A = 2.4 kips ←
8.53	d	p. 1-103, Chord force $M_w = M_x = \left(\frac{w_s \cdot L}{2}\right) \cdot x - \frac{w_s \cdot x^2}{2}$ $= [(0.30\text{ klf})(50') / 2] \cdot 10' - (0.30\text{ klf})(10')^2 / 2 = 60.0$ kip-ft $CF_w = CF_x = M_w / d = (60.0\text{ kip-ft}) / 100' = 0.6$ kips Line 2 is not a boundary member for longitudinal direction seismic forces, so there will be no chord force on this line: $CF_y = 0$ kips From Problem 8.52, $CF_z = w_s \cdot L_1^2 / 8d = 2.4$ kips (governs) ∴ the maximum chord force occurs at “z” ←
8.54	a	p. 1-104, Drag force From Problem 8.48, unit roof shear $v_1 = 0.12$ klf Drag force at “w” & “x”, $F_d = (\text{roof } v_1)(10') = (0.12\text{ klf})(10') = 1.2$ kips ←
8.55	c	p. 1-104, Drag force From Problem 8.49, unit roof shear $v_2 = 0.15$ klf Drag force at “y”, $F_d = (\text{roof } v_2)(35') = (0.15\text{ klf})(35') = 5.25$ kips ←
8.56	b	p. 1-104, Drag force From Problem 8.51, unit roof shear $v_A = 0.075$ klf Drag force at “z”, $F_d = (\text{roof } v_A)(20' + 40') = (0.075\text{ klf})(60') = 4.5$ kips ←
8.57	c	p. 1-104, Drag force From Problems 8.54 to 8.56 - Drag force at “w” & “x”, $F_d = 1.2$ kips Drag force at “y”, $F_d = 5.25$ kips (governs) Drag force at “z”, $F_d = (\text{roof } v_A)(20' + 40') = (0.075\text{ klf})(60') = 4.5$ kips ∴ the maximum drag force occurs at “y” ←
8.58	a	p. 1-104 From Problem 8.45, $V_1 = 6.0$ kips Unit wall shear $v_1 = V_1 / \sum b = (6.0\text{ kips}) / (30') = 0.20\text{ klf} = 200$ plf ←
8.59	d	p. 1-104 From Problem 8.46, $V_2 = V / 2 = 7.5$ kips Unit wall shear $v_2 = V_2 / \sum b = (7.5\text{ kips}) / (15') = 0.50\text{ klf} = 500$ plf ←
8.60	a	p. 1-51, Vertical distribution of seismic forces & *ASCE 7-05* p. 130, *§12.8.3* 1st floor = Base 2nd floor = Level 1 (i.e., first level above the Base) *(continued)*

