Vertical Forces Component: Breadth Module Exam

1. The elevation of a building where the roof must be designed for drifting snow loads is shown.

2. A simply supported, single-span, concrete slab bridge is shown. The main flexural reinforcement is parallel to traffic.

Design Code
- ASCE/SEI7

Design Criteria
- ground snow load, $p_g = 60 \text{lbf/ft}^2$
- do not include the balanced snow load as part of the drift surcharge load

The maximum intensity of the drift surcharge load, $p_d$, is most nearly

(A) 43 lbf/ft$^2$
(B) 67 lbf/ft$^2$
(C) 94 lbf/ft$^2$
(D) 130 lbf/ft$^2$

3. A single-span, concrete bridge is shown. The abutments are rigid, and displacements at the ends of the slab are restrained in the longitudinal direction by the abutments.

Design Code
- AASHTO

Design Criteria
- concrete density, $\gamma = 0.145 \text{kips/ft}^3$
- concrete strength, $f'_c = 3 \text{kips/in}^2$
87. (e) Sketch an elevation of the end moment frame joint at gridline A-1. Show all required bars, but do not design.

II. sections A-A, B-B, and C-C

Note: Transverse reinforcement not shown for clarity.

III. column interaction diagram

24 in × 24 in column with 12-No. 8 longitudinal bars

88. Illustration I shows a typical floor plan of a four-story, steel office building with special concentrically braced frames. An elevation of the inverted V (chevron) braced frame on gridline 1, with horizontal seismic forces for each floor level, is shown in illustration II.

Design Codes
- ACI 318
- AISC 341
- AISC Steel Construction Manual
- AISC 360
- AISC Seismic Construction Manual
- ASCE/SEI7

Steel
- columns and beams, ASTM A992 grade 50; \( F_y = 50 \text{kips/in}^2; F_u = 65 \text{kips/in}^2 \)
- HSS braces, ASTM A500 grade B; \( F_y = 46 \text{kips/in}^2; F_u = 58 \text{kips/in}^2 \)
- welding electrodes, \( F_{EEX} = 70 \text{kips/in}^2 \)

Concrete Footing
- concrete strength, \( f'_c = 4000 \text{lbf/in}^2 \) (normal weight)
- reinforcing strength, \( f_y = 60,000 \text{lbf/in}^2 \)
Solutions
Vertical Forces Component: Breadth Module
Exam

1. From the problem illustration and ASCE/SEI Fig. 7-8, \( l_u \) is 50 ft for leeward drift and 200 ft for windward drift. The clear height, \( h_c \), is 10 ft.

From ASCE/SEI Sec. 7.7.1, the drift height, \( h_d \), is calculated using ASCE/SEI Fig. 7-9. The controlling drift height, \( h_{cd} \), is the larger of the value from ASCE/SEI Fig. 7-9 considering \( l_u \) for leeward drift or \( \frac{3}{4} \) of the value from Fig. 7-9 considering \( l_u \) for windward drift. (These equations are not dimensionally consistent.)

The snow density, \( \gamma \), is found using ASCE/SEI Eq. 7.7-1 and cannot be greater than 30 lbf/ft\(^3\). (This equation is not dimensionally consistent.)

\[
\gamma = 0.13 p_g + 14 = (0.13)\left(\frac{60 \text{ lbf}}{\text{ft}^2} + 10 \text{ ft} - 1.5\right)
\]
\[
= 21.8 \text{ lbf/ft}^3 < 30 \text{ lbf/ft}^3 \ [\text{OK}]
\]

From ASCE/SEI Sec. 7.7.1, the maximum intensity for the drift surcharge load, \( p_d \), is

\[
p_d = h_d \gamma = (4.33 \text{ ft})\left(21.8 \frac{\text{lbf}}{\text{ft}^3}\right) = 94 \text{ lbf/ft}^2
\]

The answer is (C).

2. From AASHTO Sec. 3.6.1.2.1, the HL-93 live load consists of a design truck or tandem and a design lane load.

The design lane load, \( w \), is given in AASHTO Sec. 3.6.1.2.4 as 0.64 kips/ft. The maximum moment for the design lane load, \( M_{lane} \), is

\[
M_{lane} = \frac{w l^2}{8} = \frac{(0.64 \text{ kips/ft})(30 \text{ ft})^2}{8} = 72 \text{ ft-kips}
\]

For single spans of less than 40 ft, the design tandem loading of AASHTO Sec. 3.6.1.2.3 will produce larger moment demands than the design truck loading. Therefore, use two 25 kip axle loads, \( P \), with a longitudinal spacing, \( a \), of 4 ft.

