APPENDIX C

Sample Design Calculations

This appendix presents design examples of the retrofitting techniques for elevation, dry floodproofing, wet floodproofing, and construction of a floodwall in a residential setting. Examples C1 through C5 are a set of examples that illustrate the elevation of a single-story home with a crawlspace. Example C6 demonstrates how to size a sump pump for dry floodproofing. Example C7 shows how to calculate both the net buoyancy force exerted on a liquid propane tank in a wet floodproofing scenario and the volume of concrete needed to offset the buoyancy force. The final example, Example C8, demonstrates how to design a cantilevered floodwall to protect a residence subject to 3 feet of flooding. Please note that Examples C6 through C8 do not use the same example home presented in Examples C1 through C5, but are instead standalone examples.

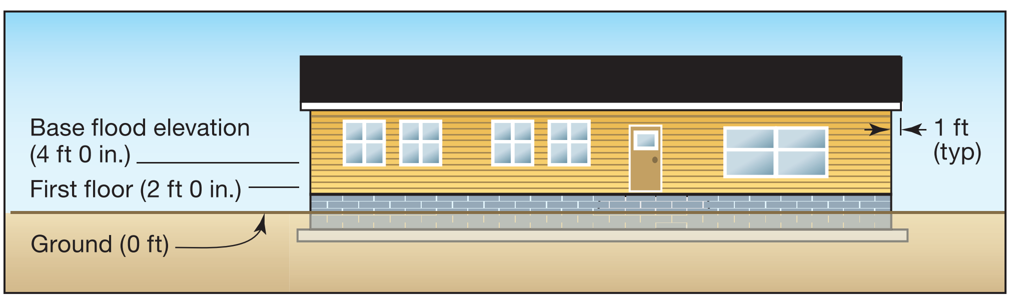
The analyses and design solutions presented in this appendix apply to the example problems only. A licensed design professional should be engaged for actual projects involving the residential flood retrofitting techniques presented herein.

Elevation

Given or Obtained from the Field Investigation:

The owner of a single-story crawlspace home intends to elevate the structure to eliminate a repetitive flooding hazard. Her desire is to raise the structure one full story (8 feet) and use the lower level for parking and storage. She contracted with a local engineer to perform the design. The engineer's investigation revealed the following information about the existing structure:

- crawlspace home has 8-inch CMU ungrouted foundation (no vertical reinforcement);
- the first-floor elevation is 2 feet above the surrounding grade (which is level);
- the property is located in a FEMA-designated floodplain (Zone AE) and the base flood elevation (BFE) is 4 feet above ground level;
- floodwater velocities during a base flood will average 6 feet per second;
- floodwater flows parallel to the side elevation and impact the front elevation;
- floodwater debris hazard exists and is characterized as normal;
- the structure is classified as a pre-Flood Insurance Rate Map (pre-FIRM) structure with no existing flood openings; and
- local regulations require an additional 1 foot of freeboard above the 100-year flood elevation.

