Part 3 – Multiple Choice Problems

Part 4 – Multiple Choice Solutions

Part 5 – Appendices & Index

California Special Civil P.E. Seismic Principles Examination Statistics

Exam	% Passed	Cut off score	Total Score	Passing %
October 2000	39.4%	148	288	51%
April 2001	37.3%	121	268	45%
October 2001	40.3%	150	294	51%
April 2002	39.6%	138	276	50%
October 2002	44.2%	136	287	47%
April 2003	37.1%	155	300	52%
October 2003	40.4%	136	281	48%
April 2004	35.6%	125	263	48%
October 2004	38.5%	154	300	51%
April 2005	39.8%	159	292	54%
October 2005	44.8%	164	295	56%
April 2006	37.4%	152	300	51%
October 2006	37.2%	142	263	54%
April 2007	36.7%	156	292	53%
October 2007	39.9%	177	292	61%
April 2008	36.3%	153	295	52%
October 2008	36.6%	151	285	53%
April 2009	39.5%	25	50	50%
October 2009	39.2%	*Pass / Fail Only*		
April 2010	38.6%	*Pass / Fail Only*		
October 2010	38.7%	*Pass / Fail Only*		
April 2011	43.0%	*Pass / Fail Only*		
October 2011	35.3%	*Pass / Fail Only*		
April 2012	40.8%	*Pass / Fail Only*		
October 2012	41.0%	*Pass / Fail Only*		
April 2013	46.6%	*Pass / Fail Only*		
October 2013	44.6%	*Pass / Fail Only*		
Spring 2014	48.0%	*Pass / Fail Only*		
Fall 2014	41.1%	*Pass / Fail Only*		
Spring 2015	51.7%	*Pass / Fail Only*		
Fall 2015	41.1%	*Pass / Fail Only*		
Spring 2016	53.7%	*Pass / Fail Only*		
Fall 2016	43.5%	*Pass / Fail Only*		
Spring 2017	54.9%	*Pass / Fail Only*		
Fall 2017	43.9%	*Pass / Fail Only*		
2018 – Q2	41.5%	*Pass / Fail Only*		
2018 – Q3	43.9%	*Pass / Fail Only*		
2018 – Q4	43.3%	*Pass / Fail Only*		
2019 – Q1	47.7%	*Pass / Fail Only*		
2019 – Q2	50.6%	*Pass / Fail Only*		
2019 – Q3	47.7%	*Pass / Fail Only*		
2019 – Q4	48.7%	*Pass / Fail Only*		

Design Spectral Response Acceleration Parameters *IBC §1613.2.4*

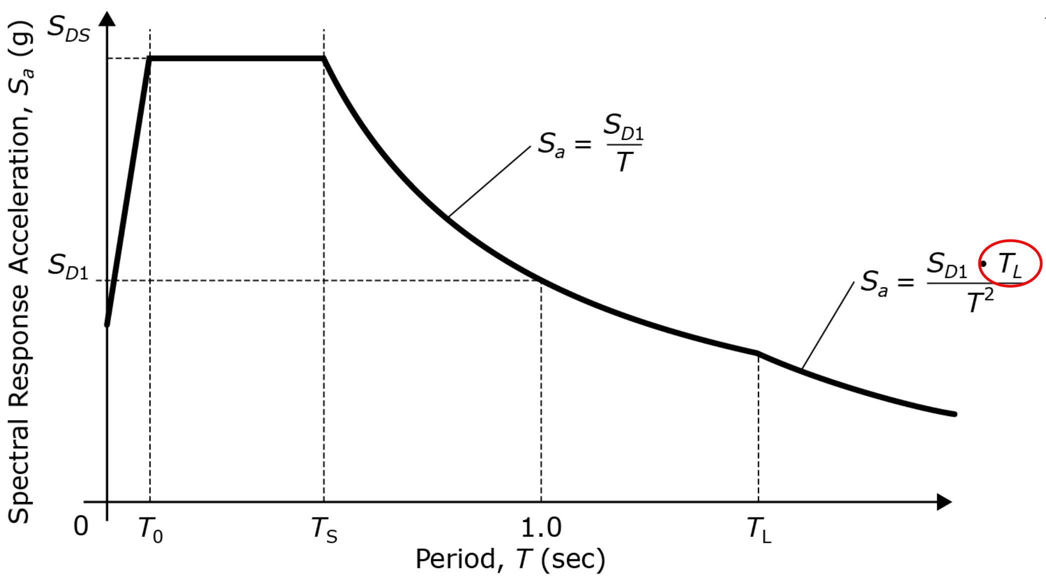
S_{DS} & S_{D1} represent the 5% damped design spectral response acceleration parameters at short periods and at 1-second period respectively and they are determined by the following equations:

$S_{DS} = 2/3\ S_{MS}$ *IBC (16-38)*

$S_{D1} = 2/3\ S_{M1}$ *IBC (16-39)*

NOTE: Table 3.1 (p. 1-35) and Table 3.2 (p. 1-36) are provided as short cut procedures in determining S_{DS} & S_{D1} respectively when S_S, S_1 & *site class* are known. These tables should not be used if F_a & F_v and/or S_{MS} & S_{M1} are required to be determined. Apply the footnotes as applicable including the use of *Site Class* D (default) when that *site class* is assumed (i.e., no geotechnical report).

Figure 3.1 – Design Response Spectrum (Ref. 9 - *ASCE 7 – Figure 11.4-1*)



where:

$$T_S = \frac{S_{D1}}{S_{DS}}$$

$$T_0 = 0.2T_S$$

T_L = per *ASCE 7-16 Figure 22-14*

Determination of Seismic Design Category, *SDC* *IBC §1613.2.5*

The *Seismic Design Category* (*SDC*) of a structure is used to determine the following:

- Permitted seismic force-resisting systems (SFRS)
- Building height limits
- Permitted lateral analysis procedures
- Restrictions on buildings with horizontal and/or vertical irregularities
- Seismic detailing requirements
- Requirements for nonstructural components

NOTE: The *Seismic Design Category* (*SDC*) for a structure is permitted to be determined in accordance with *IBC §1613.2.5* or *ASCE 7-16.* Where the alternative simplified design procedure of *ASCE 7-16* is used, the *Seismic Design Category* (*SDC*) shall be determined in accordance with *ASCE 7-16 – §12.14.*

▪ Inherent Torsional Moment, M_t

The *inherent* torsional moment is then equal to the story shear times the calculated eccentricity as follows:

<u>*X*-Direction</u>: $M_t = V_x\, e_y$

<u>*Y*-Direction</u>: $M_t = V_y\, e_x$

Accidental Torsion — *ASCE 7 – §12.8.4.2*

Where diaphragms are <u>not</u> flexible (i.e., rigid or semi-rigid), the distribution of lateral forces at each level shall consider the effect of the *inherent* torsional moment (M_t) … plus the *accidental* torsional moments (M_{ta}) caused by … the *accidental* eccentricities.

