CHAPTER 5

Question No.	Brief Explanation	Correct Answer A B C D
5.1	From Fig. IBC 1613.5(3) and (4) enlarged region 1 (ASCE 7 Fig. 22-3 and 22-4) S_S = 1.5g, and S_1 = 0.6g. The g term is already factored in the equations, thus use only the digit.	● ○ ○ ○
5.2	Shear wave velocity = 7500 in/sec = 625 ft/sec. From IBC Table 1613.5-2, Site Class D.	○ ○ ○ ●
5.3	Table 5.1: Site Class C, S_S = 0.85, by interpolation, F_a = 1.06; and S_1 = 0.4, F_v = 1.4.	○ ● ○ ○
5.4	Table 5.1: Site Class D, S_S = 0.90 and S_1 = 0.4 F_a = 1.14, F_v = 1.6 S_{MS} = 0.9*1.14 = 1.026; S_{M1} = 1.6*0.4 = 0.64 S_{DS} = 2/3*1.026 = 0.69; S_{D1} = 2/3*0.64 = 0.43	○ ○ ● ○
5.5	Table 5.1: Site Class E, S_S = 1.12 and S_1 = 0.35 F_a = 0.9, F_v = 2.6 (by interpolation) S_{MS} = 0.9*1.12 = 1.01; S_{M1} = 2.6*0.35 = 0.91 S_{DS} = 2/3*1.01 = 0.67; S_{D1} = 2/3*0.91 = 0.61	○ ● ○ ○
5.6	Shear wave 2400 ft/sec. IBC Table 1613.5-2, Site Class C Table 5.1: Site Class C, S_S = 0.95 and S_1 = 0.52 F_a = 1.02 (by interpolation), F_v = 1.3 S_{MS} = 0.95*1.02 = 0.97; S_{M1} = 1.3*0.52 = 0.68 S_{DS} = 2/3*0.97 = 0.65; S_{D1} = 2/3*0.68 = 0.46	● ○ ○ ○
5.7	S_{DS} = 0.65, and S_{D1} = 0.46 T_0 = 0.2 * S_{D1}/S_{DS} = 0.2 * 0.46/0.65 = 0.142 T_S = S_{D1}/S_{DS} = 0.46/0.65 = 0.71	○ ○ ● ○
5.8	S_{DS} = 0.9, and S_{D1} = 0.4 T_0 = 0.2 * S_{D1}/S_{DS} = 0.2 * 0.4/0.9 = 0.09 T_S = S_{D1}/S_{DS} = 0.4/0.9 = 0.44	○ ○ ○ ●
5.9	ASCE 7 Fig 22-16 (Region 1, California) T_L = 8 sec	○ ● ○ ○
5.10	Concrete moment frame. h_n = 120 ft The approximate fundamental period T_a: $T_a = 0.016\,(120)^{0.90} = 1.19$ sec	● ○ ○ ○
5.11	Steel eccentrically braced frame. h_n = 180 ft The approximate fundamental period T_a: $T_a = 0.03\,(180)^{0.75} = 1.47$ sec	○ ● ○ ○
5.12	Steel moment frame. h_n = 270 - 10 = 260 ft The approximate fundamental period T_a: $T_a = 0.028\,(260)^{0.80} = 2.39$ sec	○ ○ ● ○

5.13 Masonry shear wall (other structures). $h_n = 55$ ft ○ ● ○ ○
The approximate fundamental period T_a:
$$T_a = 0.02\,(55)^{0.75} = 0.4 \text{ sec}$$

5.14 Moment frame (concrete or steel) with number of stories not more than 12 and minimum story height of 10 ft ● ○ ○ ○
$T_a = 0.1\,N = 0.1*9 = 0.9$ sec N: number of stories

5.15 Steel eccentric braced frame. $T_a = 0.03\,(180)^{0.75} = 1.47$ sec ○ ● ○ ○
From Table 5.4 using the value of S_{D1}
$S_{D1} = 0.5$ $C_u = 1.4$
Thus $T_{rational} \leq 1.4 * 1.47 = 2.1$ sec.

5.16 Steel moment frame. $T_a = 0.028\,(260)^{0.80} = 2.39$ sec ○ ○ ● ○
From Table 5.4 using the value of S_{D1}
$S_{D1} = 0.42$ $C_u = 1.4$
Thus $T_{rational} \leq 1.4 * 2.39 = 3.35$ sec.