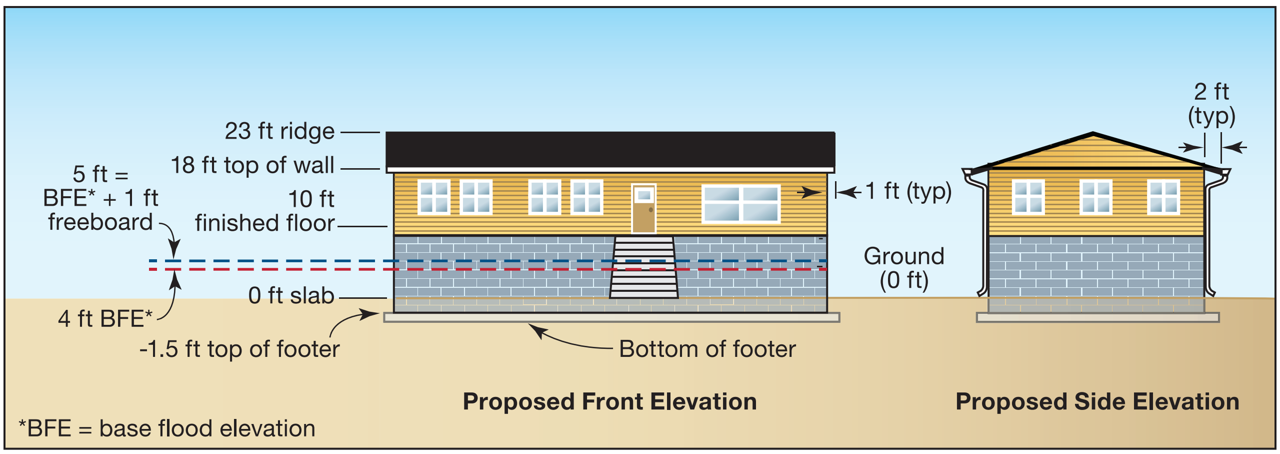


Additional Information on Existing House:

- wood-frame house 30 feet x 60 feet; and
- gable roof with 4:12 slope.

Per International Residential Code (IRC) and American Society of Civil Engineers (ASCE) 7:

- 90 miles per hour Basic Wind Speed (3-second gust);
- flat open terrain surrounding house;
- wind Exposure Category C, enclosed building;
- seismic importance factor I_e = 1.0:, S_{DS} = 0.2; and
- ground snow load of 20 lb/ft^2.



Extended foundation walls are proposed to be constructed of 8-inch-thick concrete masonry units. The existing footing is 2 feet wide by 1 foot thick concrete reinforced with (3) #4 rebar continuously. Top of footing is 18 inches below grade; soil type is stiff residual silts. New slab-on-grade will be 3½ to 4 inches thick and set at grade.

Interior walls of the living area (elevated) are composed of 2x4 studs at 16 inches on center (o.c.) with plaster on each side. Exterior walls have 2x4 studs at 16 inches o.c., plaster on the inside, and sheathing and wood siding on the exterior walls are insulated with fiberglass insulation.

First floor framing consists of 2x12s at 16 inches o.c. supported by the exterior long walls and a center support. Floor coverings are hardwood (oak) with a ¾-inch plywood subfloor. There are 10 inches of insulation between the joists. A gypsum ceiling is planned for the proposed lower area.

Roof framing consists of pre-engineered wood trusses at 16 inches o.c. The top chord consists of 2x6s and the web and bottom chord consist of 2x4s. The roof is fiberglass shingles with felt on ½-inch plywood. The ceiling is ½-inch plaster with ½-inch plywood backup. There are 16 inches of fiberglass insulation above the ceiling.

EXAMPLE C1. CALCULATE VERTICAL LOADS

Given:

Per original Sample Calculation Statement:

- Property is located in a FEMA-designated floodplain (Zone AE) and is subject to a 100-year flood 4 ft in depth above ground level;
- Local regulations require an additional 1 ft of freeboard above the 100-year flood elevation; and
- Field investigation information and additional information on the site

Find:

1. Design flood elevation (DFE)
2. Floodproofing design depth
3. Total vertical flood load due to buoyancy
4. Vertical loads on the house (excluding buoyancy force)
5. Total structure dead weight
6. Is the total structural dead weight sufficient to prevent flotation of the house from the buoyancy force during a flood event?

Solution for #1: To find the DFE, use Equation 4-2:

$DFE = FE + f = 4\,\text{ft} + 1\,\text{ft} = \mathbf{5\ ft}$

NOTE: The DFE calculated includes freeboard.

Solution for #2: To find the floodproofing design depth over which flood forces will be considered, use Equation 4-3:

$H = DFE - GS = 5\,\text{ft} - 0\,\text{ft} = 5\ \text{ft}$

Solution for #3: The buoyancy force can be found as follows:

- The calculation of buoyancy forces and comparison with structure weight is a critical determination of this problem. While buoyancy of the first floor is not an issue (since it is elevated 5 ft above the DFE), buoyancy of the entire structure (slab, foundation walls, and superstructure) must be checked if dry floodproofing is being considered for the lower level. If buoyancy forces control, or if the slab and walls cannot withstand the hydrostatic and other design flood forces, dry floodproofing of the lower level is not applicable. Using Equation 4-6:

$F_{bouy} = \gamma_w AH = (62.4\,\text{lb/ft}^3)(30\,\text{ft})(60\,\text{ft})(5\,\text{ft}) = \mathbf{561.6\ kips}$

EXAMPLE C1. CALCULATE VERTICAL LOADS (continued)

Solution for #4: The vertical loads can be determined as follows:

Calculate Structure Weight by Level

- Tabulate Dead Loads by Floor (based on ASCE 7-10, Table C3-1)

Roof: 2x6 Top Chord and 2x4 Web and Bottom
Shingles – Asphalt – 1 layer: 2.0 lb/ft^2
Felt: 0.7 lb/ft^2
Plywood – 32/16–1/2 in.: 1.5 lb/ft^2
Trusses @ 16 in. o.c.: 5.0 lb/ft^2
$Total = 2.0\,\text{lb/ ft}^2 + 0.7\,\text{lb/ ft}^2 + 1.5\,\text{lb/ ft}^2 + 5.0\,\text{lb/ ft}^2 = 9.2\,\text{lb/ ft}^2$

First Floor Ceiling:
Insulation – 16 in. of fiberglass: 8.0 lb/ft^2
1/2 in. plywood: 1.5 lb/ft^2
1/2 in. plaster and lath: 10.0 lb/ft^2
Misc., heating, electrical, cabinets: 2.0 lb/ft^2
$Total = 8.0\,\text{lb/ ft}^2 + 1.5\,\text{lb/ ft}^2 + 10.0\,\text{lb/ ft}^2 + 2.0\,\text{lb/ ft}^2 = 21.5\,\text{lb/ ft}^2$

First Floor:
Oak Floor: 4.0 lb/ft^2
Subfloor – ¾ in. plywood: 3.0 lb/ft^2
Joists (2x12): 4.0 lb/ft^2
Insulation – 10 in. fiberglass: 5.0 lb/ft^2
Misc., piping, electrical: 3.0 lb/ft^2
Gypsum ceiling – 1/2 in.: 2.5 lb/ft^2
$Total = 4.0\,\text{lb/ ft}^2 + 3.0\,\text{lb/ ft}^2 + 4.0\,\text{lb/ ft}^2 + 5.0\,\text{lb/ ft}^2 + 3.0\,\text{lb/ ft}^2 + 2.5\,\text{lb/ ft}^2 = 21.5\,\text{lb/ ft}^2$

Walls:
Interior – wood stud, plaster each side: 20 lb/ft^2
Exterior – 2x4 @ 16 in. o.c., plaster insulation, wood siding: 18 lb/ft^2
Lower Level – 8 in. masonry, reinforcement at 48 in. o.c.: 46 lb/ft^2

- Now to determine the total weights by level

Roof: Using the roof overhang of 2 ft
Surface Area = $(15.81\,\text{ft} + 2\,\text{ft})(60\,\text{ft} + 2\,\text{ft})(2) = 2{,}208\,\text{ft}^2$

Projected Area = $\left[15\,\text{ft} + 2\,\text{ft}\left(\dfrac{15}{15.81}\right)\right](60\,\text{ft} + 2\,\text{ft})(2) = 2{,}095\,\text{ft}^2$

Shingles: $2{,}208\,\text{ft}^2(2\,\text{lb/ ft}^2) = 4{,}416\,\text{lbs}$
Felt: $2{,}208\,\text{ft}^2(0.7\,\text{lb/ ft}^2) = 1{,}546\,\text{lbs}$
Plywood: $2{,}208\,\text{ft}^2(1.5\,\text{lb/ ft}^2) = 3{,}312\,\text{lbs}$
Truss: 2,095 ft^2 (5lb/ft^2) = 10,475 lbs

EXAMPLE C1. CALCULATE VERTICAL LOADS (continued)

Gable end walls: $150\,\text{ft}^2(18\,\text{lb/ ft}^2) = 2{,}700\,\text{lbs}$
$\textit{Total} = 4{,}416\,\text{lbs} + 1{,}546\,\text{lbs} + 3{,}312\,\text{lbs} + 10{,}475\,\text{lbs} + 2{,}700\,\text{lbs} =$ **22,449 lbs** for roof weight