The moment due to the design tandem can be found from AISC Steel Construction Manual Table 3-23, case 44.

Determine whether \( a \) is greater than or less than 0.568.

\[
0.568 = (0.568)(30 \text{ ft}) = 17 \text{ ft} > 4 \text{ ft}
\]

So, for \( a < 0.568 \), the maximum moment for the design tandem, \( M_{tandem} \), is

\[
M_{tandem} = \frac{P l}{2}\left(l - \frac{a}{2}\right)^2 = \frac{(25 \text{ kips})(30 \text{ ft} - 4 \text{ ft})^2}{2} = 327 \text{ ft-kips}
\]

From AASHTO Table 3.4.1-1, live loads must be factored by 1.75 for the strength I limit state.

From AASHTO Sec. 3.6.2.1, the design tandem moment (but not the lane load) must be increased by the dynamic load allowance, \( IM \), specified in AASHTO Table 3.6.2.1-1. For the design tandem moment, which falls under “all other limit states,” \( IM \) is 33%.

Although the maximum moments due to the lane load and tandem load do not occur in the same location, it is sufficiently accurate to add the lane and tandem

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The requirements of ACI 318 Sec. 21.6.2.2 are met.

87. (d) From ACI 318 Sec. 18.8.2.2, beam longitudinal reinforcement that ends in a column must be extended to the far face of the confined column core and anchored in tension, according to ACI 318 Sec. 18.8.5.

Per ACI 318 App. A, the nominal diameter, \( d_b \), of a no. 9 bar is 1.128 in. From ACI 318 Sec. 18.8.5.1, the development length for a hooked bar in tension, \( l_{dh} \), is the largest of 6 in, \( 8d_b \), and ACI 318 Eq. 18.8.5.1.

\[
l_{dh} = \max\left(6 \text{ in}, \frac{f_y d_b}{60000 \frac{\text{lbf}}{\text{in}^2}}, \frac{\sqrt{5000 \frac{\text{lbf}}{\text{in}^2}}}{(65)}, 14.7 \text{ in} \right)
\]

Considering any reasonable column clear cover, this length is less than the dimension of the confined column core of a 24 in column. Therefore, the no. 9 beam longitudinal reinforcement can be fully developed in tension with a standard 90° hook at an end moment frame joint.

87. (e) A sketch of an elevation of the end moment frame joint at gridline A-1 is shown.

88. (a) Use ASCE/SEI7 Sec. 12.4.2.3, load combination 5.

\[
(1.2 + 0.2S_{DS})D + \rho Q_e + 0.5L + 0.2S
\]

In accordance with ASCE/SEI7 Sec. 12.4.2.3, note 1, the load factor on \( L \) is taken as 0.5 for live loads that do not exceed 100 lbf/ft².

With a redundancy factor of \( \rho = 1.0 \), the factored load at footing B-1 is

\[
P_s = (1.2 + 0.2S_{DS})P_D + P_{Q_e} + 0.5P_L + 0.2P_S
\]

\( P_D, P_{Q_e}, P_L, \) and \( P_S \) are the footing loads due to dead, horizontal seismic, live, and snow loads, respectively.

From problem illustration I, the column at gridline B-1 has a tributary area at each floor of

\[
A = (16 \text{ ft})(16 \text{ ft}) = 256 \text{ ft}^2
\]

The unfactored load at the footing due to the dead load, \( P_{Dn} \), is

\[
P_{Dn} = (\text{floors affected})Aq_{\text{dead}} = \frac{(4)(256 \text{ ft}^2)(150 \frac{\text{lbf}}{\text{ft}^2})}{1000 \text{ lbf/kip}} = 153.6 \text{ kips}
\]

The unfactored load at the footing due to the snow load, \( P_{Sn} \), is

\[
P_{Sn} = (\text{floors affected})Aq_{\text{snow}} = \frac{(1)(256 \text{ ft}^2)(30 \frac{\text{lbf}}{\text{ft}^2})}{1000 \text{ lbf/kip}} = 7.7 \text{ kips}
\]

The tributary area, \( A_T \), for live loads is

\[
A_T = (\text{floors affected})A = (3)(256 \text{ ft}^2) = 768 \text{ ft}^2
\]

From ASCE/SEI7 Table 4-2, the live load element factor, \( K_{LL} \), for an exterior column without cantilever slabs is 4.

\[
K_{LL}A_T = (4)(768 \text{ ft}^2) = 3072 \text{ ft}^2 \geq 400 \text{ ft}^2
\]

From ASCE/SEI7 Sec. 4.7.2, members for which a value of \( K_{LL}A_T \) is 400 ft² or greater may have live loads reduced using ASCE/SEI7 Eq. 4.7-1.