5.17 The office building has an occupant load of 10,000. From IBC Table 1604.5, Occupancy Category III. ○ ○ ● ○
Site Class C, $S_S = 1.6$ and $S_1 = 0.6$
Table 5.1: $F_a = 1.0$, and $F_v = 1.3$
$S_{DS} = 2/3*1.0*1.6 = 1.07$ and $S_{D1} = 2/3*1.3*0.6 = 0.523$
From Table 5.5: $S_1 < 0.75$
Based on S_{DS}, SDC = D
Based on S_{D1}, SDC = D

5.18 Essential facility building with Occupancy Category IV. ○ ○ ● ○
Site Class D, $S_S = 1.10$ and $S_1 = 0.35$
Table 5.1: $F_a = 1.06$, and $F_v = 1.7$
$S_{DS} = 2/3*1.06*1.10 = 0.78$ and $S_{D1} = 2/3*1.7*0.35 = 0.398$
From Table 5.5: $S_1 < 0.75$
Based on S_{DS}, SDC = D
Based on S_{D1}, SDC = D

5.19 The college building has an occupant load of 1,000. From IBC Table 1604.5, Occupancy Category III. ○ ○ ○ ●
Site Class E, $S_S = 1.7$ and $S_1 = 0.9$
From Table 5.5: $S_1 > 0.75$
Thus, SDC = E

5.20 Agricultural facility building. From IBC Table 1604.5, Occupancy Category I. ○ ○ ○ ●
Site Class C, $S_S = 0.65$ and $S_1 = 0.20$
Table 5.1: $F_a = 1.14$, and $F_v = 1.60$
$S_{DS} = 2/3*1.14*0.65 = 0.49$ and $S_{D1} = 2/3*1.60*0.20 = 0.26$
From Table 5.5: $S_1 < 0.75$
Based on S_{DS}, SDC = C
Based on S_{D1}, SDC = D Thus, SDC = D

5.21	For a building frame system composed of space frame and braced frame, the space frame resists gravity loads and braced frame resists seismic loads	● ○ ○ ○
5.22	Dual system composed of moment frame resisting more than 25% of seismic loads and shear wall	○ ● ○ ○
5.23	In a bearing wall system, light framed shear walls support the building's vertical gravity loads in addition to the lateral loads	○ ● ○ ○
5.24	For a special moment resisting frame building with any SDC, there is no limit on the height of the building as shown on ASCE 7 Table 12.2-1.	○ ○ ○ ●
5.25	Steel eccentrically braced frames with rigid connections and SDC = F. From ASCE 7 Table 12.2-1 for building frame systems, height is limited to 100 ft.	○ ● ○ ○
5.26	light framed wood structural panel shear walls with SDC = F. From ASCE 7 Table 12.2-1 for building frame systems, height is limited to 65 ft.	● ○ ○ ○
5.27	The response modification coefficient, R, is used to measure the ability of the building to deform inelastically and dissipate earthquake energy. It is calculated as the ratio of the theoretical seismic base shear which would develop in a linear elastic system, to the prescribed design base shear. It also reflects the observed performance of specific materials and structural systems in past earthquakes. All of the above.	○ ○ ○ ●
5.28	Over strength factor, Ω_0, accounts for the maximum force that can be developed in the structure at the formation of the inelastic collapse mechanism.	○ ● ○ ○
5.29	Deflection amplification factor is used to determine the actual inelastic displacement exhibited by the structure when subjected to the design ground motions.	○ ○ ● ○
5.30	The most ductility has the highest R from ASCE 7 Table 12.2-1. intermediate reinforced concrete moment frame, $R = 5$ light framed wood structural panels shear walls (bearing wall system), $R = 6.5$ special reinforced masonry shear walls, $R = 5.5$ ordinary steel concentrically braced frame, $R = 3.25$	○ ● ○ ○
5.31	intermediate steel moment frames (resisting more than 25% of seismic loads) with special reinforced concrete shear walls (Dual system). From ASCE 7 Table 12.2-1, $R = 6.5$, and $\Omega_0 = 2.5$.	● ○ ○ ○

5.32 SDC = F. ASCE 7 Table 12.2-1, height limit is 100 ft and deflection amplification factor, $C_d = 5$ — ○ ○ ● ○

5.33 Dual system of shear wall and intermediate moment frame (other structures). $h_n = 100$ ft
The approximate fundamental period T_a:
$T_a = 0.02\,(100)^{0.75} = 0.63$ sec. — ○ ○ ○ ●