First Floor Ceiling:
Area = $(60\,\text{ft})(30\,\text{ft}) = 1{,}800\,\text{ft}^2$
Insulation: $(1{,}800\,\text{ft}^2)(8\,\text{lb/ ft}^2) = 14{,}400\,\text{lbs}$
Plywood: $(1{,}800\,\text{ft}^2)(1.5\,\text{lb/ ft}^2) = 2{,}700\,\text{lbs}$
Plaster: $(1{,}800\,\text{ft}^2)(10\,\text{lb/ ft}^2) = 18{,}000\,\text{lbs}$
Miscellaneous: $(1{,}800\,\text{ft}^2)(2\,\text{lb/ ft}^2) = 3{,}600\,\text{lbs}$
$\textit{Total} = 14{,}400\,\text{lbs} + 2{,}700\,\text{lbs} + 18{,}000\,\text{lbs} + 3{,}600\,\text{lbs} =$ **38,700 lbs** for the first floor ceiling

Walls:
Exterior: $(180\,\text{ft})(4\,\text{ft})(18\,\text{lb/ ft}^2) = 12{,}960\,\text{lbs}$
Interior: $(157\,\text{ft})(4\,\text{ft})(20\,\text{lb/ ft}^2) = 12{,}560\,\text{lbs}$
$\textit{Total} = 12{,}960\,\text{lbs} + 12{,}560\,\text{lbs} =$ **25,520 lbs** for the walls

Calculating the subtotal for the roof, first floor ceiling, and walls

$W_1 = 22{,}449\,\text{lbs} + 38{,}700\,\text{lbs} + 25{,}520\,\text{lbs} = 86{,}669\,\text{lbs}$

First Floor Including Lower Level: Each of the components has the following area:
Area = $(60\,\text{ft})(30\,\text{ft}) = 1{,}800\,\text{ft}^2$

Floors:
Oak Floor: $(1{,}800\,\text{ft}^2)(4\,\text{lb/ ft}^2) = 7{,}200\,\text{lbs}$
Subfloor: $(1{,}800\,\text{ft}^2)(3\,\text{lb/ ft}^2) = 5{,}400\,\text{lbs}$
Joists: $(1{,}800\,\text{ft}^2)(4\,\text{lb/ ft}^2) = 7{,}200\,\text{lbs}$
Insulation: $(1{,}800\,\text{ft}^2)(5\,\text{lb/ ft}^2) = 9{,}000\,\text{lbs}$
Miscellaneous: $(1{,}800\,\text{ft}^2)(3\,\text{lb/ ft}^2) = 5{,}400\,\text{lbs}$
$\textit{Total} = 7{,}200\,\text{lbs} + 5{,}400\,\text{lbs} + 7{,}200\,\text{lbs} + 9{,}000\,\text{lbs} + 5{,}400\,\text{lbs} =$ **34,200 lbs** for the floors

Ceiling:
$(1{,}800\,\text{ft}^2)(2.5\,\text{lb/ ft}^2) =$ **4,500 lbs** for the ceiling

Walls:
Exterior: $(180\,\text{ft})(4\,\text{ft})(18\,\text{lb/ ft}^2) = 12{,}960\,\text{lbs}$
Interior: $(157\,\text{ft})(4\,\text{ft})(20\,\text{lb/ ft}^2) = 12{,}560\,\text{lbs}$
Lower level above proposed dry floodproofed slab: $(180\,\text{ft})(9\,\text{ft})(46\,\text{lb/ ft}^2) = 74{,}520\,\text{lbs}$
Weight of water per square foot of masonry wall: $(\frac{8}{12}\,\text{ft})(62.4\,\text{lb/ ft}^3) = 41.6\,\text{lb/ ft}^2$
Lower level below proposed dry floodproofed slab: $(180\,\text{ft})(1.5\,\text{ft})(46 - 41.6\,\text{lb/ ft}^2) = 1{,}188\,\text{lbs}$

EXAMPLE C1. CALCULATE VERTICAL LOADS (concluded)

$Total = 12{,}960\,\text{lbs} + 12{,}560\,\text{lbs} + 74{,}520\,\text{lbs} + 1{,}188\,\text{lbs} =$ **101,228 lbs** for the walls

