Design and Detailing of Structures

PROBLEM 16

A concrete slab designed for a parking garage constructed in Chicago uses normalweight concrete with a compressive strength of 5000 lbf/in^2. The maximum aggregate size is 1 in. The total percentage of air content of the concrete mix is most nearly

(A) 0.06%

(B) 0.45%

(C) 4.5%

(D) 6.0%

Hint: The parking garage in Chicago is exposed to freezing and thawing conditions. Refer to ACI Sec. 19.3.

PROBLEM 17

A reinforced concrete building has a 6 in flat-plate slab on each floor and 12 in diameter columns. The floor-to-floor distance is 13 ft. A column has a factored axial load of 300 kips and equal factored end moments of 100 ft-kips and is subjected to double curvature. The modulus of elasticity of concrete is 3.6×10^6 lbf/in^2 and $kl_u/r = 50$. The column does not have any transverse loads and is not subject to sway. The critical buckling load is most nearly

(A) 400 kips

(B) 680 kips

(C) 1600 kips

(D) 3600 kips

Hint: Use ACI 318 Sec. 6.6.4.4 to determine the critical buckling load.

PROBLEM 18

A two-story office building has a 60 ft × 100 ft rectangular floor plan. Columns spaced 20 ft apart carry the following loads from the roof and second floor. Live load reductions are not permitted.

roof dead load	15 lbf/ft^2
roof live load	20 lbf/ft^2
floor dead load	15 lbf/ft^2
floor live load	80 lbf/ft^2

What is most nearly the total load on an interior first-floor column using the *International Building Code* (IBC) basic load combinations for allowable stress design?

(A) 2.2 kips

(B) 14 kips

(C) 44 kips

(D) 50 kips

Hint: Determine the tributary area for an interior column.

PROBLEM 19

A single-story steel-framed building has columns spaced 15 ft on center in the north/south direction and 20 ft on center in the east/west direction. The columns support a uniform dead load of 40 lbf/ft^2. The building has an ordinary roof with a 1:2 pitch. Using the *International Building Code* (IBC), the minimum live load on an interior column is most nearly

(A) 4.1 kips

(B) 4.9 kips

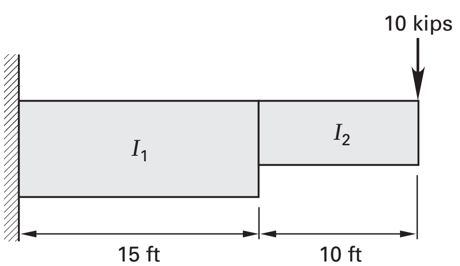
(C) 6.0 kips

(D) 17 kips

Hint: IBC Sec. 1607 covers live loads.

PROBLEM 20

The cantilevered beam shown has a varying moment of inertia. The moment of inertia for the first 15 ft, I_1, is 2000 in^4. The second moment of inertia, I_2, is 1000 in^4. The modulus of elasticity of the beam is 29×10^6 lbf/in^2.



The deflection at the free end of the beam is most nearly

(A) 0.0017 in

(B) 1.7 in

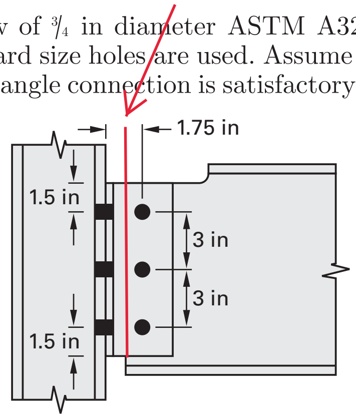
(C) 3.0 in

(D) 3.1 in

Hint: Use the moment-area method to find the deflection of the beam.

The dimension line in the figure should show 1.75 in from the center of the holes to the edge of the beam, not the face of the column. I have drawn a line to show where it should be, but you should just move the current left dimension line and arrow.

A single row of $\frac{3}{4}$ in diameter ASTM A32 used. Standard size holes are used. Assume umn-to-clip angle connection is satisfactory.



(not to scale)

Using LRFD and the AISC *Steel Construction Manual*, the maximum beam reaction is most nearly

(A) 32 kips

(B) 45 kips

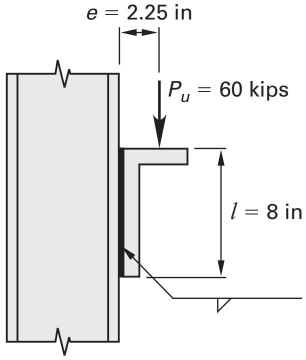
(C) 48 kips

(D) 95 kips

Hint: Refer to Part 10 of the AISC *Steel Construction Manual* for connection design.

PROBLEM 34

An L4 × 8 × $\frac{1}{2}$ angle (LLV) is welded to a W12 × 50 column using the shielded metal arc welding process and E70XX electrodes.



Using LRFD, what is the required fillet weld size if the angle is welded on both sides of the vertical leg only?

(A) $\frac{1}{8}$ in

(B) $\frac{3}{16}$ in

(C) $\frac{1}{4}$ in

(D) $\frac{5}{16}$ in

Hint: The weld is subject to both shear and bending.

An ASTM A992 W18 × 40 composite beam with a 4 in concrete slab supports a live load moment of 140 ft-kips and a dead load moment of 80 ft-kips. The dead load includes the weight of the slab and beam. The construction is unshored.

section modulus of the beam	68.4 in^3
transformed section modulus, measured to the bottom of the section	103 in^3
transformed section modulus, measured to the top of the concrete	350 in^3
transformed section modulus, measured to the steel/concrete interface	1680 in^3

Using ASD, the bending stress in the bottom fibers of the steel beam due to dead load is most nearly

(A) 9.3 kips/in^2

(B) 14 kips/in^2

(C) 39 kips/in^2

(D) 1200 lbf/in^2

Hint: In unshored construction, the beam carries the full dead load.

PROBLEM 36

A single-story concrete loadbearing wall supported at the roof and foundation has the following properties.

wall thickness	12 in
concrete compressive strength	4000 lbf/in^2
effective length factor	1.0
length of wall	16 ft
uniform factored axial load	5 kips/ft

Ignore the self-weight of the wall. If the wall is designed using the simplified method, its maximum height is most nearly

(A) 25.0 ft

(B) 26.9 ft

(C) 31.6 ft

(D) 32.0 ft

Hint: Refer to ACI 318 Sec. 11.5.3.

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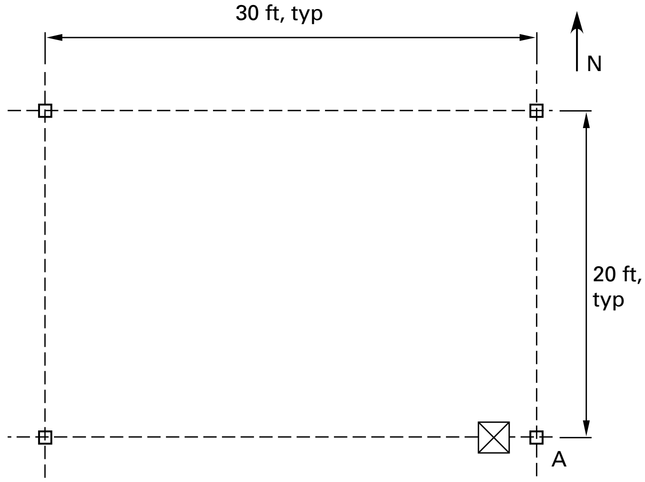
PROBLEM 41

An interior bay of a two-way flat-slab system is supported by 12 in concrete columns as shown.