Where earthquake forces are applied concurrently in two orthogonal directions, the required *accidental* eccentricity need not be applied in both of the orthogonal directions at the same time but shall be applied in the direction that produces the greater effect.

> **<u>NOTE</u>:** *Accidental* torsion <u>shall be applied to all structures</u> for determination if a torsional irregularity <u>or</u> *extreme* torsional irregularity exists as specified in *ASCE 7-16 – Table 12.3-1*.

Accidental torsional moments (M_{ta}) <u>need not</u> be included when determining the seismic forces (E) in the <u>design</u> of the structure and in the determination of the <u>design</u> *story drift* in *ASCE 7-16 – §12.8.6* … <u>except for the following</u>:

1. Structures assigned to *SDC* = <u>B</u> with *extreme* torsional irregularity (Horizontal - Type 1b) per *ASCE 7-16 – Table 12.3-1*
2. Structures assigned to *SDC* = <u>C, D, E or F</u> with torsional irregularity <u>**or**</u> *extreme* torsional irregularity (Horizontal - Type 1a <u>or</u> 1b) per *ASCE 7-16 – Table 12.3-1*

▪ Accidental Eccentricity

The *accidental* eccentricity accounts for an assumed displacement of the *center of mass* (*CM*) each way from its calculated location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces.

$\therefore$ *accidental* eccentricity, $e = \pm\, 0.05\, L_{\perp}$

Then for each direction under consideration:

<u>*X*-Direction</u>: *accidental* $e_y = \pm\, 0.05\, L_y$

<u>*Y*-Direction</u>: *accidental* $e_x = \pm\, 0.05\, L_x$

▪ Accidental Torsional Moment, M_{ta}

The *accidental* torsional moment (M_{ta}) is then equal to the story shear times the *accidental* eccentricity as follows:

<u>*X*-Direction</u>: $M_{ta} = V_x\,(\pm\, 0.05\, L_y)$

<u>*Y*-Direction</u>: $M_{ta} = V_y\,(\pm\, 0.05\, L_x)$

Final Design Forces

for *SDC* = B, C, D, E or F structures with **NO** torsional or *extreme* torsional irregularity

↓

$$M_T = M_t$$

Design torsional moment

↓

$$F_x = V_x (R_x / \sum R_x) + M_T R\, d / \sum R\, d^2$$

$$F_y = V_y (R_y / \sum R_y) + M_T R\, d / \sum R\, d^2$$

Determine which eccentricity results in the maximum force to each vertical LFR element

F = *Direct* Shear + *Torsional* Shear

(**NOTE**: *torsional* shear can be positive or negative)

NOTE: If the *CM* and *CR* are inline with each other (in a particular direction under consideration), there will be no inherent eccentricity, no inherent torsional moment (M_t), and no design torsional moment (M_T) … therefore each vertical LFR element in that direction will only resist *direct* shear (no *torsional* shear).

OR

Final Design Forces

for *SDC* = C, D, E or F structures **with** torsional or *extreme* torsional irregularity

↓

$$A_x = (\delta_{max} / 1.2\, \delta_{avg})^2$$

Torsional amplification factor - A_x

↓

$$M_{T1} = M_t + A_x (M_{ta})$$

$$M_{T2} = M_t - A_x (M_{ta})$$

Design torsional moments considering torsional amplification factor - A_x

↓

$$F_x = V_x (R_x / \sum R_x) + M_T R\, d / \sum R\, d^2$$

$$F_y = V_y (R_y / \sum R_y) + M_T R\, d / \sum R\, d^2$$

Determine which eccentricity results in the maximum force to each vertical LFR element

F = *Direct* Shear + *Torsional* Shear

(**NOTE**: *torsional* shear can be positive or negative)

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*
- *AWC Wood Frame Construction Manual (WFCM)* – per *IBC §2309*
- *ICC 400* – for design and construction of log structures

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

General — *IBC §2305.1*

Structures using wood-frame *shear walls* or wood-frame *diaphragms* to resist wind, seismic or other lateral loads shall be designed and constructed in accordance with *ANSI/AWC Special Design Provisions for Wind and Seismic* (*SDPWS-2015*) and the applicable 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.

B.) Seismic Design Category, *SDC*

$S_1 = 0.71 < 0.75$ → therefore, use *IBC Table 1613.2.5(1)* & *Table 1613.2.5(2)* to determine *SDC*

$S_{DS} = 1.19$ & RC = II → *2018 IBC Table 1613.2.5(1)* → SDC = D

$S_{D1} = 0.38$ & RC = II → *2018 IBC Table 1613.2.5(2)* → SDC = D

∴ $\boxed{SDC = \text{D}}$

C.) Approximate Fundamental Period, T_a

$$T_a = C_t h_n^x \qquad \textit{ASCE 7 (12.8-7)}$$

Steel SMF → *ASCE 7-16 – Table 12.8-2*: Steel MRF – $C_t = 0.028$ & $x = 0.8$

$T_a = 0.028\,(h_n)^{0.8}$

$= 0.028\,(95 \text{ feet})^{0.8} = \boxed{1.07 \text{ second}}$

Or using Table C1 (Appendix C, p. 5-20) → Steel MRF & $h_n = 95$ feet → T_a = <u>1.07 second</u>

> **<u>NOTE</u>:** $T = T_a = 1.07$ second $> T_S = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.38}{1.19} = 0.32$ second → ∴ *ASCE 7 (12.8-2)* <u>will not</u> govern C_S