Footing:
$(180\,\text{ft})(2\,\text{ft})(1\,\text{ft})(150 - 62.4\,\text{lb/ft}^2) =$ **31,536 lbs** for footing

Slab:
$(1{,}800\,\text{ft}^2)(0.33\,\text{ft})(150\,\text{lb/ft}^3) =$ **89,100 lbs** for the slab

Calculating the subtotal for the floor, ceiling, walls, footing, and slab:

$W_2 = 34{,}200\,\text{lbs} + 4{,}500\,\text{lbs} + 101{,}228\,\text{lbs} + 31{,}536\,\text{lb}\,s + 89{,}100\,\text{lbs} = 260{,}564\,\text{lbs}$

Solution for #5: The total dead load of the structure can be found by adding the two above subtotals:
$W = W_1 + W_2 = 86{,}669\,\text{lbs} + 260{,}564\,\text{lbs} =$ **347,233 lbs**

Solution for #6: To determine if the dead load (structure weight) from the house is sufficient to prevent overturning from the buoyancy force, compare the buoyancy force to the structure weight:
$W \geq F_b$
347,233 lbs ≤ 561,600 lbs
347 kips ≤ 562 kips N.G. (No Good)

Therefore, the structural weight is not enough to prevent floatation of house during design flooding events. Additionally, in the case that the dead weight of the elevated structure could resist the buoyancy forces, the new slab would have to be designed to transfer buoyancy loads to exterior walls without cracking.

NOTE: Buoyancy forces control the building (if dry floodproofed) during a flooding event unless structural measures, such as floor anchors or additional slab mass, or non-structural measures such as allowing the lower level to flood, are utilized to offset/equalize the buoyancy forces.

In our example, since buoyancy controls and the magnitude of the project represent a substantial improvement, the homeowner is required to allow the lower level to flood by incorporating vent openings in the foundation wall. While this action will equalize hydrostatic pressures on the foundation walls, hydrodynamic and flood-borne debris impact forces will still apply.

Section 5E.1.2.1 of FEMA Technical Bulletin 1-08 provides the following guidance on flood openings: "A minimum of two openings shall be provided on different sides of each enclosed area, having a total net area of not less than 1 square inch for every square foot of enclosed area subject to flooding." The enclosed area of the example building is 1,800 square feet (60 feet x 30 feet); therefore, a minimum of two flood openings with a minimum combined area of 1,800 square inches shall be installed 12 inches or less above grade.

EXAMPLE C2. CALCULATE LATERAL LOADS

Given:

Per original Sample Calculation Statement:

- Floodwater velocities in the area of the house average 6 ft/sec
- Floodwater flows parallel to front elevation and impact side elevation
- Floodwater debris hazard exists and is characterized as normal
- Flood openings will be installed in the foundation walls
- V = basic wind speed = 90 mph
- Exposure Category C
- I = importance factor = 1.00 for residential construction
- S_{DS} = 3.0 sec

Additional information for this example per Equation 4-13, Table 4-6 and Table 4-7:

- W = weight of debris = 1,000 lb
- C_D = depth coefficient = 0.75
- C_B = blockage coefficient = 0.6
- C_{str} = building structure coefficient = 0.8

Find:

1. Lateral hydrostatic force
2. Lateral hydrodynamic forces
3. Lateral debris impact forces
4. Lateral forces for wind perpendicular and parallel to the main ridge
5. Seismic forces

Solution for #1: To calculate lateral hydrostatic forces from 5 ft of water moving at 6 ft/sec:

First, calculate hydrostatic force from 5 ft of freestanding water using Equation 4-4.

$$f_{sta} = \frac{1}{2} P_h H = \frac{1}{2} \gamma_w H^2$$

$$f_{sta} = (0.5)\left(62.4\, {}^{\text{lb}}\!/_{\text{ft}^3}\right)(5\,\text{ft})^2$$

f_{sta} = 780 lb/ft acting at 1.67 ft above ground surface

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

In the "additional information" provided in the description of the house, the top of footing is 24 in. below ground surface and the soil type is stiff residual clay (S = 82 lb/ft³ per Table 4-3).