If no special analysis is used, the maximum size square opening that can be located adjacent to column A, centered on the east/west column centerline, is most nearly

(A) 0.0 ft^2

(B) 0.39 ft^2

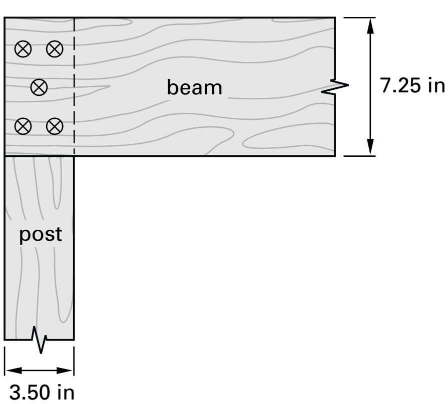
(C) 1.6 ft^2

(D) 2.3 ft^2

Hint: Refer to ACI 318 Chap. 8 on two-way slabs.

PROBLEM 42

A deck is built on the back of a house in a cool humid climate using the design requirements contained in the NDS. A 2 × 8 beam is screwed to the face of a 4 × 4 post as shown. The wood is southern pine. The screws shall have a penetration greater than or equal to ten times the shank diameter. The dead-load plus live-load end reaction of the beam is 605 lbf.



If five 12-gage wood screws are used, what is most nearly the allowable load on the connection?

(A) 450 lbf

(B) 510 lbf

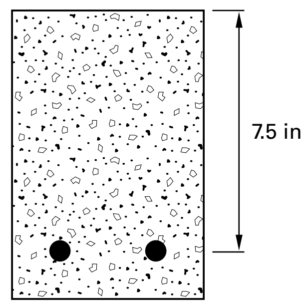
(C) 570 lbf

(D) 800 lbf

Hint: Refer to NDS Chap. 12 for wood screw design values.

PROBLEM 43

The maximum factored shear in the 6 in × 9 in concrete beam shown is 2000 lbf. Assume normalweight concrete with a compressive strength of 3000 lbf/in^2, and the yield stress of the steel reinforcement is 60,000 lbf/in^2.



What is most nearly the required shear reinforcement?

(A) no. 3 U-stirrups at 4.0 in spacing

(B) no. 3 U-stirrups at 4.5 in spacing

(C) no. 3 U-stirrups at 24 in spacing

(D) none

Hint: Refer to ACI 318 Sec. 22.5.

PROBLEM 44

Plywood sheathing is attached to 2 × 12 roof rafters with 6d box nails. The nails penetrate 1 in into the rafters. The rafters are spaced 16 in on center. All members are southern pine. Sustained temperatures do not exceed 100°F. If the design uplift wind load on the roof is 36 lbf/ft^2, the maximum nail spacing is most nearly

(A) 7.8 in

(B) 11 in

(C) 12 in

(D) 14 in

Hint: Refer to NDS Chap. 12

PROBLEM 58

Which of the following statements are true?

I. Mat foundations can be used in areas where the basement is below the ground water table (GWT).

II. Mat foundations always require a top layer of reinforcing bars.

III. Conventional spread footings tolerate larger differential settlements than do mat foundations.

IV. Mat foundations are not suitable where settlement may be a problem.

(A) I only

(B) I and II

(C) III and IV

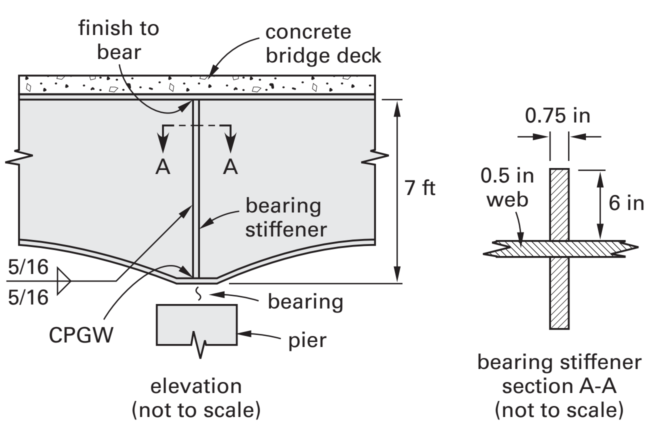
(D) II, III, and IV

Hint: A mat foundation is a large concrete slab in contact with the soil and is commonly used to support several columns or pieces of equipment.

PROBLEM 59

The elevation of an interior girder of a composite two-lane, two-span continuous highway bridge is shown. The web and bearing stiffeners form column sections.

steel specification	ASTM A709
minimum web yield stress	50 kips/in^2
minimum stiffener yield stress	50 kips/in^2
steel modulus of elasticity	29,000 kips/in^2



The factored axial resistance of the effective bearing stiffener column section is most nearly

(A) 570 kips

(B) ~~590~~ kips 600

(C) ~~650~~ kips 670

(D) ~~730~~ kips 750

Hint: Use *AASHTO LRFD Bridge Design Specifications* Sec. 6.

PROBLEM 60

A reinforced concrete building has a 6 in flat-plate slab on each floor and 12 in diameter columns. The floor-to-floor distance is 13 ft. A column has a factored axial load of 300 kips and equal factored end moments of 100 ft-kips and is subjected to double curvature. The modulus of elasticity of concrete is 3.6×10^6 lbf/in^2 and $kl_u/r = 50$. The critical buckling load is 405 kips. If the column does not have any transverse loads and is not subject to sway, the design moment is most nearly

(A) −1600 ft-kips

(B) 1600 ft-kips

(C) 8100 ft-kips

(D) 13,000 ft-kips

Hint: Use magnified moments to determine the design moment.

Construction Administration

PROBLEM 61

According to the AISC *Steel Construction Manual*, which of the following is NOT an acceptable method of assessing stability requirements of a steel building?

(A) direct a nalysis method

(B) effective length method

(C) first-order analysis method

(D) plastic drift analysis method

Hint: Refer to Table 2-2 of the AISC *Steel Construction Manual*.

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PROBLEM 62

A reinforced brick masonry residence located in Los Angeles is designed using strength design provisions. According to *Building Code Requirements for Masonry Structures* (TMS 402/602), which of the following requirements must be met during design and construction of this building?

I. Verify placement of reinforcement prior to grouting.

II. Verify placement of grout continuously during construction.

III. Observe preparation of mortar specimens.

IV. Verify the compressive strength of masonry prior to construction and every 5000 ft^2 during construction.

(A) I and II only

(B) I and III only

(C) I, II, and III

(D) none of the above

Hint: These requirements are part of a quality assurance program.

Temporary Structures and Other Topics

PROBLEM 63

According to the *International Building Code* (IBC), which of the following statements is true regarding construction documents?

I. The size and location of all structural members must be shown.

II. Information on seismic loads need not be shown if wind governs the design of the lateral force-resisting system.

III. The design wind speed must be shown regardless of whether wind loads govern the design of the lateral force-resisting system.

IV. Live load reductions must be shown.

(A) I only

(B) I and II only

(C) I and III only

(D) I, III, and IV only

Hint: Refer to IBC Sec. 1603.

PROBLEM 64

A 30 ft tall office building under construction is located 15 ft from its lot line. According to the *International Building Code* (IBC), which safeguard is required to protect pedestrians during construction?

(A) construction railings

(B) barriers

(C) barriers and covered walkway

(D) none

Hint: Refer to IBC Chap. 33.

PROBLEM 65

Design of formwork must include consideration of all of the following factors EXCEPT

(A) rate and method of placing concrete

(B) special form requirements

(C) design loads, including vertical, horizontal, and impact loads

(D) construction loads, including vertical, horizontal, and impact loads

Hint: Refer to ACI 318 Chap. 26.11.