D.) Seismic Response Coefficient, C_S

$$C_S = \frac{S_{DS}}{(R/I_e)} = \frac{(1.19)}{(8/1.0)} = 0.149 \qquad \textit{ASCE 7 (12.8-2)}$$

C_S <u>need not</u> exceed the following,

$$C_S = \frac{S_{D1}}{T(R/I_e)} = \frac{(0.38)}{1.07(8/1.0)} = 0.044 \qquad \textit{ASCE 7 (12.8-3)}$$

> **<u>NOTE</u>:** *ASCE 7 (12.8-4)* is <u>not</u> applicable since $T = 1.07$ seconds << $T_L = 8$ seconds … … BUT need to check *ASCE 7 (12.8-5)* <u>and</u> *(12.8-6)* since $S_1 = 0.71 > 0.6$

C_S <u>shall not</u> be less than the following,

$$C_S = 0.044 S_{DS} I_e = 0.044(1.19)(1.0) = \underline{0.052} \quad \leftarrow \underline{\text{governs}} \qquad \textit{ASCE 7 (12.8-5)}$$

≥ 0.01 minimum

C_S <u>shall not</u> be less than the following,

$$C_S = \frac{0.5 S_1}{(R/I_e)} = \frac{0.5(0.71)}{(8/1.0)} = 0.044 \qquad \textit{ASCE 7 (12.8-6)}$$

∴ $C_S = \boxed{0.052}$

E.) Seismic Base Shear, *V*

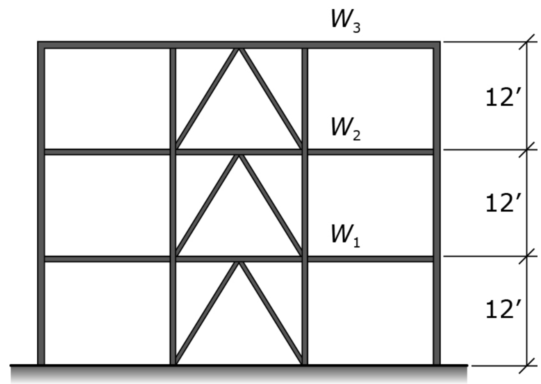
$$V = C_S W \qquad \textit{ASCE 7 (12.8-1)}$$

$= 0.052\,(3500 \text{ kips}) = \boxed{182 \text{ kips}}$

Problem #5

Given:

- 3-story Office building
- *Site Class* C (very dense soil) per Geotechnical Report
- T_L = 16 seconds per *ASCE 7-16 – Figure 22-14*
- Mapped acceleration parameters –
  - $S_S = 1.22$
  - $S_1 = 0.48$
- Building Frame Systems –
  - Steel special concentrically braced frames (SCBF)
- All story heights, h_s = 12 feet
- Effective seismic weights –
  - Level 1 & 2: $w_1 = w_2 = 100$ kips
  - Level 3 (roof): $w_3 = 80$ kips



Elevation

Find: Using the Equivalent Lateral Force (ELF) procedure –

A.) Design spectral response acceleration parameters, S_{DS} & S_{D1}

B.) Seismic Design Category, *SDC*

C.) Approximate fundamental period, T_a

D.) Seismic response coefficient, C_S

E.) Seismic base shear, V

Solution:

Office building = *Risk Category* II → *2018 IBC Table 1604.5*

$I_e = 1.0$ → *ASCE 7-16 – Table 1.5-2*: *Risk Category* II

$F_a = 1.2$ → *2018 IBC Table 1613.2.3(1)*: Site Class C & $S_S = 1.22$

$F_v = 1.5$ → *2018 IBC Table 1613.2.3(2)*: Site Class C & $S_1 = 0.48$

$R = 6$ → *ASCE 7-16 – Table 12.2-1, item B.2*: Building Frame Systems - Steel SCBF

Building height, h_n = 3 stories (12 feet/story) = 36 feet

Total effective seismic weight, $W = w_1 + w_2 + w_3 = 100 + 100 + 80 =$ <u>280 kips</u>

A.) Design Spectral Response Acceleration Parameters, S_{DS} & S_{D1}

$S_{MS} = F_a\, S_S$ *IBC (16-36)*

$= 1.2\,(1.22) = 1.46$

$S_{DS} = 2/3\; S_{MS}$ *IBC (16-38)*

$= 2/3\,(1.46) = \boxed{0.98}$

$S_{M1} = F_v\, S_1$ *IBC (16-37)*

$= 1.5\,(0.48) = 0.72$

$S_{D1} = 2/3\, S_{M1}$ *IBC (16-39)*

$= 2/3\,(0.72) = \boxed{0.48}$

> **NOTE:** Alternatively, S_{DS} & S_{D1} can be quickly determined using Tables 3.2 & 3.3 (p. 1-35 & 36):
>
> $S_S = 1.22$ & *Site Class* C $\rightarrow$ Table 3.2 $\rightarrow$ $S_{DS} = \underline{0.98}$ … by interpolation
>
> $S_1 = 0.48$ & *Site Class* C $\rightarrow$ Table 3.3 $\rightarrow$ $S_{D1} = \underline{0.48}$

B.) Seismic Design Category, *SDC*

$S_1 = 0.48 < 0.75$ $\rightarrow$ therefore, use *IBC Table 1613.2.5(1)* & *Table 1613.2.5(2)* to determine *SDC*

$S_{DS} = 0.98$ & $RC =$ II $\rightarrow$ *2018 IBC Table 1613.2.5(1)* $\rightarrow$ $SDC =$ D

$S_{D1} = 0.48$ & $RC =$ II $\rightarrow$ *2018 IBC Table 1613.2.5(2)* $\rightarrow$ $SDC =$ D

$\therefore \boxed{SDC = \text{D}}$

C.) Approximate Fundamental Period, T_a

$$T_a = C_t h_n^x$$ *ASCE 7 (12.8-7)*

Steel SCBF $\rightarrow$ *ASCE 7-16 – Table 12.8-2*: all other structural systems – $C_t = 0.02$ & $x = 0.75$

$T_a = 0.02\,(h_n)^{0.75}$

$= 0.02\,(36\text{ feet})^{0.75} = \boxed{0.29\text{ second}}$

Or using Table C1 (Appendix C, p. 5-20) $\rightarrow$ CBF & $h_n = 36$ feet $\rightarrow$ $T_a = \underline{0.29\text{ second}}$