5.34 When combining SFRS in different directions, the limitations of application will be calculated for each direction independent from the other one. Thus for N-S direction, $R = 5$ and for E-W direction, $R = 3.25$. — ○ ○ ● ○

5.35 When combining SFRS in different directions, the limitations of application will be calculated for each direction independent from the other one. Thus for N-S direction, $\Omega_0 = 2.5$ and for E-W direction, $\Omega_0 = 2.0$. — ○ ○ ○ ●

5.36 When different SFRS are used in combination to resist seismic forces in the same direction, the more stringent limitations of ASCE 7-Table 12.2-1 are used. Special steel moment frame, $R = 8$, and for special steel concentric braced frame, $R = 6$. Use the lowest R. $R = 6$. — ● ○ ○ ○

5.37 Special steel moment frame, $\Omega_0 = 3.0$, and for special steel concentric braced frame, $\Omega_0 = 2.0$. Use the larger Ω_0. $\Omega_0 = 3.0$. — ○ ○ ○ ●

5.38 Special steel moment frame, $C_d = 5.5$, and for special steel concentric braced frame, $C_d = 5.0$ Use the larger C_d. $C_d = 5.5$ — ○ ○ ○ ●

5.39 If combinations of different systems are used to resist lateral forces in the same direction, the R shall not be greater than the lowest value of R of any system utilized in that direction. Special steel moment frame, $R = 8$, and for special steel plate shear wall, $R = 7$. Use the lowest R. $R = 7$. — ○ ● ○ ○

5.40 Special steel moment frame, $\Omega_0 = 3.0$, and for special steel plate shear wall, $\Omega_0 = 2.0$. Use the larger Ω_0. $\Omega_0 = 3.0$. — ○ ○ ○ ●

5.41 Soft story exist when story lateral stiffness is less than 70% of the story above, or less than 80% of the average of the three stories above. — ○ ○ ● ○

5.42 Type 5 horizontal irregularity has the vertical lateral force resisting elements placed non-parallel to major axes. — ○ ○ ● ○

5.43 Extreme weak story vertical irregularity has the story lateral strength less than 65% of the story above. — ● ○ ○ ○

5.44 20/90 = 22% > 15%; 90/145 = 62% > 15%. Both projections are above 15% of corresponding dimensions. Reentrant Corner. — ○ ○ ● ○
$\delta_{avg} = (1.5 + 2.5) / 2 = 2.0''$

$1.2\ \delta_{avg} = 1.2 * 2.0 = 2.4"$
$\delta_{max} = 2.5" > 1.2\ \delta_{avg} = 2.4"$ Torsional irregularity.

5.45 First floor weight > 1.5 * adjacent floor weight
130 kips < 1.5 * 100 = 150 kips. No weight irregularity.
horizontal dimension of first floor > 1.3 * horizontal dimension of adjacent story
120 ft > 1.3 * 90 = 117 ft. Vertical geometric irregularity

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5.46 Building D has extreme soft story vertical irregularity.
First story has bracing in two bays compared to four braces in the stories above (i.e., 50% less).
First Story stiffness < 60% of stiffness of story above.

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5.47 In-plane discontinuity of seismic force resisting elements.

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5.48 Type 3 horizontal irregularity of ASCE 7 Table 12.3.1
open area in a diaphragm more than 50% of gross enclosed area

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5.49 ASCE7 12.3.3 limits the use of certain building geometry and the existence of certain vertical and horizontal irregularities depending on the Seismic Design Category.
Extreme weak story vertical irregularity (Type 5-b) is not permitted for structures with SDC = D as listed in Table 5.8

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5.50 Out of plane offset horizontal irregularity is not permitted for structures with SDC = E as listed in Table 5.7

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5.51 Roof effective seismic weight:

○ ○ ○ ●

Dead load	=	20 x 6000	=	120 kips
Snow load	=	0.2x40 x 6000	=	48 kips
Wall dead load	=	30x320x8	=	76.8 kips

Total roof effective seismic weight, W_3 = 120 + 48 + 76.8 = 244.8 kips
1st and 2nd story effective seismic weight:

Dead load	=	40 x 6000	=	240 kips
Wall dead load	=	30x320x16	=	153.6 kips
Partition load	=	10 x 6000	=	60 kips

