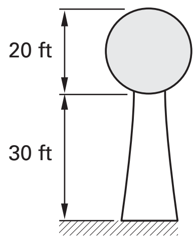
2 Lateral Forces Breadth

Analysis of Structures

ACM: 0000005029

PROBLEM 1

A 20 ft diameter, smooth, circular water tank with a projected area of 315 ft^2 provides drinking water to a seaside resort town. The velocity pressure on the tank is 25 lbf/ft^2, and the gust factor is 0.85.



Using ASCE/SEI7, the design wind force on the tank is most nearly

Change answer options to:
(A) 3300 lbf
(B) 3500 lbf
(C) 4900 lbf
(D) 5000 lbf

Hint: Use ASCE/SEI7 Chap. 29.

PROBLEM 2

A retail shopping center located in Utah is built with concrete masonry bearing walls and ordinary reinforced concrete masonry shear walls and is founded on soil. The walls of the shopping center are 15 ft tall. The maximum considered earthquake ground motion for 0.2 sec spectral response acceleration is 30% g. Using the simplified analysis procedure for seismic design found in ASCE/SEI7, the seismic base shear is

(A) $0.07W$

(B) $0.14W$

(C) $0.15W$

(D) $14W$

Hint: Refer to ASCE/SEI7 Sec. 12.14 for simplified seismic design.

PROBLEM 3

Which of the following seismic design statements are true?

I. The seismic base shear of a building increases as the building dead load increases.

II. In areas of high seismic activity, it is best to design buildings with an irregular floor plan to better break up the seismic load.

III. According to ASCE/SEI7, flat roof snow loads under 30 lbf/ft^2 need not be included in the effective seismic weight of the structure, W.

IV. Buildings with a soft first story and heavy roofs performed well during the Northridge earthquake in 1994.

(A) I and II only

(B) I and III only

(C) II and IV

(D) I, II, and III

Hint: Chapter 12 of ASCE/SEI7 provides information on seismic design.

PROBLEM 4

A two-story wood-framed apartment building is classified as seismic design category C according to the *International Building Code* (IBC). Which of the following statements about this building is FALSE?

(A) The total design lateral seismic force increases as the building weight increases.

(B) The short-period response accelerations for this site must be between 0.33 g and 0.50 g.

(C) There is no limit on story drift.

(D) When soil properties are not known in sufficient detail to determine the site class, site class D should be used unless determined otherwise by the building official or unless geotechnical data indicates that site class E or F soil is likely to be present.

Hint: Refer to IBC Sec. 1613.

ACM: 0000005029

PROBLEM 29

Which statement is INCORRECT regarding structural observations as required by the IBC?

(A) Structural observations must be provided when the building height is greater than 75 ft and the nominal design wind speed exceeds 110 mph.

(B) Structural observations must be provided for structures assigned to seismic design category D and classified as risk category III.

(C) Structural observations must be provided for structures classified as risk category I or II if the structure is assigned to seismic design category E and is greater than two stories above grade plane.

(D) Structural observations must be performed by the registered design professional responsible for the structural design of the project.

Hint: See IBC Sec. 1704.6.

(B) This incorrect solution does not apply the minimum loading criterion.

SOLUTION 1

ASCE/SEI7 Chap. 29 covers wind loads on other structures such as water tanks. The design wind force for other structures is given in ASCE/SEI7 Sec. 29.5 as

$$F = q_z G C_f A_f \quad \text{[ASCE/SEI7 Eq. 29.5-1]}$$

The velocity pressure, q_z, is given as 25 lbf/ft^2. The gust effect factor, G, is given as 0.85. The projected area, A_f, is given as 315 ft^2. Find the force coefficient, C_f.

The force coefficient for tanks is found in ASCE/SEI7 Fig. 29.5-1 and is based on the ratio of the structure's height-to-diameter ratio and the velocity pressure. In this case,

$$h = 30 \text{ ft} + 20 \text{ ft} = 50 \text{ ft}$$

$$D = 20 \text{ ft}$$

$$\frac{h}{D} = \frac{50 \text{ ft}}{20 \text{ ft}} = 2.5$$

$$D\sqrt{q_z} = (20 \text{ ft})\sqrt{25 \frac{\text{lbf}}{\text{ft}^2}} = 100$$

For a moderately smooth tank, interpolate ASCE/SEI7 Fig. 29.5-1 to determine the force coefficient.

$$C_f = 0.53$$

The design wind force for the tank is

REPLACE this text with the text on the following page.
Change the correct answer to D.

Why Other Options Are Wrong

~~(A) This incorrect solution is the velocity pressure with incorrect units.~~

(~~B~~) This incorrect solution does not interpolate ASCE/SEI7 Fig. 29.5-1 correctly and uses a value of 0.5 for the force coefficient.

(~~D~~) This incorrect solution uses a height of 30 ft when calculating the h/D ratio and the resulting force coefficient.

$$F = q_z G C_f A_f = \left(25\frac{\text{lbf}}{\text{ft}^2}\right)(0.85)(0.53)\left(315\,\text{ft}^2\right) \geq 16\frac{\text{lbf}}{\text{ft}^2} A_f = \left(16\frac{\text{lbf}}{\text{ft}^2}\right)\left(315\,\text{ft}^2\right)$$

$$F = 3548\,\text{lbf} \geq 5040\,\text{lbf}$$

Use 5040 lbf (5000 lbf).