> **NOTE:** $T = T_a = 0.29$ second $< T_S = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.48}{0.98} = 0.49$ second $\rightarrow$ $\therefore$ *ASCE 7 (12.8-2)* <u>will</u> govern C_S

D.) Seismic Response Coefficient, C_S

$$C_S = \frac{S_{DS}}{(R/I_e)} = \frac{(0.98)}{(6/1.0)} = \underline{0.163} \qquad \leftarrow \underline{\text{governs}}$$ *ASCE 7 (12.8-2)*

C_S <u>need not</u> exceed the following,

$$C_S = \frac{S_{D1}}{T(R/I_e)} = \frac{(0.48)}{0.29(6/1.0)} = 0.276$$ *ASCE 7 (12.8-3)*

> **NOTE:** *ASCE 7 (12.8-4)*, *(12.8-5)*, and *(12.8-6)* are <u>not</u> applicable since $T < T_S$ (and $T << T_L$)

$\therefore C_S = \boxed{0.163}$

E.) Seismic Base Shear, *V*

$V = C_S\, W$ *ASCE 7 (12.8-1)*

$= 0.163\,(280\text{ kips}) = \boxed{45.6\text{ kips}}$

Problem #8

Given:

- 5-story Emergency Operations Center
- Mapped acceleration parameters –
  - $S_S = 1.93$
  - $S_1 = 0.77$
- $T_L = 12$ seconds per *ASCE 7-16 – Figure 22-14*
- *Site Class* E per Geotechnical report
- Dual System –
  - Steel eccentric braced frames (EBF) with steel SMF
- All story heights, $h_s = 12$ feet
- Total effective seismic weight, $W = 24{,}000$ kips

Find: Using the Equivalent Lateral Force (ELF) procedure –

A.) Design spectral response acceleration parameters, S_{DS} & S_{D1}

B.) Seismic Design Category, *SDC*

C.) Approximate fundamental period, T_a

D.) Seismic response coefficient, C_S

E.) Seismic base shear, V

Solution:

Emergency Operations Center = *Risk Category* IV → *2018 IBC Table 1604.5*

$I_e = 1.5$ → *ASCE 7-16 – Table 1.5-2*: *Risk Category* IV

$F_a = 1.2$ → *2018 IBC Table 1613.2.3(1)*: *Site Class* E & $S_S = 1.93$

$F_v = 2.0$ → *2018 IBC Table 1613.2.3(2)*: *Site Class* E & $S_1 = 0.77$

$R = 8$ → *ASCE 7-16 – Table 12.2-1, item D.1*: Dual system – Steel SMF & Steel EBF

Building height, $h_n = 5$ stories (12 feet/story) = 60 feet

Total effective seismic weight, W = <u>24,000 kips</u>

A.) Design Spectral Response Acceleration Parameters, S_{DS} & S_{D1}

$S_{MS} = F_a S_S$ *IBC (16-36)*

$= 1.2\,(1.93) = 2.32$

$S_{DS} = 2/3\, S_{MS}$ *IBC (16-38)*

$= 2/3\,(2.32) = \boxed{1.55}$

$S_{M1} = F_v S_1$ *IBC (16-37)*

$= 2.0\,(0.77) = 1.54$

$S_{D1} = 2/3\, S_{M1}$ *IBC (16-39)*

$= 2/3\,(1.54) = \boxed{1.03}$

$S_S = 1.93$ & *Site Class* E $\rightarrow$ Table 3.2 $\rightarrow$ $S_{DS} = \underline{1.55}$ … by interpolation
$S_1 = 0.77$ & *Site Class* E $\rightarrow$ Table 3.3 $\rightarrow$ $S_{D1} = \underline{1.03}$ … by interpolation

B.) Seismic Design Category, *SDC*

$S_1 = 0.77 > 0.75$ & $RC = \text{IV}$ $\rightarrow$ *2018 IBC §1613.2.5* $\rightarrow$ $SDC = \text{F}$

$\therefore \boxed{SDC = \text{F}}$

C.) Approximate Fundamental Period, *Tₐ*

$T_a = C_t h_n^x$ *ASCE 7 (12.8-7)*

Dual System D.1 - steel EBF & steel SMF $\rightarrow$ *ASCE 7-16 – Table 12.8-2*: $C_t = 0.03$ & $x = 0.75$

$T_a = 0.03\,(h_n)^{0.75}$

$= 0.03\,(60 \text{ feet})^{0.75} = \boxed{0.65 \text{ second}}$

Using Table C1 (Appendix C, p. 5-20) $\rightarrow$ Dual System D.1 & $h_n = 60$ feet $\rightarrow$ $T_a = \underline{0.65 \text{ second}}$

NOTE: $T = T_a = 0.65$ second $< T_S = \dfrac{S_{D1}}{S_{DS}} = \dfrac{1.03}{1.55} = 0.66$ second $\rightarrow$ $\therefore$ *ASCE 7 (12.8-2)* will govern C_S

D.) Seismic Response Coefficient, *C_S*

$C_S = \dfrac{S_{DS}}{(R/I_e)} = \dfrac{(1.55)}{(8/1.5)} = \underline{0.291}$ $\leftarrow$ governs *ASCE 7 (12.8-2)*

C_S need not exceed the following,

$C_S = \dfrac{S_{D1}}{T(R/I_e)} = \dfrac{(1.03)}{0.65(8/1.5)} = 0.298$ *ASCE 7 (12.8-3)*

NOTE: *ASCE 7 (12.8-4)*, *(12.8-5)*, and *(12.8-6)* are not applicable since $T < T_S$ (and $T << T_L$).

Also, per *ASCE 7-16 – §11.4.8 - Exception 3*, a ground motion hazard analysis (in accordance with *ASCE 7-16 – §21.2*) is not required for structures on *Site Class* E sites with $S_1 \geq 0.2$ provided that the structures period $T \leq T_S$ and the Equivalent Lateral Force (ELF) procedure is used for design. Therefore, if it was determined that $T > T_S$ … a ground motion hazard analysis would have been required.