Problem	**Answer**	**Reference / Solution**
9.6	b	p. 1-122, Table 9.2 & *SDPWS Table 4.3.4* $h/b \le 3.5$ maximum (requires reduction factor $2b/h$ for seismic) ∴ minimum $b = (h/3.5) = 10' / 3.5 = 2.86' \approx 2'\text{-}10''$ ←
9.7	a	p. 1-121, Wood structural panel diaphragms $V_{max} = w \cdot L / 2 = V / 2 = 33.6 \text{ kips} / 2 = 16.8 \text{ kips}$ for ASD, roof $v = (0.7 \cdot V_{max}) / d = 0.7(16.8 \text{ kips}) / 50' = 235 \text{ plf} \approx 240 \text{ plf}$ ←
9.8	b	p. 1-104, 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) – for ASD, max F_d = roof $v(20') = (240 \text{ plf})(20') = 4{,}800 \text{ lbs} = 4.8 \text{ kips}$ ←
9.9	b	p. 1-130, Shear wall overturning / Hold-downs $\rho = 1.0$ (given) $V_1 = V_2 = V / 2 = 33.6 \text{ kips} / 2 = 16.8 \text{ kips}$ $T = \dfrac{-0.7\rho(V_1 \cdot h)}{b} = \dfrac{-0.7(1.0)(16.8)(10')}{30'} = -3.92 \text{ kips}$ ∴ for ASD, uplift $T = 4.0$ kips ←
9.10	b	p. 1-121, Wood structural panel diaphragms $V = C_S \cdot W = 0.196 \cdot W$ (given) For a single-story building – $w_s = f_{p1} = F_{p1} / L = C_S \cdot w_{p1}$ East-West: $w_s = 0.196\,[(25 \text{ psf})(75') + (15 \text{ psf})(12'/2) \cdot 4 \text{ walls}] = 438 \text{ plf}$ $V_{max} = w_s \cdot L / 2 = (438 \text{ plf})(40') / 2 = 8{,}760 \text{ lbs}$ for ASD, roof $v = (0.7 \cdot V_{max}) / d = 0.7 \cdot (8{,}760 \text{ lbs}) / 75' = 82 \text{ plf} \approx 80 \text{ plf}$ ←
9.11	c	*2009 IBC* p. 474, *Table 2306.3* 3/8″ rated sheathing w/ 8d common (2½″ x 0.131″) @ 2″ o.c. = 530 plf 15/32″ Structural I w/ 10d common @ 6″ o.c. = 340 plf < 530 plf **NG!** 15/32″ Structural I w/ 10d common @ 4″ o.c. = 510 plf < 530 plf **NG!** 15/32″ Structural I w/ 10d common @ 3″ o.c. = 665 plf > 530 plf **OK** 15/32″ Structural I w/ 10d common @ 2″ o.c. = 870 plf >> 530 plf **OK** ∴ use 15/32″ Structural I w/ 10d common @ 3″ o.c. = 665 plf > 530 plf ←
9.12	d	*2009 IBC* p. 470, *Table 2306.2.1(1)* Load parallel to continuous panel joints = Case 3 (weak direction) 15/32″ sheathing w/ 8d @ 6″ o.c. unblocked = 180 plf < 275 plf **NG!** 15/32″ sheathing w/ 10d @ 6″ o.c. unblocked = 190 plf < 275 plf **NG!** 15/32″ sheathing w/ 8d @ 6″ o.c. blocked = 270 plf < 275 plf **NG!** 15/32″ sheathing w/ 10d @ 6″ o.c. blocked = 290 plf > 275 plf **OK** ∴ use 15/32″ sheathing w/ 10d common @ 6″ o.c. = 290 plf > 275 plf ←
9.13	d	p. 1-100, Flexible diaphragm analysis $V = C_S \cdot W$ *(continued)*

Problem	Answer	Reference / Solution
9.32	b	p. 1-126 & *2009 IBC* p. 474, *Table 2306.3* $h/b = 12'/4.25' = 2.82{:}1 > 2{:}1$ … unit shear values will require a reduction factor for seismic reduction factor = $2b/h = 2(4.25'/12') = 0.71$ 15/32″ Structural I w/ 10d common at 3″ o.c. → *Table 2306.3* → ∴ allowable unit wall shear = (0.71)(665 plf) = 470 plf ←
9.33	b	p. 1-126 & *2009 IBC* p. 474, *Table 2306.3* $h/w = 12'/3.42' = 3.5{:}1 > 2{:}1$ … unit shear values will require a reduction factor for seismic reduction factor = $2b/h = 2(3.42'/12') = 0.57$ 15/32″ Structural I w/ 10d common at 3″ o.c. → *Table 2306.3* → ∴ allowable unit wall shear = (0.57)(665 plf) = 380 plf ←
9.34	d	p. 1-115 & *2009 IBC* p. 451, *§2301.2 - items 1, 2 & 3* The design of structural elements or systems constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods: *Allowable Stress Design* (ASD), *Load and Resistance Factor Design* (LRFD), *Conventional Light-Frame Construction* … etc. ∴ I, II & III ←
9.35	c	p. 1-140 & *ASCE 7-05* p. 133, *§12.11.2.2* SDC = C, D, E or F - subdiaphragms are to be designed for the … wall anchorage force per *ASCE 7 – §12.11.2* ←
9.36	d	p. 1-123 & *SDPWS §4.3.5* The *SDPWS* provides for two methods for designing shear walls with openings: force transfer around openings & perforated shear walls. ∴ Both b & c ←
9.37	a	p. 1-122, Table 9.2 & *2009 IBC* p. 469, *§2306.5 & §2306.7* I. Wood structural panels - permitted in all SDC's II. Gypsum wallboard, etc. - Not permitted for seismic in SDC = E & F III. Particle board (blocked) - Not permitted for seismic in SDC = D, E & F ∴ I ←
9.38	c	p. 1-138, Figure 9.15 In-plane overturning load path … hold-down post (D) to hold-down connector (A) to hold-down anchor bolt (E) to footing (F). ∴ D – A – E – F ←
9.39	a	p. 1-138, Figure 9.15 In-plane shear load path … shear wall sheathing to transfer the in-plane shear to the sill plate (C) to sill bolts (B) to footing (F). ∴ C – B – F ←