PROBLEM 66

A concrete masonry non-loadbearing wall is constructed using 12 in units and running bond. The wall is subject to out-of-plane wind loads and is vertically reinforced with grade 60 no. 5 bars located in the center of the wall spaced at 24 in on center. The compressive strength of masonry is 2000 lbf/in^2. Using strength design, the moment capacity of the wall is most nearly

(A) 2300 ft-lbf/ft

(B) 3900 ft-lbf/ft

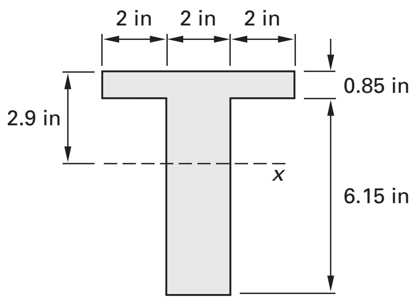
(C) 4300 ft-lbf/ft

(D) 7800 ft-lbf/ft

Hint: See TMS 402 Sec. 9.3.5.

The distance from the top of the section to the centroid is

$$
\begin{aligned}
y_c &= \frac{\sum A_i y_{ci}}{\sum A_i} \\
&= \frac{(5.1\ \text{in}^2)\left(\dfrac{0.85\ \text{in}}{2}\right) + (12.3\ \text{in}^2)\left(\dfrac{6.15\ \text{in}}{2} + 0.85\ \text{in}\right)}{5.1\ \text{in}^2 + 12.3\ \text{in}^2} \\
&= 2.9\ \text{in} \quad [\text{from top of section}]
\end{aligned}
$$



The shear stress is

$$\tau = \frac{VQ}{Ib}$$

Calculate I and Q. For a rectangular section,

$$
\begin{aligned}
I_1 &= \frac{bh^3}{12} = \frac{(6\ \text{in})(0.85\ \text{in})^3}{12} = 0.31\ \text{in}^4 \\
I_2 &= \frac{bh^3}{12} = \frac{(2\ \text{in})(6.15\ \text{in})^3}{12} \\
&= 38.8\ \text{in}^4
\end{aligned}
$$

The moment of inertia about the centroid is

$$
\begin{aligned}
I_x &= \sum (I_i + A_i d_i^2) \\
&= 0.31\ \text{in}^4 + (5.1\ \text{in}^2)\left(2.9\ \text{in} - \frac{0.85\ \text{in}}{2}\right)^2 + 38.8\ \text{in}^4 \\
&\quad + (12.3\ \text{in}^2)\left(7.0\ \text{in} - \frac{6.15\ \text{in}}{2} - 2.9\ \text{in}\right)^2 \\
&= 83.3\ \text{in}^4
\end{aligned}
$$

The statical moment of the area, Q, is the product of the area above or below the point in question and the distance from the centroidal axis to the centroid of the area. Looking at the area below the centroid,

$$
\begin{aligned}
Q &= A\bar{y} = \tfrac{1}{2} b (h - y_c)^2 \\
&= \left(\frac{1}{2}\right)(2\ \text{in})(7.0\ \text{in} - 2.9\ \text{in})^2 \\
&= 16.8\ \text{in}^3
\end{aligned}
$$

The shear stress at the centroid is

$$
\begin{aligned}
\tau &= \frac{VQ}{Ib} = \frac{(100\ \text{lbf})(16.8\ \text{in}^3)}{(83.3\ \text{in}^4)(2\ \text{in})} \\
&= 10.1\ \text{lbf/in}^2 \quad (10\ \text{lbf/in}^2)
\end{aligned}
$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect solution miscalculates the location of the centroid of the section. The distance from the centroid of area A_2 to the top of the section fails to include the 0.85 in thickness of area A_1.

(B) This incorrect solution miscalculates A_2 and carries the mistake throughout the subsequent calculations. The length of A_2 is taken as the overall length of 7.0 in instead of the actual length of 6.15 in.

(D) This incorrect solution does not properly calculate the transformed moment of inertia. It directly adds the moments of inertia for each area instead of calculating the transformed moment of inertia.

SOLUTION 11

The fixed-end moments (FEM) at joint B, as taken from a reference text, are

$$
\begin{aligned}
\text{FEM}_{\text{BA}} &= \frac{P}{L^2}\left(b^2 a + \frac{a^2 b}{2}\right) \\
&= \left(\frac{100\ \text{kips}}{(14\ \text{ft})^2}\right)\left((6\ \text{ft})^2(8\ \text{ft}) + \frac{(8\ \text{ft})^2(6\ \text{ft})}{2}\right) \\
&= 244.9\ \text{ft-kips}
\end{aligned}
$$

$$\text{FEM}_{\text{BC}} = \frac{wL^2}{30} = \frac{\left(50\ \dfrac{\text{kips}}{\text{ft}}\right)(21\ \text{ft})^2}{30} = 735.0\ \text{ft-kips}$$

The unbalanced moment at joint B is

$$
\begin{aligned}
&\text{FEM}_{\text{BA}} - \text{FEM}_{\text{BC}} \\
&\quad = 244.9\ \text{ft-kips} - 735.0\ \text{ft-kips} \\
&\quad = -490.1\ \text{ft-kips} \quad (490\ \text{ft-kips, clockwise})
\end{aligned}
$$

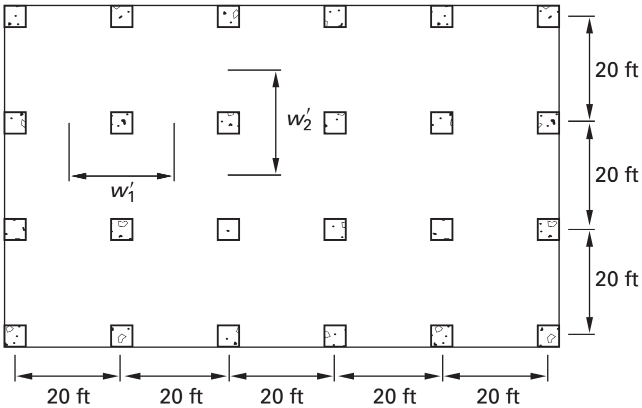
The answer is (A).

Why Other Options Are Wrong

(C) This incorrect solution reverses the distribution factors at joint B, putting the distribution factor for BA at BC and vice versa.

The tributary area for an interior column is

$$A = w_1'w_2' = (20\ \text{ft})(20\ \text{ft}) = 400\ \text{ft}^2$$



From IBC Sec. 1605.3.1, the applicable basic load combinations for allowable stress design are

$$D + L_{\text{floor}}$$
$$D + L_{\text{roof}}$$
$$D + 0.75(L_{\text{floor}} + L_{\text{roof}})$$
$$0.6D$$

Since the floor live load is 80 lbf/ft^2, partition loads need not be added (IBC Sec. 1607.5).

By inspection, $0.6D$ does not control.

The total uniform load per unit area is the larger of

$$\begin{aligned} w_{\text{uniform}} &= (D_{\text{roof}} + D_{\text{floor}}) + L_{\text{floor}} \\ &= \left(15\ \frac{\text{lbf}}{\text{ft}^2} + 15\ \frac{\text{lbf}}{\text{ft}^2}\right) + \boxed{80}\ \frac{\text{lbf}}{\text{ft}^2} \\ &= \boxed{110}\ \text{lbf/ft}^2 \end{aligned}$$

$$\begin{aligned} w_{\text{uniform}} &= (D_{\text{roof}} + D_{\text{floor}}) + L_{\text{roof}} \\ &= \left(15\ \frac{\text{lbf}}{\text{ft}^2} + 15\ \frac{\text{lbf}}{\text{ft}^2}\right) + 20\ \frac{\text{lbf}}{\text{ft}^2} \\ &= 50\ \text{lbf/ft}^2 \end{aligned}$$

$$\begin{aligned} w_{\text{uniform}} &= (D_{\text{roof}} + D_{\text{floor}}) + 0.75(L_{\text{floor}} + L_{\text{roof}}) \\ &= \left(15\ \frac{\text{lbf}}{\text{ft}^2} + 15\ \frac{\text{lbf}}{\text{ft}^2}\right) + (0.75)\left(\boxed{80}\ \frac{\text{lbf}}{\text{ft}^2} + 20\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= \boxed{105}\ \text{lbf/ft}^2 \end{aligned}$$

The total column load is

$$\begin{aligned} P = w_{\text{uniform}}A &= \left(\boxed{110}\ \frac{\text{lbf}}{\text{ft}^2}\right)(400\ \text{ft}^2) \\ &= \boxed{44}{,}000\ \text{lbf} \quad (\boxed{44}\ \text{kips}) \end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect solution uses tributary width instead of tributary area when calculating the total load on the column.