$\therefore C_S = \boxed{0.291}$

E.) Seismic Base Shear, *V*

$V = C_S W$ *ASCE 7 (12.8-1)*

$= 0.291\,(24{,}000 \text{ kips}) = \boxed{6980 \text{ kips}}$

A.) Seismic Base Shear, *V*

$$V = \frac{F \cdot S_{DS}}{R} W \qquad \text{ASCE 7 (12.14-12)}$$

$$= [1.2\,(1.4)\,/\,5\tfrac{1}{2}\,]\ W = 0.306\ W$$

$$= 0.306\,(800) = \boxed{245 \text{ kips}}$$

B.) Lateral Force at Each Level, *F$_x$*

$$F_x = \frac{w_x}{W} V \qquad \text{ASCE 7 (12.8-13)}$$

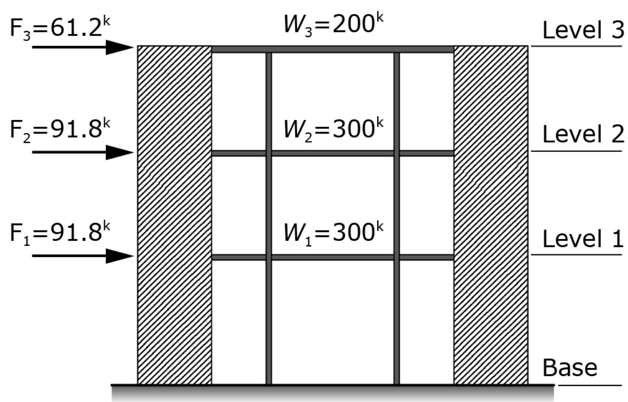
$$= \frac{F \cdot S_{DS}}{R} w_x = 0.306\ w_x$$

$$F_1 = 0.306\ w_1 = 0.306\,(300 \text{ kips}) = \boxed{91.8 \text{ kips}}$$

$$F_2 = 0.306\ w_2 = 0.306\,(300 \text{ kips}) = \boxed{91.8 \text{ kips}}$$

$$F_3 = 0.306\ w_3 = 0.306\,(200 \text{ kips}) = \boxed{61.2 \text{ kips}}$$

Check … $V = \sum_{i=1}^{n} F_i = 91.8 + 91.8 + 61.2 = \underline{245 \text{ kips}} \quad \underline{\text{OK}}$



Elevation

C.) Story Shear at Each Story, *V$_x$*

$$V_x = \sum_{i=x}^{n} F_i \qquad \text{ASCE 7 (12.14-14)}$$

3rd story shear - $V_3 = F_3 = 61.2 = \boxed{61.2 \text{ kips}}$

2nd story shear - $V_2 = F_3 + F_2 = 61.2 + 91.8 = \boxed{153 \text{ kips}}$

1st story shear - $V_1 = F_3 + F_2 + F_1 = 61.2 + 91.8 + 91.8 = \boxed{245 \text{ kips}}$

D.) Diaphragm Design Force at Each Level, *F$_{px}$*

ASCE 7-16 – §12.14.7.4 - floor and roof diaphragms shall be designed to resist the design seismic forces at each level (F_x) calculated in accordance with *§12.14.8.2*:

Level 1 - $F_{p1} = F_1 = \boxed{91.8 \text{ kips}}$

Level 2 - $F_{p2} = F_2 = \boxed{91.8 \text{ kips}}$

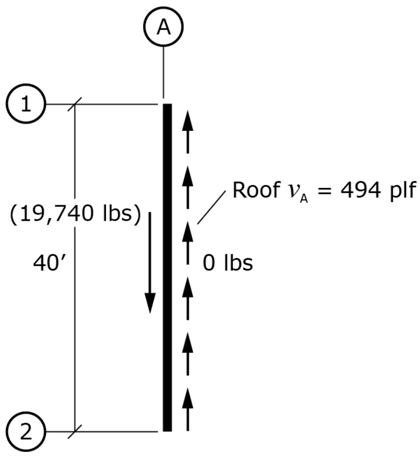
Level 3 (roof) - $F_{p3} = F_3 = \boxed{61.2 \text{ kips}}$

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

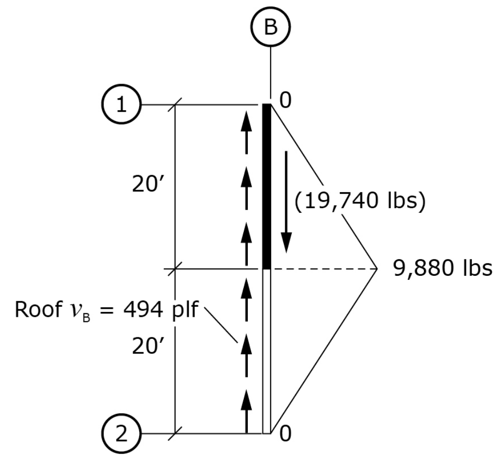
roof $v_A = v_B = 494$ plf

Wall Line A: F_d = 0 lbs

Wall Line B: F_d = (494 plf)(20′) = 9,880 lbs (SD/LRFD force level)



Drag Force – Line A



Drag Force – Line B

> **NOTE:** The actual design of the "*collector elements and their connections …*" would need to consider the overstrength factor as required by *ASCE 7-16 - §12.10.2.1* for this structure assigned to *SDC* = D (i.e., design for $\Omega_0 \cdot Q_E$ where Q_E is the drag force determined above … see p. 1-121).

B.) E-W DIRECTION: $L = 40'$, $d = 70'$

1. Design Seismic Force to Diaphragm, $w_s = f_{p1} = F_{p1}/L$

$$W_{p1} = \underbrace{(16\text{ psf} + 20\%\cdot 100\text{ psf})(70')(40')}_{\text{roof DL + 20\% snow}} + \underbrace{(85\text{ psf})(14'/2 + 2')(2\text{ walls})(40')}_{\text{East \& West exterior walls}}$$
$$= 100{,}800\text{ lbs} + 61{,}200\text{ lbs} = 162{,}000\text{ lbs}$$

$F_{p1} = 0.190\ W_{p1} = 0.190\ (162{,}000\text{ lbs}) = 30{,}780$ lbs

$w_s = f_{p1} = F_{p1}/L = (30{,}780\text{ lbs})/(40') = \boxed{770\text{ plf}}$

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

$V_1 = V_2 = w_s\,L/2 = (770\text{ plf})(40'/2) = \underline{15{,}400\text{ lbs}}$