Total story effective seismic weight, $W_1 = W_2$ = 240 + 153.6 + 60 = 453.6 kips
Total effective seismic weight, $W = W_1 + W_2 + W_3$ = 453.6 + 453.6 + 244.8 = 1152 kips

5.52 Roof effective seismic weight:

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Dead load	=	20 x 1000	=	20 kips
Snow load	=	less than 30 psf	=	0.0 kips

Total roof effective seismic weight, W_2 = 20 + 0.00 = 20 kips
1st story effective seismic weight:

Dead load	=	40 x 1000	=	40 kips

Live load = 0.25x250x1000 = 62.5 kips
Total story effective seismic weight, W_1 = 40 + 62.5 = 102.5 kips
Total effective seismic weight, W = 20 + 102.5 = 122.5 kips

5.53 Effective seismic weight = 100 + 100 + 100 + 120 = 420 kips. ○ ○ ● ○

5.54 Occupancy Category III. Importance Factor I = 1.25
From ASCE 7 Table 12.2-1, for special reinforced concrete shear wall: R = 6 ● ○ ○ ○
$C_S = S_{DS}\ I / R = 1.2 * 1.25 / 6.0 = 0.25$
The *maximum* seismic design coefficient for $T \leq T_L$
$T = 0.46 \leq T_L$ = (8 sec to 16 sec in Region I California)
$C_{S,\,max} = S_{D1}\ I / R\ T = 0.45 * 1.25 / (6.0 \text{ x } 0.46)$
= 0.20 governs (this usually indicates no need to check C_S minimum however it is solved here for completeness)
The *minimum* seismic design coefficient
$C_{S,\,min} = 0.01$
Since $S_1 = 0.65 \geq 0.6$
$C_{S,\,min} = (0.5\ S_1\ I) / R = (0.5 * 0.65 * 1.25) / 6.0 = 0.068$
Thus: $C_S = 0.20$
Effective seismic weight, W = 420 kips:
$V = C_S\ W = 0.20 \text{ x } 420 = 84$ kips

5.55 Since T = 0.46 sec $\leq$ 0.5 Thus: k = 1. V = 84 kips ○ ● ○ ○

Level	Height h_x	h_x^k	Weight w_x	$w_x \cdot h_x^k$	C_{vx}	Lateral Force F_x
4	65	65	100	6500	0.374	31.42
3	50	50	100	5000	0.287	24.11
2	35	35	100	3500	0.201	16.88
1	20	20	120	2400	0.138	11.59
			420	17400	1.00	84

$\boldsymbol{h_x}$ in feet, $\boldsymbol{w_x}$ in kips $\boldsymbol{F_x}$ in kips

5.56 Story shear at 2nd story, V_2 = 31.42 + 24.11+ 16.88 = 72.41 kips ○ ○ ● ○

5.57 Overturning moment at level 1, M_1
M_1 = 31.42 * 45 + 24.11 * 30 + 16.88 * 15 = 2390.4 kips.ft ○ ● ○ ○

5.58 Maximum allowable story drift, Δ_a, for the 4th story, OC = III
From Table 5.10: structures $\leq$ 4 stories for non structural components designed to accommodate story drift, ○ ○ ○ ●
$\Delta_a = 0.02\ h_{sx} = 0.02 * 15' = 0.3$ ft = 3.6 in.

5.59 Occupancy Category II. Importance Factor I = 1.0
From ASCE 7 Table 12.2-1, for special reinforced concrete

shear wall: $R = 7$

$C_S = S_{DS}\ I\ /\ R = 0.9 * 1.0 / 7.0 = 0.129$

$T_a = 0.02\ (3*14)^{0.75} = 0.33$ sec

The *maximum* seismic design coefficient for $T \le T_L$

$T = 0.33 \le T_L =$ (8 sec to 16 sec in Region I California)

$C_{S,\,max} = S_{D1}\ I\ /\ R\ T = 0.5 * 1.0 / (7.0 \times 0.33)$

$= 0.22$

The *minimum* seismic design coefficient

$C_{S,\,min} = 0.01$ ○ ● ○ ○

Since $S_1 = 0.72 \ge 0.6$

$C_{S,\,min} = (0.5\ S_1\ I) / R = (0.5 * 0.72 * 1.0) / 7.0 = 0.05$

Thus: $C_S = 0.129$

Effective seismic weight, $W = 210$ kips:

$V = C_S\ W = 0.129 \times 210 = 27.09$ kips

5.60 Since $T = 0.33$ sec ≤ 0.5 Thus: $k = 1$. $V = 27.09$ kips

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Level	Height h_x	$h_x^{\ k}$	Weight w_x	$w_x \cdot h_x^{\ k}$	C_{vx}	Lateral Force F_x
3	42	42	70	2940	0.50	13.55
2	28	28	70	1960	0.333	9.03
1	14	14	70	980	0.166	4.51
			210	**5880**	**1.00**	**27.09**

$\boldsymbol{h_x}$ in feet, $\boldsymbol{w_x}$ in kips $\boldsymbol{F_x}$ in kips

Story shear at 2nd story, $V_2 = 13.55 + 9.03 = 22.58$ kips

5.61 Maximum overturning moment at the base (level 0), ○ ● ○ ○

$M_{max} = 13.55 * 42 + 9.03 * 28 + 4.51 * 14 = 885.08$ kips.ft

5.62 Actual (inelastic) displacement at the roof level, δ_x

Roof elastic displacement $\delta_{xe} = 1.4$ in. Deflection ○ ○ ● ○

amplification factor C_d from ASCE 7 Table 12.2-1 = 6

$$\delta_x = \frac{C_d . \delta_{xe}}{I} = \frac{6 * 1.4}{1} = 8.4''$$

5.63 3rd story drift = 1.4" - 0.7" = 0.7"

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$$\delta_x = \frac{C_d . \delta_{xe}}{I} = \frac{6 * 0.7}{1} = 4.2''$$

5.64 Maximum allowable story drift, Δ_a, for the 3rd story, OC = II

From Table 5.10: structures $\le$ 4 stories for non structural ● ○ ○ ○

components designed to accommodate story drift,

$\Delta_a = 0.025\ h_{sx} = 0.025 * 14' = 0.35$ ft = 4.2 in.

5.65 Occupancy Category IV. Importance Factor $I = 1.5$

From ASCE 7 Table 12.2-1, for special reinforced concrete

moment frame: $R = 8$ ○ ○ ● ○

C_S = S_{DS} I / R = 1 .2 * 1.5 / 8.0 = 0.225

T_a = 0.016 $(2*16)^{0.90}$ = 0.36 sec

The *maximum* seismic design coefficient for $T \leq T_L$

T = 0.36 $\leq$ T_L = (8 sec to 16 sec in Region I California)

$C_{S,\,max}$ = S_{D1} I / R T = 0.55 * 1.5 / (8.0 x 0.36)

= 0.286

The *minimum* seismic design coefficient

$C_{S,\,min}$ = 0.01

Since S_1 = 0.8 $\geq$ 0.6

$C_{S,\,min}$ = (0.5 S_1 I) / R = (0.5 * 0.8 * 1.5) / 8.0 = 0.075

Thus: C_S = 0.225

Effective seismic weight, W = 80 + 140 = 220 kips:

V = C_S W = 0.225 x 220 = 49.5 kips

5.66 Since T = 0.36 sec $\leq$ 0.5 Thus: k = 1. V = 49.5 kips

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Level	Height h_x	h_x^k	Weight w_x	$w_x \,.\, h_x^k$	C_{vx}	Lateral Force F_x
2	32	32	80	2560	0.533	26.4
1	16	16	140	2240	0.466	23.1
			220	**4800**	**1.00**	**49.5**

$\boldsymbol{h_x}$ in feet, $\boldsymbol{w_x}$ in kips $\boldsymbol{F_x}$ in kips

5.67 Story shear at 1st story, V_1 is the same as base shear. V_1 = 49.5 kips. See Figure Example 5.3.

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Overturning moment at level 1 = 26.4 * 16 = 422.4 kips.

5.68 δ_{xe1} = (F_1 + F_2) / k_1 = (26.4 + 23.1) / 45 = 1.1 in.

δ_{xe2} = δ_{xe1} + (F_2) / k_2 = 1.65 + (23.1) / 45 = 1.61 in.

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5.69 2nd story drift = δ_{xe2} = 1.61" - 1.1" = 0.51"

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From ASCE 7 Table 12.2-1 C_d = 5.5

δ_x = C_d δ_{xe2} / I = 5.5 * 0.51 / 1.5 = 1.87 in.

5.70 Maximum allowable story drift, Δ_a, for the 2nd story, OC = IV

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From Table 5.10: all other structures

Δ_a = 0.01 h_{sx} = 0.01 * 16' = 0.16 ft = 1.92 in.

