9.27 What is the maximum length-width ratio for the blocked wood structural panel (WSP) horizontal diaphragms (second floor and roof)?

a. 2:1
b. 2½:1
c. 3:1
d. 4:1

9.28 Given a structure assigned to *Seismic Design Category* C with a typical *subdiaphragm* span of 20 feet, what would be the minimum required depth of each structural *subdiaphragm*?

a. 10′-0″
b. 8′-0″
c. 6′-8″
d. 5′-0″

9.29 What is the minimum seismic design force for structural *subdiaphragms* that are part of a flexible diaphragm in *SDC* = C, D, E or F?

a. $\frac{F_{px}}{L}$ plf
b. $\frac{F_x}{L}$ plf
c. $0.2K_a I_e W_p$
d. 280 plf

Given a single-story wood frame Police Station assigned to *Seismic Design Category* F with wood structural panels used for the flexible roof diaphragm and for the shear walls. The roof diaphragm is to use 19/32″ rated sheathing with 10d common nails (3″ x 0.148″) fastened to 2x nominal framing members, with blocking omitted at intermediate joints. The shear walls are to use 15/32″ Structural I sheathing with 10d common nails (3″ x 0.148″) fastened to 2x nominal framing members. Answer questions 9.30 through 9.33.

9.30 What is the allowable unit shear for the roof diaphragm with seismic loads perpendicular to the continuous panel joints (CASE 1)?

a. 285 plf
b. 255 plf
c. 215 plf
d. 190 plf

9.31 What is the allowable unit shear for a shear wall with 4″ o.c. edge nailing, a height (h) of 12′-0″ and a width (b_s) of 8′-6″ resisting seismic loads?

a. 310 plf
b. 380 plf
c. 460 plf
d. 510 plf

Problem	Answer	Reference / Solution
27	d	p. 1-110 - Vertical Flexible Diaphragm Analysis NOTE: this is a flexible diaphragm and you do not distribute the story shear (or base shear for a 1-story building) based on the shear wall rigidities provided. The rigidities are ignored to determine V_1 & V_2 (i.e., use tributary area). $\therefore\ V_1 = V_2 = V / 2 = 116$ kips / 2 = 58 kips ←
28	a	*ASCE 7-10* p. 63 - *§12.2.5.2* – Cantilever Column Systems *ASCE 7-10 – Table 12.2-1, Type G* refers to *ASCE 7-10 – §12.2.5.2* - " … shall not exceed 15% of the available axial strength, …" ∴ 15% ←
29	d	p. 1-117 to 122 From the figure: $\overline{X}_{CM} = 150' / 2 = 75'$ & $\overline{Y}_{CM} = 100' / 2 = 50'$ $\overline{X}_{CR} = 60'$ & $\overline{Y}_{CR} = 50'$ calculated/inherent eccentricity $e_x = \overline{X}_{CM} - \overline{X}_{CR} = 75' - 60' = 15'$ accidental eccentricity $e_x = \pm 5\%\ L_x = 5\%\ (150') = \pm 7.5'$ The governing (i.e., maximum) force to the shear wall on line 2 will occur when the *CM* is moved nearest to line 2 where the maximum additive torsional shear will occur: $e_{x1} = 15' + 7.5' = 22.5'$ $M_{T1} = V \cdot e_{x1} = 155$ kips $(22.5') = 3488$ kip-ft $\sum Rd^2 = R_1 d_1^2 + R_2 d_2^2 + R_A d_A^2 + R_B d_B^2$ $= 3\ (60')^2 + 2\ (90')^2 + 1.5\ (50')^2 + 1.5\ (50')^2 = 34{,}500$ ft^2 max. $F_2 = V \dfrac{R_2}{R_1 + R_2} + \dfrac{M_{T1} R_2 d_2}{\sum Rd^2}$ $= 155$ kips $(2) / (2 + 3) + 3488$ kip-ft $(2)(90$ ft$) / 34{,}500$ ft^2 $= 62$ kips + 18 kips = 80 kips ←
30	b	p. 1-72 - Redundancy Factor, ρ Redundancy is a characteristic of structures in which multiple paths of resistance to loads are provided. ∴ Redundancy ←
31	b	p. 1-33 to 35 & *2015 IBC* p. 398 - *Tables 1613.3.5(1) & 1613.3.5(2)* Office building w/ fire station in 1st story → *IBC Table 1604.5* → *RC* = IV Site Class E & $S_S = 2.13$ → *Table 3.2* → $S_{DS} = 1.28$ Site Class E & $S_1 = 0.74$ → *Table 3.3* → $S_{D1} = 1.18$ $S_1 = 0.74 < 0.75$ → must use *Tables 1613.3.5(1) & (2)* to determine *SDC* $S_{DS} = 1.28$ & *RC* = IV → *Table 1613.3.5(1)* → *SDC* = D $S_{D1} = 1.18$ & *RC* = IV → *Table 1613.3.5(2)* → *SDC* = D ∴ *SDC* = D ←