Calculate submerged soil and water forces per Equation 4-5.

$$f_{dif} = \tfrac{1}{2}(S - \gamma_w)D^2$$

$$f_{dif} = \tfrac{1}{2}(82\,\text{lb/ft}^3 - 62.4/\text{ft}^3)(1.5\,\text{ft})^2$$

$f_{dif} = 22.5$ lb/ft acting at 0.75 ft below ground surface

f_{dh} will be calculated in #2 to yield the force due to flowing water, using the equivalent hydrostatic method.

Finally, compute total lateral hydrostatic force using Equation 4-4, after removing the f_{dh} term (which is being used in #2).

$$F_{sta} = f_{sta} + f_{dif}$$

$$F_{sta} = 0\,\text{lb}/\text{ft} + 22\,\text{lb}/\text{ft}$$

$F_{sta} =$ **22 lb/ft**

Solution for #2: Since V < 10 ft/sec, use Eq. 4-7 to convert hydrodynamic force to equivalent hydrostatic force

$$dh = \frac{C_d V^2}{2g}$$

Determine drag coefficient C_d by calculating $\frac{b}{H}$ and using Table 4-5

$$\frac{b}{H} = \frac{30\,\text{ft}}{5\,\text{ft}} = 6$$

$$C_d = 1.25$$

$$dh = \frac{1.25\left(6\,\frac{\text{ft}}{\text{sec}}\right)^2}{2\left(32.2\,\frac{\text{ft}}{\text{sec}^2}\right)}$$

$dh =$ 0.70 ft

Convert the equivalent head to equivalent hydrostatic force.

From Equation 4-8:

$$f_{dh} = \gamma_w (dh) H = P_{dh} H$$

$$f_{dh} = \left(62.4\,\text{lb}/\text{ft}^3\right)(0.70\,\text{ft})(5\,\text{ft})$$

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

$f_{dh} =$ **218.4 lb/ft** acting at 2.5 ft above ground surface

Now to calculate total force due to flow velocity on the building face (upstream) $F_d = f_{dh}W$

$$F_d = \left(218.4 \,{}^{\text{lb}}\!/_{\text{ft}}\right)(30\,\text{ft})$$

$F_d =$ **6,552 lbs = 6.55 kips**

Solution for #3: To calculate lateral debris impact loads, use Equation 4-11:

$$F_i = WVC_D C_B C_{Str}$$

The parameters in Equation 4-11 are briefly discussed in Chapter 4 of this publication and discussed in greater detail in Chapter 8 of FEMA P-55 (Fourth Edition), *Coastal Construction Manual* (FEMA, 2011).

$$F_i = (1{,}000\,\text{lbs})(6\,\text{ft/sec})(0.75)(0.6)(0.8)$$

$F_i =$ **2,160 lbs = 2.16 kips**

NOTE: Since vents are being used to equalize the hydrostatic pressure, the wall will be subject to a net load equal to the combined hydrodynamic and impact loads. The ability of the new foundation wall to withstand these forces is presented toward the end of Example C5.

Solution for #4: To calculate lateral forces for wind perpendicular and parallel to the main ridge:

Since the house is being elevated, wind pressures will be increased on the building. Depending upon the amount of elevation, additional bracing of the roof or walls may be necessary.

Reference: International Residential Code (IRC) and ASCE 7

Basic Wind Speed has been determined to be 90 mph from the 2012 IRC and verified with the local building official. The method to determine lateral forces is the Directional Procedure per ASCE 7-10. Because ASCE 7-10 procedures and load combinations are used in this sample problem, the equivalent ASCE 7-10 wind speed must be used for the same geographical region. That wind speed is 115 mph (3-second gust).