For moment frames, $\delta_x \leq \Delta_a$ / ρ = 1.92 / 1 = 1.92"

5.71 Stability coefficient, θ, for the 1st story column:

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P_x = unfactored axial load at the roof level = 50 kips

V_x = 1st story shear = V_{x1} = 49.5 kips

1st story drift, Δ_{xe1} = δ_{xe1} = 1.1 - 0.0 = 1.1 in.

$$\Delta_{x1} = \frac{C_d \; \Delta_{xe1}}{I} = \frac{5.5 * 1.1}{1.5} = 4.03 \text{ in.}$$

Stability coefficient,

$$\theta = \frac{P_x \, \Delta_x}{V_x \, h_{sx} \, C_d} = \frac{50 * 4.03}{49.5 * (16 * 12) * 5.5} = 0.0038$$

5.72 Maximum stability coefficient, θ, for the 1st story column:
Shear demand to capacity ratio of the 1st story column (β) = 0.9.

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$$\theta_{\max} = \frac{0.5}{\beta . C_d} \leq 0.25$$

$$\theta_{\max} = \frac{0.5}{\beta \, C_d} = \frac{0.5}{0.9 * 5.5} = 0.10$$

5.73 P-Delta effect is an additional moment and shear on a building's column produced by the axial load acting on the laterally displaced column

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5.74 P-Delta effect is most critical with heavy elements (larger P for concrete than steel) and less stiffness elements (columns rather than shear walls). Concrete moment frame.

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5.75 The P-Delta effect need not to be considered in the design of a building's column when the stability coefficient, θ, is less than or equal to 10%.

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5.76 Since OC = II for both buildings, I = 1.0, and
Both buildings have the same δ_{xe} at 12th story height
$\delta_{xmax\text{-}A} = 4 \ \delta_{xe}$ and $\delta_{xmax\text{-}B} = 5.5 \ \delta_{xe}$

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$$\delta_{MT} = \sqrt{(4 \, \delta_{xe})^2 + (5.5 \, \delta_{xe})^2} = \sqrt{(16 + 30.25) \, \delta_{xe}{}^2} = 6.8 \, \delta_{xe}$$

5.77 OC = III, I = 1.25. From Table 12.2-1, C_d = 4. for steel eccentrically braced frame.
Required setback: $\delta_{x\text{-max}} = 4 \ \delta_{xe} / 1.25 = 3.2 \ \delta_{xe}$

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5.78 *Orthogonal Combinations Procedure* of ASCE 7-§12.5.3 combines 100% of seismic demands from one principal direction plus 30% of seismic demands from orthogonal principal direction.

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5.79 SDC = D. Column part of two orthogonal SFRS
Uplift = 100% * 60 + 30% * 75 = 82.5 kips
Uplift = 100% * 75 + 30% * 60 = 93.0 kips Govern

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5.80 Concrete moment frame structure with fundamental period, T, larger than 3.5 T_s

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5.81 T_s equal 0.7. Thus: 3.5 T_s = 3.5 * 0.7 = 2.45 sec., and
T < 3.5 T_s for all the answers.
From Table 5.12, extreme soft story (vertical irregularity type 1-b) is not permitted for EFLA.

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5.82 Rock soil, $F_a = 1.0$, and $S_{DS} = 2/3\ F_a\ S_S = 2/3*1.0*0.7 = 0.467$
From Table 5.5 based on $S_{DS} = 0.467$, SDC = C ○ ● ○ ○

5.83 $F = 1.1,\ S_{DS} = 0.467,\ W = 120\ \text{kips},\ R = 5$ ASCE 7 Table 12.14-1 ● ○ ○ ○

$$V = \frac{F\ S_{DS}}{R} W = \frac{1.1*0.467}{5} 120 = 12.33\ \text{kips}$$

5.84 $F = 1.1,\ S_{DS} = 0.467,\ w_2 = 60\ \text{kips},\ R = 5$ ASCE 7 Table 12.14-1 ○ ○ ● ○

$$F_2 = \frac{F\ S_{DS}}{R} w_2 = \frac{1.1*0.467}{5} 60 = 6.165\ \text{kips}$$

5.85 Seismic base isolation, friction dampers, and plastic hinging at predetermined locations are all used to reduce the elastic seismic base shear demand on the structures. ○ ○ ○ ●