(B) This incorrect solution does not include the second-floor loads when calculating the total load on the column.

(D) This incorrect solution ~~does not consider the appli-~~ includes a 15 lbf/ft^2 partition load in the total floor live load.

SOLUTION 19

Section 1607.12 of the IBC states that roofs must be designed for the appropriate live loads. The minimum uniformly distributed roof live loads are given in IBC Table 1607.1 and are permitted to be reduced per IBC Eq. 16-26. The minimum roof live load, L_o, is given in IBC Table 1607.1 as 20 lbf/ft^2 for an ordinary pitched roof. The reduced live load is

$$L_r = L_oR_1R_2 \quad \text{[IBC Eq. 16-26]}$$

The reduction factors are based on the tributary area of the structural member and the slope of the roof. The tributary area of the column is

$$A_t = (20\ \text{ft})(15\ \text{ft}) = 300\ \text{ft}^2$$

The number of inches of rise per foot of the roof is

$$F = 6$$

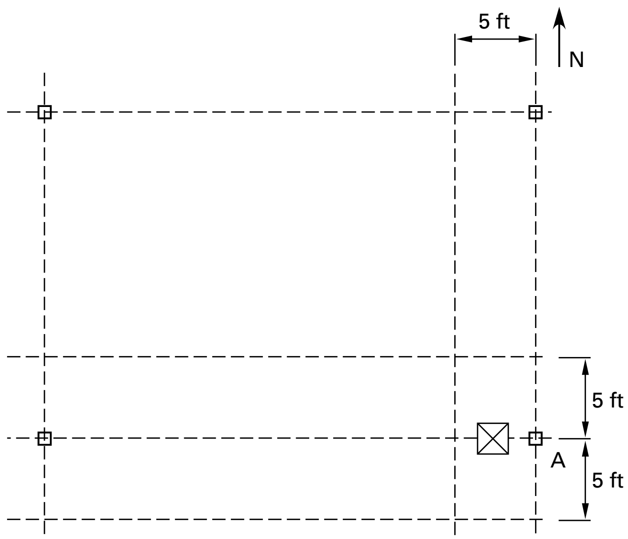
The reduction factors are calculated using IBC Eq. 16-28 for R_1 if 200 ft$^2 < A_t <$ 600 ft^2 and using IBC Eq. 16-31 for R_2 if $4 < F < 12$.

$$\begin{aligned} R_1 &= 1.2 - 0.001A_t \\ &= 1.2 - (0.001)(300) \\ &= 0.90 \\ R_2 &= 1.2 - 0.05F \\ &= 1.2 - (0.05)(6) \\ &= 0.90 \end{aligned}$$

The reduced live load is

$$\begin{aligned} L_r &= L_oR_1R_2 \\ &= \left(20\ \frac{\text{lbf}}{\text{ft}^2}\right)(0.90)(0.90) \\ &= 16.2\ \text{lbf/ft}^2 \end{aligned}$$

The lesser value controls.



The opening falls within intersecting column strips. ACI 318 Sec. 8.5.4.2(b) limits openings in intersecting column strips to not more than 1/8 the width of the column strip in either span.

$$a = b \leq \left(\frac{1}{8}\right)(10 \text{ ft}) = 1.25 \text{ ft}$$

The maximum opening is

$$ab = (1.25 \text{ ft})^2 = 1.56 \text{ ft}^2 \quad (1.6 \text{ ft}^2)$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect solution assumes that since no special analysis is done, no openings are permitted.

(B) In this incorrect solution, one-half the column strip width is used instead of the full column strip width in calculating the opening size.

(D) This incorrect solution does not use the lesser value in calculating the column strip widths according to ACI 318 Sec. 8.4.1.5.

SOLUTION 42

The beam-to-post connection shown imparts a lateral load on the wood screws. Because two wood members are joined in this connection, the connection is subjected to single shear. Appendix I of the NDS illustrates connections in single and double shear. Use NDS Chap. 11 and Chap. 12 and NDS Table 12L to determine the allowable lateral load on the connection.

The allowable lateral design value for a single connector is given in NDS Table 11 3.1 as

$$Z' = ZC_DC_MC_tC_gC_\Delta C_{eg}C_{di}C_{tn}$$

In this case, C_D is 1.0 for dead load plus live load, since the load duration factor for the shortest duration load applies (NDS Sec. 2.3.2 and App. B). C_M is 0.7, since the moisture content of this deck built in a humid climate will exceed 19% (NDS Table 11.3.3). Values for C_g and C_Δ depend on the diameter of the screw, D. Since $D <$ 0.25 in for a 12-gage screw ($D = 0.216$ in), C_g and C_Δ equal 1.0 (NDS Sec. 11 3.6.1 and Sec. 12.5.1). C_{eg}, C_{di}, and C_{tn} do not apply. C_t is 1.0 since it is a cool climate.

Therefore,

$$Z' = ZC_DC_MC_tC_gC_\Delta = Z(1.0)(0.7)(1.0)(1.0)(1.0)$$

The cut-thread wood screw design values are given in NDS Table 12L. First, determine the side-member thickness. The side member is the 2×8 beam. Using Table 1B of the NDS Supplement, the side-member thickness is 1.5 in. Next, find the column for southern pine and the row for a 12-gage wood screw. The tabulated design value is

$$\begin{aligned} Z &= 161 \text{ lbf} \\ Z' &= ZC_DC_MC_tC_gC_\Delta = (161 \text{ lbf})(1.0)(0.7)(1.0)(1.0)(1.0) \\ &= 113 \text{ lbf} \quad \text{[per screw]} \end{aligned}$$

The capacity of the connection is

$$\begin{aligned} Z'_{\text{total}} &= (5 \text{ screws})\left(113 \ \frac{\text{lbf}}{\text{screw}}\right) \\ &= 565 \text{ lbf} \quad (570 \text{ lbf}) \end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect solution misreads NDS Table 12L and uses the value for a 10-gage screw ($Z = 128$ lbf) instead of the 12-gage screw value.

(B) This incorrect solution uses the wrong load duration factor. The load duration factor for the shortest-duration load applies. This solution uses the smallest load duration factor, $C_D = 0.9$.

(D) This incorrect solution fails to apply the adjustment factors to the tabulated design value to determine the allowable design value.

SOLUTION 43

Per ACI 318 Sec. 22.5.5.1, the nominal shear strength provided by the concrete alone is

$$\begin{aligned} V_c &= 2\sqrt{f_c'}\,b_w d = 2\sqrt{3000\ \frac{\text{lbf}}{\text{in}^2}}(6\text{ in})(7.5\text{ in}) \\ &= 4929.5\text{ lbf} \\ \phi V_c &= 0.75V_c = (0.75)(4929.5\text{ lbf}) \\ &= 3697\text{ lbf} \end{aligned}$$

Based on ACI 318 Sec. 9.6.3.1, a beam with depth of less than 10 in and $0.5\phi V_c < V_u < \phi V_c$ requires no minimum shear reinforcement.

$$\begin{aligned} 0.5\phi V_c &= (0.5)(3697\text{ lbf}) = 1849\text{ lbf} \\ V_u &= 2000\text{ lbf} \quad [> 1849\text{ lbf and} < 3697\text{ lbf}] \\ h &= 9\text{ in} \quad [< 10\text{ in}] \end{aligned}$$

Since both conditions are met, no shear reinforcement is required.