<table>
<tr><th>Problem</th><th>Answer</th><th>Reference / Solution</th></tr>
<tr><td>41</td><td>c</td><td>p. 1-88 & ASCE 7-10 p. 76 - §12.11.2.1 – Wall Anchorage Forces
S_{DS} = 0.63
L_f = 150 feet <u>or</u> 110 feet … span of <u>flexible</u> diaphragms, <u>BUT</u> K_a will max out at 2.0 for any flexible diaphragm span > 100 feet
K_a = 2.0 max.
I_e = 1.5 – ASCE 7-10 p. 4 - Table 1.5-2 for Police station (RC = IV)
h_w = height of wall to roof = 14′
h_p = height of parapet (above roof) = 2.67′
$W_p = W_{wall}\left(h_w/2 + h_p\right)$ … for (one-story) walls <u>with</u> a parapet
= (75 psf)(14′/2 + 2.67′) = 725 plf
$F_p = 0.4 S_{DS} K_a I_e W_p$ ASCE 7 (12.11-1)
= 0.4 (0.63)(2.0)(1.5)(725 plf) = <u>548 plf</u> ← governs
$F_p \geq 0.2 K_a I_e W_p$ minimum
= 0.2 (2.0)(1.5)(725 plf) = 435 plf minimum
∴ <u>550 plf</u> ←</td></tr>
<tr><td>42</td><td>a</td><td>p. 1-63 - Structural Separation & ASCE 7-10 p. 77 - §12.12.3
Adjacent buildings on the same property, structural separation -
$\delta_{MT} = \sqrt{\left(\delta_{M1}\right)^2 + \left(\delta_{M2}\right)^2}$ ASCE 7 (12.12-2)
<u>Structure 1</u>:
Medical <u>Office</u> building → IBC Table 1604.5 → RC = II
I_e = 1.0 – ASCE 7-10 p. 4 - Table 1.5-2 for RC = II
Special reinforced concrete shear walls → ASCE 7-10 – Table 12.2-1, Type A.1 or B.4 → C_d = 5
$\delta_{M1} = \dfrac{C_d \delta_{max}}{I_e} = \dfrac{5(1.2")}{1.0} = 6.0''$ ASCE 7 (12.12-1)
<u>Structure 2</u>:
Hospital → IBC Table 1604.5 → RC = IV
I_e = 1.5 – ASCE 7-10 p. 4 - Table 1.5-2 for RC = IV
Steel SMF's → ASCE 7-10 – Table 12.2-1, Type C.1 → C_d = 5½
$\delta_{M2} = \dfrac{C_d \delta_{max}}{I_e} = \dfrac{5.5(4.0")}{1.5} = 14.7''$ ASCE 7 (12.12-1)
$\delta_{MT} = \sqrt{\left(\delta_{M1}\right)^2 + \left(\delta_{M2}\right)^2} = \sqrt{(6.0")^2 + (14.7")^2}$ = <u>15.9 inches</u>
∴ <u>16 inches</u> ←</td></tr>
<tr><td>43</td><td>c</td><td>p. 1-75 - Basic (SD or LRFD) Load Combinations & 2015 IBC p. 358 - §1605.2
IBC equation (16-5) will govern for the <u>maximum</u> axial compression in the column while IBC equation (16-7) will govern for the <u>minimum</u> axial compression in the column.
$E_h = \pm \rho Q_E$ ASCE 7 (12.4-3)
= ± 1.3 (±37 kips) = ± 48 kips
(continued)</td></tr>
</table>

Problem	Answer	Reference / Solution
		$T = T_a = 1.15$ sec $> T_s = 0.60$ sec → *ASCE 7 (12.8-2)* will not govern for C_s $C_S = \frac{S_{D1}}{T(R/I_e)}$ *ASCE 7 (12.8-3)* $= \frac{0.68}{1.15(8/1.0)} = \underline{0.074}$ ← governs C_S shall not be less than: $C_S = 0.044 S_{DS} I_e$ *ASCE 7 (12.8-5)* $= 0.044\ (1.13)(1.0) = 0.050 < 0.074$ In addition, when $S_1 \geq 0.6$, C_S shall not be less than: $C_S = \frac{0.5 S_1}{(R/I_e)}$ *ASCE 7 (12.8-6)* $= \frac{0.5(0.78)}{(8/1.0)} = 0.049 < 0.074$ $V = C_S W$ *ASCE 7 (12.8-1)* = 0.074 (7,250 kips) = 537 kips ∴ 535 kips ←
50	b	p. 1-113 - Chord Force, Figure 8.10 M_{max} & CF_{max} occur at midspan of $L = 100'$ (i.e., at $x = L / 2 = 50'$) … BUT this problem is asking for the Chord Force at "A" where $x = 25'$ which is not the maximum. $V_1 = V_2 = V / 2 = 25$ kips / 2 = 12.5 kips $w_s = F_{p1} / L = V / L$ = (25 kips) / (100′) = 0.25 klf = 250 plf $M_x = \left(\frac{w_s L}{2}\right)x - \frac{w_s x^2}{2}$ = (250 plf)(100′)(25′) / 2 – (250 plf)(25′)^2 / 2 = 234,400 lb-ft $CF_x = \frac{M_x}{d}$ = (234,400 lb-ft) / (40′) = 5860 lbs ∴ 5900 lbf ←
51	a	p. 1-61 & *ASCE 7-10* p. 77 - *§12.12.1* Emergency Operations Center (1st story) → *IBC Table 1604.5* → *RC* = IV 6-story building > 4 stories Steel CBF (All other structures) & *RC* = IV → *ASCE 7-10 – Table 12.12-1* → $\Delta_{ax} \leq 0.010\, h_{sx}$ allowable *story drift ratio* = (Δ_{ax} / h_{sx}) = 0.010 ← ∴ 0.010 ←
52	b	p. 1-33 to 35 & *2015 IBC* p. 398 - *Tables 1613.3.5(1)* & *1613.3.5(2)* Retail warehouse → *IBC Table 1604.5* → *RC* = II "Rock" = Site Class B Site Class B & $S_S = 0.699$ → *Table 3.2* → $S_{DS} = 0.47$ Site Class B & $S_1 = 0.316$ → *Table 3.3* → $S_{D1} = 0.21$ (by interpolation) *(continued)*