5.86 Seismic isolation reduces the base shear by increasing the natural period of the structure, and hence reducing the spectral acceleration response. This technique is most effective for structures resting on rock or stiff soil. ● ○ ○ ○

5.87 Friction dampers are used as a passive energy dissipation technique to reduce the spectral acceleration imparted to the structure. ● ○ ○ ○

5.88 seismic effect, Q_E is the forces in beams/columns caused by horizontal seismic forces as determined by the seismic base shear. ○ ○ ● ○

5.89 The redundancy factor, ρ, is used to increase the seismic load on the structure in the case of few seismic force resisting systems and no multiple path of resistance. ○ ○ ○ ●

5.90 The redundancy factor, ρ, for the horizontal seismic effect on a multi-story building assigned to SDC = B equals 1.0. ● ○ ○ ○

5.91 The redundancy factor, ρ, for the horizontal seismic effect on an office building assigned to SDC = D with plan irregularity equals 1.3. ○ ● ○ ○

5.92 $S_{DS} = 2/3\ S_{MS} = 2/3 * 1.1 = 0.737$ $D = 60$ kips ○ ○ ○ ●

$$E_v = \pm 0.2\ S_{DS}\ D = \pm 0.2 * 0.737 * 60 = \pm 8.84\ \text{kips}$$

5.93 $E_h = \pm\ \rho\ Q_E = \pm 1.3 * 45 = \pm 58.5\ kips$ ○ ● ○ ○

5.94 $S_{DS} = 1.14$ $D = 120$ kips ○ ○ ● ○

$$E_v = \pm 0.2\ S_{DS}\ D = \pm 0.2 * 1.14 * 120 = \pm 27.4\ \text{kips}$$

5.95 IBC §1605.2 strength design or load and resistance factor ○ ○ ○ ●

design:
Maximum effects: $1.2D + 1.0E + f_1L + f_2S$
$D = 120$ kips, $L = 60$ kips, $f_1 = 0.5$, $S = 0$, $E = \rho Q_E + 0.2 S_{DS} D$
Maximum axial force = $(1.2 + 0.2 S_{DS}) D + \rho Q_E + 0.5 L$
= (1.2 + 0.2 * 1.14) 120 + 1.3 * 45 + 0.5 * 60 = 259.9 kips

5.96 IBC §1605.2 strength design or load and resistance factor design: ○ ● ○ ○
Minimum effects: $0.9D + 1.0E + 1.6H$
$D = 120$ kips, $H = 0$, $E = - \rho Q_E - 0.2 S_{DS} D$
Minimum axial force = $(0.9 - 0.2 S_{DS}) D - \rho Q_E$
= (0.9 - 0.2 * 1.14) 120 - 1.3 * 45 = 22.14 kips Compression

5.97 Maximum effect of horizontal and vertical seismic forces ○ ○ ● ○
$E_m = E_{mh} + E_v = \Omega_0 Q_E + 0.2 S_{DS} D$
E_m = 3 * 45 + 0.2 * 1.14 * 120 = 162.4 kips

5.98 ASCE 7 12.4.3.2 strength design with overstrength factor: ● ○ ○ ○
Maximum effects: $1.2D + 1.0E_m + f_1L + f_2S$
$D = 120$ kips, $L = 60$ kips, $f_1 = 0.5$, $S = 0$, $E_m = \Omega_0 Q_E + 0.2S_{DS} D$
Maximum axial force = $(1.2 + 0.2 S_{DS}) D + \Omega_0 Q_E + 0.5 L$
= (1.2 + 0.2 * 1.14) 120 + 3 * 45 + 0.5 * 60 = 336.4 kips

5.99 ASCE 7 12.4.3.2 strength design with overstrength factor: ● ○ ○ ○
Minimum effects: $0.9D - 1.0E_m + 1.6H$
$D = 120$ kips, $H = 0$, $E_m = - \Omega_0 Q_E - 0.2 S_{DS} D$
Minimum axial force = $(0.9 - 0.2 S_{DS}) D - \Omega_0 Q_E$
= (0.9 - 0.2 * 1.14) 120 - 3 * 45 = 54.4 kips Tension

5.100 Columns, beams and other structural elements shall be designed to resist special seismic load combination if they have horizontal irregularity 4, vertical irregularity 4. Also, collectors for structures in SDC = C, D, E and F and foundation supporting cantilever columns shall be designed for the maximum seismic loads. ○ ○ ● ○