The answer is (D).

Why Other Options Are Wrong

(A) This incorrect solution misses the exception, given in ACI 318 Sec. 9.6.3.1, to the minimum shear requirement and mistakenly uses the minimum shear reinforcement requirement found in Sec. 9.7.6.2.2.

(B) This incorrect solution misses the exception, given in ACI 318 Sec. 9.6.3.1, to the minimum shear requirement and uses h for d when calculating the maximum stirrup spacing.

(C) This incorrect solution misses the exception, given in ACI 318 Sec. 9.6.3.1, to the minimum shear requirement and incorrectly uses the largest stirrup spacing found in ACI 318 Sec. 9.7.6.2.2 instead of the smallest.

SOLUTION 44

Section 12.2.3 of the NDS gives the withdrawal values for a single nail. From NDS Table 12.3.3A, the specific gravity of southern pine is 0.55 for E=1,700,000 lbf/in^2

NDS Table 12P gives the diameter of a 6d box nail as 0.099 in.

From NDS Table 12.2C, the tabulated withdrawal design value for a specific gravity of 0.55 and a nail diameter of 0.099 in is 31 lbf per inch of penetration.

The design withdrawal value is

$$\begin{aligned} C_D &= 1.6 \quad \text{[for wind load combinations]} \\ C_M &= 1.0 \quad \text{[for moisture content } \leq 19\%\text{]} \\ C_t &= 1.0 \quad \text{[according to NDS Table 11.3.4]} \\ C_{\text{tn}} &= 1.0 \quad \text{[for non-toe-nailed connections]} \end{aligned}$$

$$\begin{aligned} W' &= WC_DC_MC_tC_{\text{tn}} \\ &= (31\text{ lbf})(1.6)(1.0)(1.0)(1.0) \\ &= 49.6\text{ lbf} \end{aligned}$$

Calculate the maximum area per nail.

$$A = \frac{W'}{w} = \frac{49.6\text{ lbf}}{36\ \frac{\text{lbf}}{\text{ft}^2}} = 1.38\text{ ft}^2$$

If the rafters are spaced at 16 in on center,

$$s_r = (16\text{ in})\left(\frac{1\text{ ft}}{12\text{ in}}\right) = 1.33\text{ ft}$$

The maximum nail spacing is

$$\begin{aligned} s_n &= \frac{A}{s_r} = \left(\frac{1.38\text{ ft}^2}{1.33\text{ ft}}\right)\left(12\ \frac{\text{in}}{\text{ft}}\right) \\ &= 12.4\text{ in} \quad (12\text{ in}) \end{aligned}$$

The answer is (C).

Why Other Options Are Wrong

(A) This incorrect solution neglects the load duration factor in calculating the withdrawal capacity.

(B) This incorrect solution uses a rafter spacing of 1.5 ft instead of 1.33 ft.

(D) This incorrect solution bases the tabulated withdrawal value on the diameter of a 6d common nail (0.113 in) instead of a 6d box nail.

SOLUTION 45

The temperature factor, C_t, is discussed in NDS Sec. 2.3.3. The temperature factor applies to members subjected to sustained exposure to elevated (over 100° F) temperatures. Because the statement applies to members subjected to extreme cold, not heat, statement I is false.

The volume factor, C_V, is discussed in NDS Sec. 5.3 on structural glued laminated timber and in Sec. 8.3 on structural composite lumber. Statement II is true.

Statement II is true. Mat foundations require both top and bottom reinforcement because both positive and negative moments are developed in the foundation.

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution only identifies statement I as true and misses the fact that statement II is also true.

(C) This incorrect solution mistakenly identifies the two solutions that are false rather than those that are true. Statements III and IV are false. Mat foundations are suitable where settlements may be a problem or where settlements may be large due, in part, to their rigidity. The rigidity of the mat tends to bridge over areas of erratic soil types or voids.

(D) This incorrect solution wrongly identifies statements III and IV as true. Mat foundations are suitable where settlements may be a problem or where settlements may be large due, in part, to their rigidity. The rigidity of the mat tends to bridge over areas of erratic soil types or voids. Statements III and IV are false.

SOLUTION 59

The design of bearing stiffeners is found in AASHTO Sec. 6.10.11.2. According to this section, the factored axial resistance, P_r, is determined in accordance with AASHTO Sec. 6.9.2.1. Since the bearing stiffener is a non-composite (steel only) element, use Sec. 6.9.4.

$$P_r = \phi_c P_n \quad \text{[AASHTO Eq. 6.9.2.1-1]}$$

From AASHTO Sec. 6.5.4.2, the resistance factor for axial compression, ϕ_c, is 0.9. The selection of the equation for nominal axial resistance, P_n, depends on the ratio of P_e/P_o. Using AASHTO Table 6.9.4.1.1-1, the elastic critical buckling resistance, P_e, is found as

$$P_e = \left(\frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2}\right) A_g \quad \text{[AASHTO Eq. 6.9.4.1.2-1]}$$

$$P_o = QF_y A_g$$

According to Sec. C6.9.4.1.1, Q is always taken as 1.0 for bearing stiffeners.

Determine the properties of the bearing stiffeners.

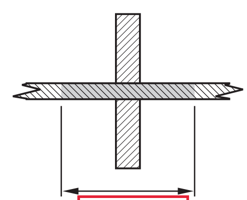
Check that the minimum width of the stiffener, b_t, of 6 in meets the AASHTO requirement. The thickness of the projecting stiffener element, t_p, is given as 0.75 in.

$$\begin{aligned} b_t &\le 0.48 t_p \sqrt{\frac{E}{F_{ys}}} \quad \text{[AASHTO Eq. 6.10.11.2.2-1]} \\ &= (0.48)(0.75\ \text{in})\sqrt{\frac{29{,}000\ \frac{\text{kips}}{\text{in}^2}}{50\ \frac{\text{kips}}{\text{in}^2}}} \\ &= 8.67\ \text{in} \quad [b_t = 6\ \text{in, OK}] \end{aligned}$$

The effective web width is given in AASHTO Sec. 6.10.11.2.4b as

$$w = 2(9t_w) = (2)\big((9)(0.5\ \text{in})\big) = 9.75\ \text{in}$$

Add dimension lines on each side of the vertical element to the edge of the shaded area and label the dimension as $9t_w$



9.75 in

The cross-sectional area of the bearing stiffener column section is the sum of the area of the bearing stiffener and the area of the beam web.

9.75 in

$$\begin{aligned} A_s &= 2b_t t_p + w t_{\text{web}} \\ &= (2)(6\ \text{in})(0.75\ \text{in}) + (9\ \text{in})(0.5\ \text{in}) \\ &= 13.875\ \text{in}^2 \end{aligned}$$

Determine the moment of inertia about the web centerline.

$$\begin{aligned} I &= \frac{bh^3}{12} = \frac{t_p(b_t + t_{\text{web}} + b_t)^3}{12} + \frac{(w - t_p)(t_{\text{web}})^3}{12} \\ &= \frac{(0.75\ \text{in})(6\ \text{in} + 0.5\ \text{in} + 6\ \text{in})^3}{12} \\ &\quad + \frac{(9\ \text{in} - 0.75\ \text{in})(0.5\ \text{in})^3}{12} \\ &= 122\ \text{in}^4 \end{aligned}$$

The radius of gyration for the steel section is

$$r_s = \sqrt{\frac{I}{A_s}} = \sqrt{\frac{122\ \text{in}^4}{13.875\ \text{in}^2}} = 2.97\ \text{in}$$

Determine the column's slenderness ratio. From AASHTO Sec. 6.10.11.2.4a, the effective length factor, K, equals 0.75.