Problem	Answer	Reference / Solution
11.9	d	p. 1-157, Types of Work & *2009 IBC* p. 368+, *§1704* Special Inspections *IBC Table 1704.7, item 4* – required for fill placement and compaction *IBC §1704.12* – required for sprayed fire-resistant materials *IBC §1704.16* – required for smoke control systems ∴ typically NOT required for nailing of wood structural members ←
11.10	a	*2009 IBC* p. 383, *§1710.1* – General At the conclusion of the work included in the permit, the structural observer shall submit to the *building official* a written statement that the site visits have been made and report any deficiencies … ∴ Building official ←
12.1	a	p. 1-166 The *California Building Code (CBC)* is also known as the *California Code of Regulations (CCR), Title 24, Part 2.* ∴ California Building Code (CBC) ←
12.2	d	p. 1-167 The *CEBC* provides minimum seismic strengthening provisions for existing *unreinforced masonry* (URM) bearing wall buildings that undergo a change of use (or occupancy), alteration, or repair. ∴ existing unreinforced masonry (URM) buildings ←
12.3	a	p. 1-167 The *CHBC* provides alternative building regulations and building standards for the rehabilitation, etc … of buildings (or structures) designated as *historic buildings*. ∴ historic buildings ←
12.4	a	p. 1-167 The provisions of the *CRC* shall apply to the construction, etc. … of detached one- and two-family dwellings and townhouses not more than three stories … in California. ∴ *California Residential Code* (*CRC*) ←
13.1	c	p. 1-177 For pre-1971 reinforced concrete bridge columns in California, the most common retrofit is to encase the column with a *steel jacket* (i.e., steel casings) … ∴ steel jacket ←
13.2	b	p. 1-169 "X" cracking (shear cracking) occurs during in-plane cyclic loading on walls. It is most likely to occur on an unreinforced masonry (URM) wall since reinforcement is not present to prevent cracks from "opening up" during lateral loading. ∴ In-plane loading on an unreinforced masonry wall ←

Problem	Answer	Reference / Solution
13.22	a	p. 1-138, Shear Transfer – Figure 9.15 (similar) In-plane diaphragm shear load path … roof diaphragm sheathing transfers through boundary nailing (A) to wood ledger (B) to ledger bolting (C) to concrete shear wall (D). ∴ A – B – C – D ←
13.23	a	p. 1-169, Unreinforced Masonry (URM) Buildings Very stiff, very brittle, and probably the most hazardous form of construction found in seismic regions of the United States ∴ Unreinforced masonry (URM) ←
13.24	d	p. 1-177, Retrofit of existing structures The most common retrofit is to encase the column with a *steel jacket* … to increase the confinement and to improve the flexural ductility and shear capacity of the column. ∴ all of the above ←
13.25	b	p. 1-177, Retrofit of existing structures Figure A lacks an obvious continuous load path for in-plane shear between the existing sill plate and the existing footing (i.e., sill bolts are missing) … therefore, sliding of the structure is likely in the event of moderate earthquake ground motion. ∴ Sliding failure between sill plate and footing ←
13.26	c	p. 1-177, Retrofit of existing structures Figure B demonstrates a potential weak-story failure due to the existing cripple wall studs without structrural sheathing to resist the in-plane shear (i.e., lap siding is not structural). ∴ Cripple wall faiure ←
13.27	a	p. 1-177, Retrofit of existing structures Per Problem 13.25 … sill bolts are missing. The most logical seimic retrofit would be to provide new sill bolts using post-installed adhesive anchors or mechanical (e.g., wedge) anchors. ∴ Add sill plate anchor bolts ←
13.28	b	p. 1-177, Retrofit of existing structures Per Problem 13.26 … cripple wall is weak. The most logical seimic retrofit would be to provide shear walls between the main floor framing and the existing footing. This is most easily accomplished by sheathing the inside face of the existing cripple studs with wood structural panels (WSP). ∴ Add WSP sheathing to the inside face of the cripple studs ←
14.1	d	p. 1-179, Liquefaction Soils most susceptible to liquefaction are saturated, loose & uniformly graded sands. ∴ I, II & III ←

IMPORTANCE FACTOR (*I*) – a factor assigned to each structure according to its Occupancy Category (I, II, III or IV).