Roof $v_1 = v_2 = V_1/d = (15{,}400\text{ lbs})/70' = \boxed{220\text{ plf}}$ (SD/LRFD force level)

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

max. $M = w_s\,L^2/8 = (770\text{ plf})(40')^2/8 = 154{,}000$ lb-ft

max. $CF = (154{,}000\text{ lb-ft})/70' = \boxed{2{,}200\text{ lbs}}$ (SD/LRFD force level)

4. Shear Force to walls 1A & 1B

Relative Rigidities: assume cantilever walls, Table D1 - Relative Rigidity of Cantilever Shear Walls / Piers (Appendix D, p. 5-22)

Wall 1A: $H/D = 14'/11' = 1.27$ → Table D1 (p. 5-22) → $R_{1A} = 0.833$

Wall 1B: $H/D = 14'/22' = 0.64$ → Table D1 (p. 5-22) → $R_{1B} = 3.369$

$\Sigma R = R_{1A} + R_{1B} = 0.833 + 3.369 = 4.202$

3.39 Given a 10-story Office building with $S_1 = 0.67$, $S_{DS} = 0.82$ & $S_{D1} = 0.67$ … what is the appropriate *Seismic Design Category*?

a. $SDC = \text{B}$
b. $SDC = \text{C}$
c. $SDC = \text{D}$
d. $SDC = \text{E}$

3.40 Given a 2-story Apartment building with $S_1 = 0.20$, $S_{DS} = 0.41$ & $S_{D1} = 0.20$ … what is the appropriate *Seismic Design Category*?

a. $SDC = \text{B}$
b. $SDC = \text{C}$
c. $SDC = \text{D}$
d. $SDC = \text{E}$

3.41 The *Seismic Design Category* is used to determine the:

a. permissible lateral analysis procedure (based on *Risk Category*)
b. level of seismic detailing required for the seismic force-resisting system (SFRS)
c. building height limit (based on SFRS type)
d. all the above

3.42 A structure with an assigned *Seismic Design Category* C represents which of the following:

a. Very low seismic hazard level
b. Low seismic hazard level
c. Moderate seismic hazard level
d. High seismic hazard level

3.43 Which of the following occupancy types would <u>never</u> be assigned to *Seismic Design Category* F ($SDC = \text{F}$)?

a. Hospital with emergency surgery or emergency treatment
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 *Risk Category*?

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

3.45 What would be the most appropriate MCE_R spectral response acceleration parameters (S_S & S_1) for a building project proposed at 45°00′00″ Latitude and -120°00′00″ Longitude?

a. $S_S = 0.15$ & $S_1 = 0.05$
b. $S_S = 0.30$ & $S_1 = 0.10$
c. $S_S = 0.36$ & $S_1 = 0.15$
d. $S_S = 0.45$ & $S_1 = 0.21$

4.34 Which shear wall would be considered the <u>least</u> ductile?

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

4.35 A 15-story Office building utilizes steel special concentrically braced frames (R = 6) with a fundamental period of 1.2 seconds and effective seismic weight of 12,000 kips. The acceleration parameters are determined to be $S_S = 1.04$, $S_1 = 0.44$, $S_{DS} = 0.83$ & $S_{D1} = 0.55$. What is the Equivalent Lateral Force procedure seismic base shear?

a. 444 kips
b. 912 kips
c. 1,140 kips
d. 1,660 kips

4.36 Given a 3-story light-framed apartment building with wood structural panel shear walls (bearing walls), $S_{DS} = 0.75$, *Seismic Design Category* E, and effective seismic weight of 200 kips. Determine the seismic base shear using the Simplified Design Procedure of *ASCE 7-16.*

a. 15 kips
b. 23 kips
c. 28 kips
d. 35 kips

4.37 What is the <u>minimum</u> seismic base shear for a *Risk Category* IV structure using steel special moment frames (SMF's) and with $S_1 = 1.10$ & $S_{DS} = 1.33$?

a. 0.069 W
b. 0.088 W
c. 0.103 W
d. 0.166 W

4.38 When using the *ASCE 7-16* Equivalent Lateral Force procedure, <u>actual</u> seismic forces from the DBE ground motion (i.e., 2/3 MCE_R) in relation to *ASCE 7-16* <u>design</u> seismic forces are:

a. slightly smaller
b. much smaller
c. equal
d. greater

4.39 In the *ASCE 7-16*, the factor Ω_0 represents an/a:

a. increase due to actual seismic forces
b. decrease due to actual seismic forces
c. increase of factor of safety for workmanship and materials
d. decrease of factor of safety for workmanship and materials

4.40 What is the approximate ratio between the <u>actual</u> DBE seismic base shear and the *ASCE 7-16* Equivalent Lateral Force procedure <u>design</u> seismic base shear?

a. 1 to 1
b. 2½ to 1
c. 4 to 1
d. 8 to 1

5.3 A lateral analysis of a 2-story Office building determines that a steel braced frame column has the following axial load effects: D = 20 kips, L = 15 kips, L_r = 0 kips and E = 25 kips. Assume ρ = 1.0 to determine the maximum axial compression force in this column using the Strength Design (SD or LRFD) load combinations of *IBC §1605.2*.

a. 45 kips
b. 53 kips
c. 57 kips
d. 61 kips

5.4 In *ASCE 7-16 – §12.4.2.1,* the symbol Q_E represents which of the following?

a. The effects of the horizontal seismic forces from the seismic base shear (V)
b. The effects of the horizontal seismic forces from the nonstructural component seismic design force (F_p)
c. The seismic design base shear (V)
d. Either a or b

5.5 Which of the following conditions would require the use of the load combinations with *overstrength factor* (Ω_0) of *ASCE 7-16 – §12.4.3.2*?

I. Vertical structural irregularity type 4 (*ASCE 7-16 – Table 12.3-2*)
II. Vertical structural irregularity type 5a (*ASCE 7-16 – Table 12.3-2*)
III. Horizontal structural irregularity type 4 (*ASCE 7-16 – Table 12.3-1*)

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

5.6 Use of a *redundancy factor* (ρ) greater than 1.0 is intended to:

a. reduce the inelastic response and ductility demand of a structure
b. increase the seismic base shear (V) on a structure
c. decrease the calculated story drift within a structure
d. increase the inelastic response and ductility demand of a structure

A column of a steel special concentrically braced frame (SCBF), in a single-story Medical Office building (SDC = D), is determined to support the following axial load effects: dead load - D = 35 kips, floor live load - L = 0 kips, roof live load - L_r = 15 kips, and horizontal seismic load effect - Q_E = 15 kips. Given S_{DS} = 1.25, overstrength factor Ω_0 = 2, and redundancy factor ρ = 1.3, answer questions 5.7 through 5.13.