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

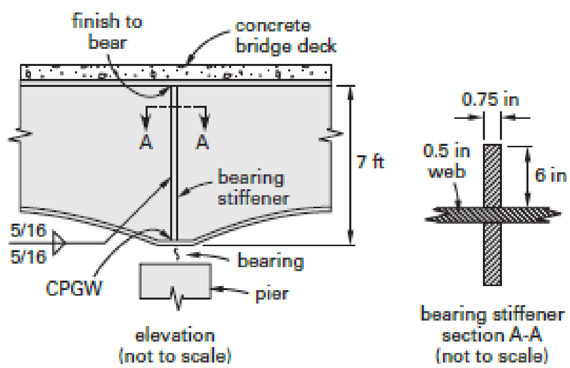
(D) II, III, and IV

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.

ACM: 0000129965

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 analysis 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*.

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 variable soil types or voids. Statements III and IV are false.

ACM: 0000129965

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.

$$b_t \le 0.48 t_{p_s} \sqrt{\frac{E}{F_{ys}}} \quad \text{[AASHTO Eq. 6.10.11.2.2-1]}$$

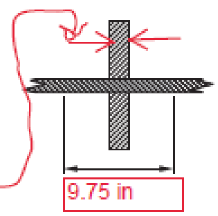
$$= (0.48)(0.75\ \text{in}) \sqrt{\frac{29{,}000\ \frac{\text{kips}}{\text{in}^2}}{50\ \frac{\text{kips}}{\text{in}^2}}}$$

$$= 8.67\ \text{in} \quad [b_t = 6\ \text{in, OK}]$$

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

$$w = 2(9t_w)$$ +t_p = (2)((9)(0.5 in))+0.75 in = 9.75 in

NOTE TO EDITORS: add dimension lines on either side of the vertical plate showing a distance of $2(9t_w)$



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.

$$A_s = 2b_t t_p + w t_{\text{web}}$$

9.75

$$= (2)(6\ \text{in})(0.75\ \text{in}) + (\cancel{9}\ \text{in})(0.5\ \text{in})$$

$$= \cancel{13.5}\ \text{in}^2$$ 13.875

Determine the moment of inertia about the web centerline.

$$I = \frac{bh^3}{12} = \frac{t_p(b_t + t_{\text{web}} + b_t)^3}{12} + \frac{(w - t_p)(t_{\text{web}})^3}{12}$$

$$= \frac{(0.75\ \text{in})(6\ \text{in} + 0.5\ \text{in} + 6\ \text{in})^3}{12}$$

9.75

$$+ \frac{(\cancel{9}\ \text{in} - 0.75\ \text{in})(0.5\ \text{in})^3}{12}$$

$$= 122\ \text{in}^4$$

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 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

$$P_e = \left(\frac{\pi^2 E}{\left(\frac{Kl}{r_s}\right)^2}\right) A_g$$

$$= \frac{\pi^2\left(29{,}000\ \frac{\text{kips}}{\text{in}^2}\right)\left(\cancel{13.5\text{ in}^2}\right)\ 13.875}{(21.2)^2}$$

$$= 8836\text{ kips}$$

The equivalent nominal yield resistance is

$$P_o = QF_yA_g = (1.0)\left(50\ \frac{\text{kips}}{\text{in}^2}\right)\left(\cancel{13.5\text{ in}^2}\right)\ 13.875$$

$$= 694\text{ kips}$$

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.

$$\frac{P_o}{P_e} = \frac{694\text{ kips}}{8836\text{ kips}} = \cancel{0.076}\ 0.079$$

The nominal compressive resistance is

$$P_n = \left(0.658^{P_o/P_e}\right)P_o = \left(0.658^{0.076}\right)(694\text{ kips})$$

$$= 671\text{ kips}$$

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

$$P_r = \phi_c P_n = (0.9)(671\text{ kips})$$

$$= 604\text{ kips}\quad (600\text{ kips})$$

The answer is (B).

The ACM for #66 is 0000145032.

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-lfb/ft

(B) 3900 ft-lbf/ft

(C) 4300 ft-lbf/ft

(D) 7800 ft-lbf/ft

Hint: See TMS 402 Sec. 9.3.5.

SOLUTION 66

The depth to reinforcement, d, is one-half the thickness of the wall, and a 12 in concrete masonry unit has a specified thickness of 11.63 in, so

$$d = \frac{t}{2} = \frac{11.63 \text{ in}}{2} = 5.81 \text{ in}$$

A no. 5 reinforcing bar has an area of 0.31 in^2.

TMS 402 Sec. 9.3.5 covers wall design for out-of-plane loads. According to TMS 402 Commentary Sec. 9.3.5.2, the nominal moment capacity of the wall is

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

In this equation, a is

$$a = \frac{A_s f_y + \dfrac{P_u}{\phi}}{0.80 f'_m b}$$

The wall is non-loadbearing, and only the out-of-plane capacity is being considered, so the factored axial load,

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.

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

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

The nominal moment capacity of the wall is

change to 0.31 in^2 / 24 in

insert:
(12 in/ft)

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

$$= \left(\frac{0\ \text{lbf}}{0.90} + (0.31\ \text{in}^2)\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right)\right)$$

$$\times \left(\frac{5.81\ \text{in} - \frac{0.4844\ \text{in}}{2}}{12\ \frac{\text{in}}{\text{ft}}}\right)$$

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

The moment capacity of the wall is

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

$$= \boxed{3884}\ \text{ft-lbf} \quad \left(\boxed{3900}\ \text{ft-lbf}\right)$$

The answer is (B).

Why Other Options Are Wrong

(A) This incorrect solution uses a yield strength, f_y, of 32,000 lbf/in^2 instead of 60,000 lbf/in^2.

(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.