$$\frac{Kl}{r_s} = \frac{(0.75)(7.0\text{ ft})\left(12\ \frac{\text{in}}{\text{ft}}\right)}{2.97\text{ in}} = 21.2$$

The elastic critical buckling resistance is

$$\begin{aligned} P_e &= \left(\frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2}\right)A_g \\ &= \frac{\pi^2\left(29{,}000\ \frac{\text{kips}}{\text{in}^2}\right)(13.875\text{ in}^2)}{(21.2)^2} \\ &= 8836\text{ kips} \end{aligned}$$

The equivalent nominal yield resistance is

$$\begin{aligned} P_o &= QF_yA_g = (1.0)\left(50\ \frac{\text{kips}}{\text{in}^2}\right)(13.875\text{ in}^2) \\ &= 694\text{ kips} \end{aligned}$$

Check the nominal compressive resistance criteria per AASHTO Sec. 6.9.4.1.1.

$$\frac{P_e}{P_o} = \frac{8836\text{ kips}}{694\text{ kips}} = 12.7 > 0.44$$

Use AASHTO Eq. 6.9.4.1.1-1.

$$\frac{P_o}{P_e} = \frac{694\text{ kips}}{8836\text{ kips}} = 0.079$$

The nominal compressive resistance is

$$\begin{aligned} P_n &= \left(0.658^{P_o/P_e}\right)P_o = \left(0.658^{0.079}\right)(694\text{ kips}) \\ &= 671\text{ kips} \end{aligned}$$

For the column section, the factored axial resistance of the effective bearing stiffener is

$$\begin{aligned} P_r &= \phi_c P_n = (0.9)(671\text{ kips}) \\ &= 604\text{ kips} \quad (600\text{ kips}) \end{aligned}$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution uses a value of 1.0 for K instead of 0.75.

(C) This incorrect solution finds the nominal resistance instead of the factored resistance.

(D) This incorrect solution mistakenly calculates the factored axial resistance as P_n/ϕ.

SOLUTION 60

When slenderness must be considered in the design of compression members, the magnified moment procedure can be used if a more refined analysis is not performed.

ACI 318 Sec. 6.6.4.5 contains the provisions for magnified moments in nonsway frames. According to ACI 318 Sec. 6.2.5, if $kl_u/r \le 34 - 12(M_1/M_2)$ and is no more than 40 for columns braced against sidesway, slenderness can be ignored.

The unsupported length of a compression member is taken as the clear distance between floor slabs.

$$l_u = (13.0\text{ ft})\left(12\ \frac{\text{in}}{\text{ft}}\right) - 6\text{ in} = 150\text{ in}$$

For a 12 in diameter column,

$$\begin{aligned} I_g &= \frac{\pi d^4}{64} = \frac{\pi(12\text{ in})^4}{64} = 1018\text{ in}^4 \\ \frac{kl_u}{r} &= 50 \quad [\text{given}] \\ M_1 &= -100\text{ ft-kips} \quad \left[\begin{array}{c}\text{for columns bent in} \\ \text{double curvature}\end{array}\right] \\ M_2 &= 100\text{ ft-kips} \end{aligned}$$

$$\begin{aligned} (34 - 12)\left(\frac{M_1}{M_2}\right) &= 34 - (12)\left(\frac{-100\text{ ft-kips}}{100\text{ ft-kips}}\right) \\ &= 46 \quad [\text{so } 40 < kl_u/r] \end{aligned}$$

Therefore, slenderness must be considered, and magnified moments can be used.

The magnified moment for non-sway columns is given by ACI 318 Eq. 6.6.4.5.1.

$$M_c = \delta_{\text{ns}} M_2$$

$$\delta_{\text{ns}} = \frac{C_m}{1 - \frac{P_u}{0.75P_c}} \ge 1.0 \quad [\text{ACI 318 Eq. 6.6.4.5.2}]$$

P_u, is zero. From TMS 402 Sec. 9.1.4.4, the strength reduction factor, ϕ, for masonry subject to flexure is 0.90.

$$a = \frac{A_s f_y + \frac{P_u}{\phi}}{0.80 f'_m b} = \frac{(0.31\ \text{in}^2)\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right) + \frac{0\ \text{lbf}}{0.90}}{(0.80)\left(2000\ \frac{\text{lbf}}{\text{in}^2}\right)(24\ \text{in})}$$

$$= 0.4844\ \text{in}$$

Change 0.31 in² to 0.31 in²/24 in and add 12 in/ft conversion after the 60,000 lbf/in²

The nominal moment capacity of the wall is

$$M_n = \left(\frac{P_u}{\phi} + A_s f_y\right)\left(d - \frac{a}{2}\right)$$

$$= \left(\frac{0\ \text{lbf}}{0.90} + (0.31\ \text{in}^2)\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right)\right) \times \left(\frac{5.81\ \text{in} - \frac{0.4844\ \text{in}}{2}}{12\ \frac{\text{in}}{\text{ft}}}\right)$$

$$= \boxed{4315}\ \text{ft-lbf}$$

The moment capacity of the wall is

$$\phi M_n = (0.90)(\boxed{4315}\ \text{ft-lbf})$$

$$= \boxed{3884\ \text{ft-lbf/ft}\ (3900\ \text{ft-lbf/ft})}$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution uses a yield strength, f_y, of 36,000 lbf/in² instead of 60,000 lbf/in².

(C) This incorrect solution neglects to multiply the nominal moment by the strength reduction factor, ϕ, of 0.9.

(D) This incorrect solution does not consider the spacing of the reinforcement in the calculations.

If the load from the joists is 3700 lbf, the load on each anchor bolt is most nearly

(A) 460 lbf shear, 390 lbf tension

(B) 820 lbf shear, 0 lbf tension

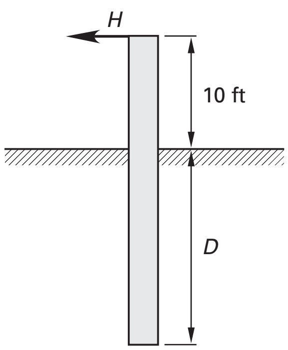
(C) 820 lbf shear, 690 lbf tension

(D) 3700 lbf shear, 3100 lbf tension

Hint: Anchor bolts are subject to both shear and tension.

PROBLEM 20

The cantilevered sheet piling shown has a concentrated lateral load, H, of 10 kips spaced 2 ft on center, horizontally along the top of the piling. The soil specific weight is 110 lbf/ft^3, and the angle of internal friction of the soil is 30°.



If the factor of safety for the minimum depth is 1.3, the total length of the sheet pile is most nearly

(A) 21 ft

(B) 26 ft

(C) 31 ft

(D) 38 ft

Hint: The solution for this problem is an iterative one.

PROBLEM 21

A three-story structural steel-framed office building is classified as a seismic design category B structure. The floor-to-floor height is 12 ft, and the structure is rectangular in plan. The interior partitions, ceilings, and walls have not been designed to accommodate story drifts. According to ASCE/SEI7, the maximum design story drift at the second story is

(A) 0.24 in

(B) 2.2 in

(C) 2.9 in

(D) 3.6 in

Hint: Refer to ASCE/SEI7 Sec. 12.12.

PROBLEM 22

A 10 ft high, 20 ft long special reinforced masonry shear wall is subjected to in-plane seismic loading. The wall is constructed from 12 in concrete masonry units laid in running bond and type S mortar. All reinforcement is adequate to resist the applied shear loads. Which shear reinforcement most nearly complies with the minimum seismic reinforcement requirements?