5.7 What is the vertical seismic load effect axial force in this column?

a. 0 kips
b. ± 7 kips
c. ± 9 kips
d. ± 12 kips

5.8 What is the horizontal seismic load effect axial force in this column?

a. ± 12 kips
b. ± 15 kips
c. ± 20 kips
d. ± 24 kips

5.9 What is the <u>maximum</u> axial compression force in this column using the <u>Strength Design</u> (SD or LRFD) load combinations of *IBC §1605.2*?

a. 53 kips
b. 62 kips
c. 67 kips
d. 70 kips

5.10 What is the <u>minimum</u> axial compression force in this column using the <u>Strength Design</u> (SD or LRFD) load combinations of *IBC §1605.2*?

a. 3 kips
b. 7 kips
c. 9 kips
d. 12 kips

5.11 What is the <u>horizontal</u> seismic load effect with overstrength factor (E_{mh}) axial force in this column?

a. ± 15 kips
b. ± 20 kips
c. ± 30 kips
d. ± 37 kips

5.12 What is the <u>maximum</u> axial compression force in this column using the Basic <u>strength design</u> load combinations with overstrength factor of *IBC §1605.1* and *ASCE 7-16 – §12.4.3*?

a. 51 kips
b. 63 kips
c. 72 kips
d. 81 kips

5.13 What is the <u>minimum</u> axial compression force in this column using the Basic <u>strength design</u> load combinations with overstrength factor of *IBC §1605.1* and *ASCE 7-16 – §12.4.3*?

a. 10 kips
b. 5 kips
c. 2 kips
d. – 7 kips

5.14 What is the maximum *redundancy factor* that needs to be considered for a structure assigned to *Seismic Design Category* E?

a. 1.0
b. 1.25
c. 1.3
d. 1.5

5.15 The vertical seismic load effect is permitted to be taken as zero (i.e., $E_v = 0$) under the following conditions:

a. structures assigned to *Seismic Design Category* B
b. structures assigned to *Seismic Design Category* B or C
c. when the Dead Load effect is equal to zero (i.e., $D = 0$)
d. Both a & c

10.9 Geotechnical investigations shall include the potential for *liquefaction* and soil strength loss evaluated for peak ground accelerations for structures assigned to *Seismic Design Category*:

a. B, C, D, E & F
b. C, D, E & F
c. D, E & F
d. E & F

10.10 Geotechnical investigations shall include an evaluation for potential slope instability, *liquefaction*, total and differential settlement, surface fault displacement, etc. for structures assigned to *Seismic Design Category*:

a. B, C, D, E & F
b. C, D, E & F
c. D, E & F
d. E & F

10.11 What design standard applies to the seismic design requirements of masonry construction per the *2018 IBC*?

a. *TMS 402-16*
b. *ACI 318-14*
c. *ACI 530-16*
d. *ASCE 7-16*

10.12 The seismic force-resisting systems (SFRS) of structural steel structures assigned to *Seismic Design Category* D, E or F shall be designed to what design standard per the *2018 IBC*?

a. *AISC 341-16*
b. *AISC 360-16*
c. *ACI 318-14*
d. *ASCE 7-16*

10.13 Which earthquake resulted in the *California Building Code (CBC)* no longer permitting the use of the pre-qualified beam-column connection for welded steel moment frames without justifying by cyclic testing or by calculation.

a. 1985 Mexico City, Mexico earthquake (M_w 8.0)
b. 1989 Loma Prieta, CA earthquake (M_w 6.9)
c. 1994 Northridge, CA earthquake (M_w 6.7)
d. 1995 Kobe, Japan earthquake (M_w 6.9)

11.1 According to the *IBC*, which of the following would be allowed to employ one or more *approved agencies* to perform the required *special inspections* on a project?

I. Building Owner
II. Owner's authorized agent
III. Contractor

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

Problem	Answer	Reference / Solution
4.65	a	p. 1-67 to 68 & *ASCE 7-16* p. 104 - *§12.8.7* *P*-delta effects need not be considered where the stability coefficient is equal to or less than 0.10 … ∴ the *stability coefficient* (θ) does not exceed 0.10 ←
4.66	b	p. 1-51 & *ASCE 7-16* p. 100 - *Table 12.6-1* … *ASCE 7-16* may require a dynamic analysis procedure for the design of these irregular structures. ∴ Dynamic lateral force procedures ←
4.67	a	1-54 & *ASCE 7-16* p. 97 - *Table 12.3-2, Type 5a* Weak story is defined to exist when the … story lateral strength < 80% of that in a story above ←
4.68	b	p. 1-54 & *ASCE 7-16* p. 97 - *Table 12.3-2, Type 1a & 5a* Stiffness: (14 kips/inch / 19.5 kips/inch)(100%) = 72% > 70% OK Strength: (57 kips / 76kips)(100%) = 75% < 80% ← NG Soft Story does not exist, but Weak Story does … ∴ II ←
4.69	a	p. 1-54 & *ASCE 7-16* p. 97 - *Table 12.3-2, Type 2* I. W_1 / W_2 = (320 kips / 200 kips)(100%) = 160% > 150% ← NG II. W_4 / W_3 = (200 kips / 150 kips)(100%) = 133% < 150% OK III. W_1 / W_2 = (250 kips / 200 kips)(100%) = 125% < 150% OK **NOTE:** A roof that is lighter than the floor below need not be considered … therefore, no need to check W_3 / W_4 for structure III. ∴ I ←
4.70	d	p. 1-70 to 71 & *ASCE 7-16* p. 95 - *Table 12.3-1* & p. 100 - *§12.5.3* For *SDC* = C, structures that have horizontal structural irregularity Type 5 in *ASCE 7-16 – Table 12.3-1* (i.e., Non Parallel Systems Irregularity) shall use one of the following procedures: *ASCE 7-16 – §12.5.3.1, item a* - Orthogonal Combination Procedure **or** *ASCE 7-16 – §12.5.3.1, item b* - Simultaneous Application of Orthogonal Ground Motion. ∴ *ASCE 7-16 – §12.5.3.1, item a* or *b* ←
4.71	a	p. 1-54 - Table 4.2 & *ASCE 7-16* p. 97 - *Table 12.3-2, Type 1a* Stiffness-Soft Story Irregularity may be present only when the lateral stiffness decreases as you proceed down the height of the structure (e.g., from top to bottom) … and only structure I loses a number of braces (i.e., stiffness) in the 1st story. ∴ I ←
4.72	c	p. 1-59 - Seismic Base Shear Design seismic forces (e.g., seismic base shear, etc.) are determined at a Strength Design (SD) force level … essentially the same as LRFD ∴ I & II ←