INTERMEDIATE MOMENT FRAME (IMF) – a moment frame of reinforced concrete satisfying the detailing requirements of *ACI 318*, of structural steel satisfying the detailing requirements of *AISC Seismic - Part I* or of composite construction satisfying the requirements of *AISC Seismic - Part II*.

INVERTED PENDULUM-TYPE STRUCTURE – a structure which has a large portion of its mass concentrated near the top and has essentially a single degree of freedom (SDOF) in horizontal translation (usually T-shaped with a single column supporting the beams or framing at the top). A structure in which more than 50 percent of the structure's mass is concentrated at the top of a slender, cantilevered structure and in which stability of the mass at the top of the structure relies on rotational restraint to the top of the cantilevered element.

JOINT – the geometric volume common to intersecting members.

LATERAL-FORCE-RESISTING SYSTEM (LFRS) – is that part of the structural system designed to resist the *Design Seismic Forces* (or wind forces).

LATERAL LOAD – any horizontal load on a structure, including the load from wind (W) or earthquake (E).

LIGHT-FRAME CONSTRUCTION – a method of construction whose vertical and horizontal structural elements are primarily formed by a system of repetitive wood or light gage steel framing members or subassemblies of these members (e.g., trusses).

LIGHT-FRAMED WALL – a wall with wood or steel studs.

LIGHT-FRAMED WOOD SHEAR WALL – a wall constructed with wood studs and sheathed with material rated for shear resistance (e.g. wood structural panels, gypsum wallboard, plaster, etc.).

LIMIT STATE – a condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended function (*serviceability limit state*) or judged to be unsafe (*strength limit state*).

LIVE LOAD – the load produced by the use and occupancy of the building or other structure. Live loads do not include construction or environmental loads such as wind load (W), snow load (S), rain load (R), earthquake load (E), flood load (F) or dead load (D).

LOAD AND RESISTANCE FACTOR DESIGN (LRFD) – a method of proportioning structural members and their connections using load and resistance factors such that no applicable limit state is reached when the structure is subjected to appropriate load combinations. The term "LRFD" is used in the design of steel and wood structures.

example: (load) factored moment ≤ (resistance) factored moment strength

$$M_u \leq \phi M_n$$

LOAD EFFECTS – forces and deformations produced in structural members by the applied loads.

LOAD FACTOR – a factor that accounts for deviations of the actual load from the nominal load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

LOADS – forces or other actions that result from the weight of building materials, occupants and their possessions, environmental effects, differential movement and restrained dimensional changes. Permanent loads are those loads in which variations over time are rare or of small magnitude, such as dead loads. All other loads are variable loads (see also "Nominal loads").

MAJOR ORTHOGONAL HORIZONTAL DIRECTIONS – the orthogonal directions that overlay the majority of lateral force resisting elements.

MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION – the most severe earthquake effects considered by the applicable code (or standard).

Appendix B – Notations

A_x = torsional amplification factor at Level x; see *ASCE 7 – §12.8.4.3*

a_p = component amplification factor determined from *ASCE 7 – §13.3.1* & *Table 13.5-1* or *13.6-1*

C_d = deflection amplification factor as given in *ASCE 7 – Tables 12.2-1*, *15.4-1*, or *15.4-2*

C_S = seismic response coefficient determined in *ASCE 7 – §12.8.1.1* & *§19.3.1*

C_t = building period coefficient in *ASCE 7 – §12.8.2.1*

C_{vx} = vertical distribution factor as determined in *ASCE 7 – §12.8.3*

c = distance from the neutral axis of a flexural member to the fiber of maximum compressive strain

D = the effect of dead load on a structural element

D_p = relative seismic displacement that a component must be designed to accommodate as defined in *ASCE 7 – §13.3.2*

d_C = total thickness of cohesive soil layers in the top 100 ft; see *IBC §1613.5.5* or *ASCE 7 – §20.4.3*

d_i = thickness of any soil or rock layer *i* (between 0 and 100 ft); see *IBC §1613.5.5* or *ASCE 7 – §20.4.1*

d_S = total thickness of cohesionless soil layers in the top 100 ft; see *IBC §1613.5.5* or *ASCE 7 – §20.4.2*

E = combined seismic load effect of horizontal and vertical earthquake-induced forces on a structural element; see *ASCE 7 – §12.4.2*