(A) one no. 5 bar in horizontal bond beams, spaced 24 in on center

(B) one no. 5 bar in horizontal bond beams, spaced 24 in on center, and no. 5 vertical bars, spaced at 24 in on center

(C) two no. 4 bars in horizontal bond beams, spaced 48 in on center, and no. 5 vertical bars, spaced at 24 in on center

(D) two no. 5 bars in horizontal bond beams, spaced 24 in on center, and no. 6 vertical bars, spaced 16 in on center

Hint: Use TMS 402 Sec. 7.3.2.6.

PROBLEM 23

Which of the following is NOT a requirement of a structural steel special moment-resisting frame (SMF)?

(A) Beam-to-column connections used in a seismic load-resisting system shall be capable of sustaining a story drift angle of at least 0.02 rad.

(B) Beam-to-column connections may be prequalified for SMF in accordance with AISC 341 Sec. K1.

(C) Continuity plates shall be welded to column flanges using CJP groove welds.

(D) The thickness of the continuity plates in a one-sided connection shall be at least one-half of the thickness of the beam flange.

Hint: Refer to ASCE/SEI7 Chap. 14.

PROBLEM 24

Which statement(s) regarding *Special Design Provisions for Wind and Seismic* (SDPWS) is/are true?

I. Wood-framed shear walls sheathed with gypsum wallboard are permitted to resist seismic forces in seismic design category C.

II. Wood-framed shear walls sheathed with gypsum wallboard can be constructed as blocked or unblocked.

III. Wood-frame shear walls with particleboard sheathing are permitted to resist seimic forces in seismic design category D.

IV. Single-layer lumber used to diagonally sheathe wood-frame shear walls must have a nominal thickness of at least 1 in.

(A) I only

(B) II and III

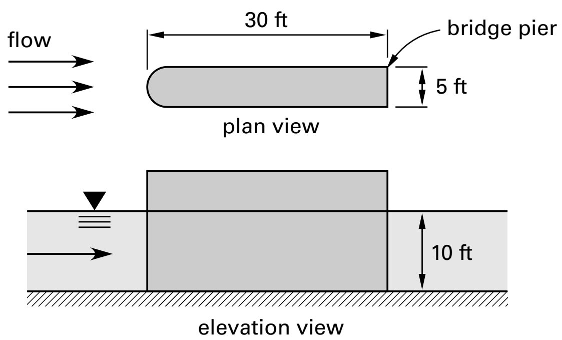
(C) I, II, and IV

(D) II, III, and IV

Hint: Refer to SDPWS Sec. 4.3.

PROBLEM 25

The plan and elevation views of a reinforced concrete bridge pier is shown.



If the water velocity of the stream's design flood is 10 ft/sec, the unfactored longitudinal drag force acting on the upstream edge of the bridge pier is most nearly

(A) 0.35 kips

(B) 3.5 kips

(C) 7.0 kips

(D) 21 kips

Hint: Use *AASHTO LRFD Bridge Design Specifications* Sec. 3.7.

PROBLEM 26

A wood-frame blocked shear wall is framed with 2 × 4 studs at 16 in on center. The exterior of the wall has ⅜ in oriented strand board (OSB) sheathing attached with 8d nails, and the panel edge fastener spacing is 6 in. The interior is sheathed with ½ in gypsum wallboard attached with no. 6 type S drywall screws (1¼ in long) spaced at 8 in around the edges. The nominal unit seismic shear capacity of the wall is most nearly

(A) 240 lbf/linear ft

(B) 440 lbf/linear ft

(C) 520 lbf/linear ft

(~~C~~ D) 580 lbf/linear ft

~~(D) 600 lbf/linear ft~~

Hint: Refer to *Special Design Provisions for Wind and Seismic* (SDPWS).

PROBLEM 27

A loadbearing concrete masonry wall in a large elementary school is 12 ft high by 30 ft long. The wall weighs 1350 lbf/linear ft and supports a precast concrete plank floor weighing 40 lbf/ft^2. The design earthquake spectral response acceleration parameter at short periods, S_{DS}, is 0.26, and the building is located in seismic design category B.

The anchorage of the wall to the precast concrete plank flooring must be capable of resisting a loading that is most nearly

(A) 89 lbf/linear ft

(B) 170 lbf/linear ft

(C) 340 lbf/linear ft

(D) 350 lbf/linear ft

Hint: Use ASCE/SEI7 Sec. 12.11.

Construction Administration

PROBLEM 28

Structural observation must be provided for a seismic design category D structure that is

(A) a two-story office building

(B) a four-story office building

(C) a public high school

(D) an agricultural arena with a height of 60 ft

Hint: Refer to IBC Sec. 1704.6.1.

The live load for the roof is 20 lbf/ft^2, and the rain load for the roof is 10 lbf/ft^2.

Determine the wind load using IBC Sec. 1609 and ASCE/SEI7 Chap. 26. The building meets the requirements of an enclosed simplified diaphragm building and can be designed according to the provisions of Part 2 of ASCE/SEI7 Chap. 27. According to ASCE/SEI7 Sec. 27.5.2, the building is a Class 1 building. Use the procedures in ASCE/SEI7 Table 27.5-1 to determine the MWFRS wind loads.

From ASCE/SEI7 Table 1.5-1, determine that a large, busy warehouse is risk category II. The basic wind speed, V, is given as 140 mph. Since no other information is provided about the site, assume the topographic factor K_{zt} is 1.0.

Using ASCE/SEI7 Table 27.6-2 and knowing $h = 30$ ft and $V = 140$ mph, find the net roof pressures. For a flat roof, zone 3, 4, and 5 apply. The values in Table 27.6-2 for exposure C must be multiplied by the adjustment factor 0.713, given in the notes to Table 27.6-2.

The maximum and minimum pressures are for zone 3.

$$\begin{aligned} p_{\text{max}} &= (0.713)\left(-44.4\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= -31.7\ \text{lbf/ft}^2 \\ p_{\text{min}} &= (0.713)\left(0\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= 0\ \text{lbf/ft}^2 \end{aligned}$$

Determine the critical load.

By inspection, determine that dead load alone is not the critical combination and that the roof rain load is less than the roof live load.

$$\begin{aligned} D + L_{\text{roof}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + 20\ \frac{\text{lbf}}{\text{ft}^2} \\ &= 36\ \text{lbf/ft}^2 \end{aligned}$$

$D + 0.6W$:

$$\begin{aligned} D + 0.6p_{\text{max}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + (0.6)\left(-31.7\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= -3.0\ \text{lbf/ft}^2 \\ D + 0.6p_{\text{min}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + (0.6)\left(0\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= 16\ \text{lbf/ft}^2 \end{aligned}$$

$D + 0.75(0.6)W + 0.75L_{\text{roof}}$:

$$\begin{aligned} D + 0.75(0.6)p_{\text{max}} + 0.75L_{\text{roof}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + (0.75)(0.6)\left(-31.7\ \frac{\text{lbf}}{\text{ft}^2}\right) + (0.75)\left(20\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= 17\ \text{lbf/ft}^2 \\ D + 0.75(0.6)p_{\text{min}} + 0.75L_{\text{roof}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + (0.75)(0.6)\left(0\ \frac{\text{lbf}}{\text{ft}^2}\right) + (0.75)\left(20\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= 31\ \text{lbf/ft}^2 \end{aligned}$$

$0.6D + 0.6W$:

$$\begin{aligned} 0.6D + 0.6p_{\text{max}} &= (0.6)\left(16\ \frac{\text{lbf}}{\text{ft}^2}\right) + (0.6)\left(-31.7\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= -9.4\ \text{lbf/ft}^2 \end{aligned}$$

INSERT: (0.6)

$$\begin{aligned} 0.6D + 0.6p_{\text{min}} &= 16\ \frac{\text{lbf}}{\text{ft}^2} + (0.6)\left(0\ \frac{\text{lbf}}{\text{ft}^2}\right) \\ &= 9.6\ \text{lbf/ft}^2 \end{aligned}$$

The largest uplift pressure is $-9.4\ \text{lbf/ft}^2$ and the largest downward pressure is $36\ \text{lbf/ft}^2$. The critical design load is the larger, $D + L_{\text{roof}}$, or $36\ \text{lbf/ft}^2$.