Problem	Answer	Reference / Solution
		$0.9(D+F)+1.0E+1.6H$ *IBC (16-7)* minimum axial, P = 0.9(35 kips + 0) + 1.0(– 28.3 kips) + 1.6(0) = 3.2 kips ∴ <u>3 kips</u> ←
5.11	c	p. 1-80 - Horizontal Seismic Load Effect w/ Overstrength Factor & *ASCE 7-16* p. 99 - *§12.4.3.1* $E_{mh} = \pm\,\Omega_0\,Q_E$ *ASCE 7 (12.4-7)* E_{mh} = ± 2 (15 kips) = ± 30.0 kips ∴ ± <u>30 kips</u> ←
5.12	d	p. 1-79 to 80 - Seismic Load Effect Including Overstrength Factor, *2018 IBC* p. 365 - *§1605.1* & *ASCE 7-16* p. 8 - *§2.3.6* E_v = ± 8.8 kips E_{mh} = ± 30.0 kips Use <u>positive</u> E_v and E_{mh} to determine <u>maximum</u> compression. The following *ASCE 7-16* (SD/LRFD) Basic load combination including overstrength factor (Ω_0) will govern for <u>maximum</u> axial compression – 6. $1.2D+E_v+E_{mh}+L+0.2S$ 1.2 (35 kips) + 8.8 kips + 30.0 kips + 0 + 0.2(0) = 80.8 kips ∴ <u>81 kips</u> ←
5.13	d	p. 1-79 to 80 - Seismic Load Effect Including Overstrength Factor, *2018 IBC* p. 365 - *§1605.1* & *ASCE 7-16* p. 8 - *§2.3.6* E_v = ± 8.8 kips E_{mh} = ± 30.0 kips Use <u>negative</u> E_v and E_{mh} to determine <u>minimum</u> compression. The following *ASCE 7-16* (SD/LRFD) Basic load combination including overstrength factor (Ω_0) will govern for <u>mimimum</u> axial compression – 7. $0.9D-E_v+E_{mh}$ 0.9 (35 kips) – 8.8 kips – 30.0 kips = – 7.3 kips ∴ – <u>7 kips</u> ←
5.14	c	p. 1-78 to 79 - Redundancy Factor & *ASCE 7-16* p. 98 - *§12.3.4.2* For structures assigned to *SDC* = D … E or F – ρ = <u>1.3</u> shall be used unless one of the following two conditions is met, where ρ = 1.0 is permitted … ∴ maximum ρ = <u>1.3</u> ←
5.15	d	p. 1-78 - Vertical Seismic Load Effect & *ASCE 7-16* p. 99 - *§12.4.2.2* Per *Exception 2* - it is permitted to use E_v = 0 for … structures assigned to <u>*SDC* = B</u> Also, if the Dead load effect is zero (i.e., D = 0) - $E_v = \pm\,0.2S_{DS}D$ *ASCE 7 (12.4-4a)* $E_v = \pm\,0.2S_{DS}(0) = 0$ ∴ <u>Both a & c</u> ←

Problem	Answer	Reference / Solution
		A geotechnical investigation shall be conducted, and shall include an evaluation of all of the following potential geologic and seismic hazards - slope instability, *liquefaction*, total and differential settlement, surface fault displacement, etc. ∴ C, D, E & F ←
10.11	a	p. 1-173 - Masonry & *IBC* p. 468 - *§2106.1* The seismic design requirements of *TMS 402-16 – Part 2: Chapter 7* shall apply to the design and construction of masonry … based on the structures *Seismic Design Category.* ∴ *TMS 402-16* ←
10.12	a	p. 1-176 - *Seismic Design Category* D, E or F & *IBC* p. 480 - *§2205.2.1.2* (Structural steel SFRS of) structures assigned to *SDC* = D, E or F shall be designed and detailed in accordance with *AISC 341-16 – Seismic Provisions for Structural Steel Buildings …* ∴ *AISC 341-16* ←
10.13	b	p. 1-177 - NOTE Following the January 17, 1994 Northridge Earthquake … the *International Conference of Building Officials (ICBO)* adopted an emergency code change to the *1994 Uniform Building Code (UBC)*. This code change omitted the pre-qualified connection (for welded steel moment frames) … ∴ 1994 Northridge, CA earthquake (M_w 6.7) ←
11.1	c	p. 1-182 & *2018 IBC* p. 416 - *§1704.2* – Special inspections The owner or the owner's agent, other than the contractor, shall employ one or more *approved agencies* to provide special inspections and tests … ∴ I & II ←
11.2	d	p. 1-183 to 184 & *2018 IBC* p. 416 to 422 - *§1705* I. Masonry construction - *§1705.4* II. Structural steel welding - *§1705.2* III. Cast-in-place deep foundations - *§1705.8* ∴ I, II & III ←
11.3	c	p. 1-186 & *2018 IBC* p. 418 - *§1704.6* … the owner or the owner's authorized agent shall employ a registered design professional to perform structural observations. Ideally, the *structural observer* should be ... the engineer (or architect) responsible for the structural design, etc. ∴ Civil engineer responsible for the structural design ←
11.4	b	p. 1-187 & *2018 IBC* p. 418 - *§1704.6.2, item 1 & 2* *Structural observations* for seismic resistance shall be provided for those structures assigned to *SDC* = D, E or F where one or more of the following conditions exist: *(continued)*