From ASCE 7-10 Equation 27.3-1, calculate velocity pressure calculated at height z above ground (q_z):

$$q_z = 0.00256 K_z K_{zt} K_d V^2$$

Compute q_z at two different heights:

- At $z_1 = 15\,\text{ft}$
- At mean roof height $z_2 = h = 18\,\text{ft} + \dfrac{5\,\text{ft}}{2} = 20.5\,\text{ft}$

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

From ASCE 7-10 Table 27.3-1, compute velocity pressure exposure coefficients (K_z) at heights listed above:

- For $z_1 = 15\,\text{ft},\ K_{z_1} = 0.85$
- For $z_2 = h = 20.5\,\text{ft},\ K_{z_2} = K_h = 0.904$ by linear interpolation

Use topographic factor, $K_{zt} = 1.0$ since house is surrounded by flat, open terrain

From ASCE 7-10 Table 26.6-1, use directionality factor, $K_d = 0.85$ for buildings

$q_{z_1} = 0.00256(0.85)(1.0)(0.85)(115\,\text{mph})^2$

$q_{z_1} = 24.5$ lb/ft²

$q_{z_2} = 0.00256(0.904)(1.0)(0.85)(115\,\text{mph})^2$

$q_{z_2} = 26.0$ lb/ft²

Calculate Design Wind Pressures on Building Main Wind Force Resisting System, MWFRS (p) using Equation 27.4-1 (ASCE 7-10)

$$p = qGC_p - q_i\left(GC_{pi}\right)$$

From ASCE 7-10 Equation 27.3-1 use $q = q_{z_1} = 24.5\,\text{lb/ft}^2$, the velocity pressure computed for windward walls calculated at wall height z_1 or z_2 above ground (lb/ft²)

From ASCE 7-10 Equation 27.3-1, use $q = q_{z_1} = 26.0\,\text{lb/ft}^2$ for all other walls and roof surfaces (lb/ft²)

From ASCE 7-10 Section 26.9.1, use gust effect factor G = 0.85 for rigid structures

From ASCE 7-10 Figure 27.4-1, compute external pressure coefficients (C_p) for the following scenarios:

- Perpendicular to the ridge, where:

a. For windward walls, $C_p = 0.8$

b. For leeward walls, $\frac{L}{B} = \frac{30\,\text{ft}}{60\,\text{ft}} = 0.5,\ C_p = -0.5$

c. For windward roof, $\frac{h}{L} = \frac{20.5\,\text{ft}}{30\,\text{ft}} = 0.683,\ \theta = \tan^{-1}\left(\frac{4\,\text{ft}}{12\,\text{ft}}\right) = 18.4,\ C_p = -0.61$ and -0.12 by linear interpolation

d. For leeward roof, $\frac{h}{L} = \frac{20.5\,\text{ft}}{30\,\text{ft}} = 0.683,\ \theta = \tan^{-1}\left(\frac{4\,\text{ft}}{12\,\text{ft}}\right) = 18.4,\ C_p = -0.58$ by linear interpolation

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

- Parallel to the ridge, where:

a. For windward walls, $C_p = 0.8$

b. For leeward walls, $\dfrac{L}{B} = \dfrac{60\,\text{ft}}{30\,\text{ft}} = 2, C_p = -0.3$

c. For windward roof, $\dfrac{h}{L} = \dfrac{20.5\,\text{ft}}{60\,\text{ft}} = 0.34, \theta = \tan^{-1}\left(\dfrac{4\,\text{ft}}{12\,\text{ft}}\right) = 18.4, C_p = -0.9$ for 0 to 20 ft from windward edge, $C_p = -0.5$ for 20 to 40 ft from windward edge

From Equation 27.3-1, use velocity pressure calculated at mean roof height, $q_h = q_i = 26.0\ \text{lb/ft}^2$

From Table 26.11-1, use internal pressure coefficients for enclosed buildings, $GC_{pi} = \pm\ 0.18$

MWFRS – Wind Perpendicular to Ridge

Walls:

Windward:

$p = (24.5\,\text{lb/ft}^2)(0.85)(0.8) - (26.0\,\text{lb/ft}^2)(0.18)$

$p =$ **12.0 lb/ft²** (inward)

$p = (24.5\,\text{lb/ft}^2)(0.85)(0.8) - (26\,\text{lb/ft}^2)(-0.18)$

$p =$ **21.3 lb/ft²** (inward)

Leeward:

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.5) - (26.0\,\text{lb/ft}^2)(0.18)$

$p =$ **-15.7 lb/ft²** (outward)

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.5) - (26.0\,\text{lb/ft}^2)(-0.18)$

$p =$ **-6.4 lb/ft²** (outward)