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution gives the largest uplift load from the $0.6D + 0.6W$ load combination ($-9.5\ \text{lbf/ft}^2$).

(C) This incorrect solution gives the maximum unadjusted uplift pressure ($-44\ \text{lbf/ft}^2$) and not the load on the roof.

(D) This incorrect solution uses the load combinations for strength design instead of those for allowable stress. The critical design case is $1.2D + 1.6L_{\text{roof}}$.

SOLUTION 11

The portal method assumes that exterior columns carry half the shear of interior columns and that the interior columns carry equal shear.

$$\begin{aligned} V_{\text{I}} &= 0.5V_{\text{II}} \\ V_{\text{IV}} &= 0.5V_{\text{II}} \\ V_{\text{II}} &= V_{\text{III}} \end{aligned}$$

SOLUTION 26

For shear walls sheathed with different (dissimilar) materials on opposite sides of the wall, SDPWS Sec. 4.3.3.3.2 specifies that the combined nominal unit seismic shear capacity is the greater of (a) two times the smaller nominal unit shear capacity, $v_{s,\mathrm{min}}$, or (b) the larger nominal unit shear capacity, $v_{s,\mathrm{max}}$.

SDPWS Table 4.3A gives the tabulated nominal shear capacities for seismic and wind design of a wood-frame shear wall sheathed with wood-based panels. For 3/8 in wood structural panel-sheathing (OSB) with 8d nails and a panel edge spacing of 6 in, the nominal unit seismic shear capacity, v_s, is 440 lbf/linear ft.

From SDPWS Table 4.3C, for a wood-frame shear wall sheathed with 1/2 in gypsum wallboard, attached with no. 6 type S drywall screws spaced at 8 in, and having unblocked studs spaced at 16 in on center, the nominal unit seismic shear capacity, v_s, is 120 lbf/linear ft.

The combined nominal unit seismic shear capacity is the larger of

$$\begin{aligned} v_{sc} &= 2v_{s,\mathrm{min}} \\ &= (2)\left(120\ \frac{\mathrm{lbf}}{\mathrm{linear\ ft}}\right) \\ &= 240\ \mathrm{lbf/linear\ ft} \\ v_{sc} &= v_{s,\mathrm{max}} \\ &= 520\ \mathrm{lbf/linear\ ft}\quad [\mathrm{controls}] \end{aligned}$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution uses the smaller of the two combined capacities rather than the larger.

(D) This incorrect solution sums the shear capacities for each side rather than following the provisions of SDPWS Sec. 4.3.3.3.2.

~~(D) This incorrect solution uses the nominal unit seismic shear capacity for a blocked wall and sums the shear capacities for each side.~~

(B) This conservative solution does not modify the capacity value using the footnote in Table 4.3A.

SOLUTION 27

ASCE/SEI7 Sec. 12.11 contains the requirements for structural walls and their anchorage.

ASCE/SEI7 Sec. 12.11.1 requires that structural walls and their anchorage be designed for an out-of-plane force normal to the wall equal to the greater of

$$F_p = 0.4S_{\mathrm{DS}}I_eW_{\mathrm{wall}}$$
$$F_p = 0.10W_{\mathrm{wall}}$$

The seismic importance factor, I_e, is based on the risk category. According to ASCE/SEI7 Sec. C1.5.1, a large elementary school is generally considered risk category III. ASCE/SEI7 Table 1.5-2 assigns a seismic importance factor of 1.25 to all risk category III buildings. Because the wall is supported at the top and bottom, the force the anchorage must resist is one-half the total weight of the wall. The force that the anchorage of the wall to the plank flooring must resist is the greater of

$$\begin{aligned} F_p &= 0.4S_{\mathrm{DS}}I_e\left(\frac{W_{\mathrm{wall}}}{2}\right) \\ &= (0.4)(0.26)(1.25)\left(\frac{1350\ \frac{\mathrm{lbf}}{\mathrm{linear\ ft}}}{2}\right) \\ &= 87.8\ \mathrm{lbf/linear\ ft} \\ F_p &= (0.10)\left(\frac{W_{\mathrm{wall}}}{2}\right) \\ &= (0.10)\left(\frac{1350\ \frac{\mathrm{lbf}}{\mathrm{linear\ ft}}}{2}\right) \\ &= 67.5\ \mathrm{lbf/linear\ ft} \end{aligned}$$

According to ASCE/SEI7 Sec. 12.11.2, the anchorage of structural walls must also provide a direct connection capable of resisting the greater of

$$F_p = 0.4S_{\mathrm{DS}}k_aI_eW_p$$
$$F_p \geq 0.2k_aI_eW_p$$

The amplification factor, k_a, is taken as 1.0 for rigid diaphragms (ASCE/SEI7 Sec. 12.11.2.1). The weight of the wall tributary to the anchor, W_p, is one-half the weight of the wall, or 675 lbf/linear ft.

$$\begin{aligned} F_p &= 0.4S_{\mathrm{DS}}k_aI_eW_p \\ &= (0.4)(0.26)(1.0)(1.25)\left(675\ \frac{\mathrm{lbf}}{\mathrm{linear\ ft}}\right) \\ &= 87.8\ \mathrm{lbf/linear\ ft} \\ F_p &\geq 0.2k_aI_eW_p \\ &= (0.2)(1.0)(1.25)\left(675\ \frac{\mathrm{lbf}}{\mathrm{linear\ ft}}\right) \\ &= 168.8\ \mathrm{lbf/linear\ ft}\quad [170\ \mathrm{lbf/linear\ ft\ controls}] \end{aligned}$$

The design force in the individual anchors is 168.8 lbf/linear ft (170 lbf/linear ft).

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution satisfies the minimum anchorage force requirements found in ASCE/SEI7 Sec. 12.11.1, but ignores those given in Sec. 12.11.2.

(C) This incorrect solution uses the full weight of the wall, rather than one-half the wall weight when determining the force on the anchorage.

(D) This incorrect solution adds the weight of the plank floor to the wall weight and uses the full weight of the wall in determining the force on the anchorage.

SOLUTION 28

IBC Sec. 1704.6.1 provides the structural observation requirements for seismic resistance. For seismic design category D, structural observations are required if the structure is classified a risk category III or IV or if the structure height is greater than 75 ft. From IBC Table 1604.5, a high school is a risk category III structure.

The answer is (C).

Why Other Options Are Wrong

(A) An office building is classified as a risk category II structure. Only risk category II structures assigned to seismic design category E that are greater than two stories above grade require structural observations.

(B) An office building is classified as a risk category II structure. Though this structure is more than two stories, only risk category II structures assigned to seismic design category E that are greater than two stories above grade require structural observations.

(D) An agricultural building is classified as a risk category I structure, and this structure has a height less than 75 ft.

SOLUTION 29

IBC Sec. 1704.6 contains the requirements for structural observations. The owner is required to employ a registered design professional to perform the structural observations, but the registered design professional does not have to be the engineer that designed the project.

The answer is (D).

Why Other Options Are Wrong

(A) This incorrect solution does not take into account that IBC Sec. 1704.6.2 requires structural observations when the nominal design wind speed exceeds 110 mph and the building height is greater than 75 ft.

(B) This incorrect solution does not take into account that IBC Sec. 1704.6.1 requires structural observations for structures assigned to seismic design categories D, E, or F and classified as risk category III or IV.

(C) This incorrect solution does not take into account that IBC Sec. 1704.6.1 requires structural observations for structures assigned to seismic design category E and classified as risk category I or II and is greater than two stories above grade plane.