E_h = effect of horizontal seismic forces as defined in *ASCE 7 – §12.4.2.1* or *§12.14.3.1.1*

E_m = maximum seismic load effect of horizontal and vertical earthquake-induced forces (including overstrength factor) on a structural element; see *ASCE 7 – §12.4.3*

E_{mh} = maximum effect of horizontal seismic forces including structural overstrength as defined in *ASCE 7 – §12.4.3.1* or *§12.14.3.2.1*

E_v = vertical seismic load effect as defined in *ASCE 7 – §12.4.2.2* or *§12.14.3.1.2*

F_a = acceleration based short-period site coefficient (at $T = 0.2$ second) defined in *IBC Table 1613.5.3(1)* or *ASCE 7 – §11.4.3*; see *ASCE 7 – §12.14.8.1* when using the Simplified Design procedures of *ASCE 7 – §12.14*

F_i, F_n, F_x = portion of the seismic base shear (V) induced at Level i, n, or x, respectively, as determined in *ASCE 7 – §12.8.3*

F_p = horizontal seismic force acting on a component of a structure as determined in *ASCE 7 – §13.3.1*

F_{px} = diaphragm design force at Level x per *ASCE 7 – §12.10.1.1* & equation *(12.10-1)*

F_v = velocity based long-period site coefficient (at $T = 1.0$ second) defined in *IBC Table 1613.5.3(2)* or *ASCE 7 – §11.4.3*

f'_c = specified compressive strength of concrete used in design (psi)

OTM_x = overturning moment at Level x

P_x = total unfactored vertical design load at and above Level x, for use in *ASCE 7 – §12.8.7*

PI = plasticity index of soil, *ASTM D4318*

Q_E = effect of horizontal seismic forces from V or F_p as specified in *ASCE 7 – §12.14.8.1*, *§12.14.7.5* & *§13.3.1*

R = relative rigidity of a concrete or masonry shear wall

R = response modification coefficient as given in *ASCE 7 – Tables 12.2-1*, *12.14-1*, *15.4-1*, or *15.4-2*

R_p = component response modification factor as defined in *ASCE 7 – §13.3.1*

RM = resisting moment

S = effect of snow load on a structural element

S_S = mapped MCE (5% damped) spectral response acceleration parameter at short periods as defined in *IBC §1613.5.1* or *ASCE 7 – §11.4.1*

S_1 = mapped MCE (5% damped) spectral response acceleration parameter at 1 second period as defined in *IBC §1613.5.1* or *ASCE 7 – §11.4.1*

S_{DS} = design (5% damped) spectral response acceleration parameter at short periods as defined in *IBC §1613.5.4* or *ASCE 7 – §11.4.4*

S_{D1} = design (5% damped) spectral response acceleration parameter at 1 second period as defined in *IBC §1613.5.4* or *ASCE 7 – §11.4.4*

S_{MS} = site class modified MCE (5% damped) spectral response acceleration parameter at short periods as defined in *IBC §1613.5.3* or *ASCE 7 – §11.4.3*

S_{M1} = site class modified MCE (5% damped) spectral response acceleration parameter at 1 second period as defined in *IBC §1613.5.3* or *ASCE 7 – §11.4.3*

s_u = undrained shear strength; see *IBC §1613.5.5* or *ASCE 7 – §20.4.3*

$\bar{s}_u$ = average undrained shear strength in top 100 feet; see *IBC §1613.5.5* or *ASCE 7 – §20.3.3* & *§20.4.3*, *ASTM D2166* or *ASTM D2850*

S_{ui} = undrained shear strength of any cohesive soil layer i (between 0 and 100 ft); see *IBC §1613.5.5* or *ASCE 7 – §20.4.3*

T = elastic fundamental period of vibration of the structure in the direction under consideration (seconds)

T_a = approximate fundamental period of the building as determined in *ASCE 7 – §12.8.2* (seconds)

T_L = long-period transition period as defined in *ASCE 7 – §11.4.5* (seconds)

T_p = fundamental period of the component and its attachment, see *ASCE 7 – §13.6.2* (seconds)

T_0 = control period equal to $0.2 S_{D1}/S_{DS}$

T_S = control period equal to S_{D1}/S_{DS}

V = seismic base shear; total seismic design lateral force or shear at the base

V_x = seismic design shear in story x as determined in *ASCE 7 – §12.8.4* or *§12.9.5*