Roof:

Windward:

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.61) - (26.0\,\text{lb/ft}^2)(0.18)$

$p =$ **-18.2 lb/ft²** (outward)

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.12) - (26.0\,\text{lb/ft}^2)(-0.18)$

$p =$ **2.0 lb/ft²** (inward)

Leeward:

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.58) - (26.0\,\text{lb/ft}^2)(0.18)$

$p =$ **-17.4 lb/ft²** (outward)

$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.58) - (26.0\,\text{lb/ft}^2)(-0.18)$

$p =$ **-8.1 lb/ft²** (outward)

MWFRS – Wind Parallel to Ridge

Walls:

Windward:

$p = (24.5\,\text{lb/ft}^2)(0.85)(0.8) - (26.0\,\text{lb/ft}^2)(0.18)$

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

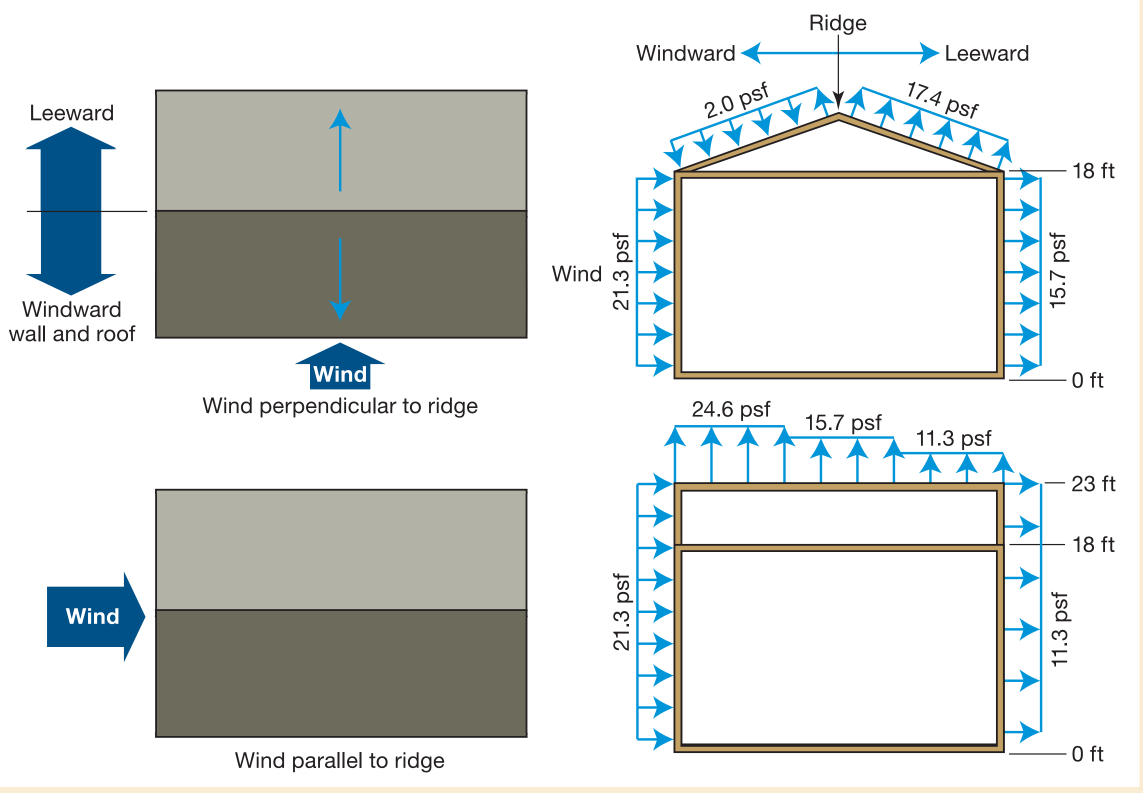
p = **12.0 lb/ft²** (inward)
$p = (24.5\,\text{lb/ft}^2)(0.85)(0.8) - (26.0\,\text{lb/ft}^2)(-0.18)$
p = **21.3 lb/ft²** (inward)
Leeward:
$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.3) - (26.0\,\text{lb/ft}^2)(0.18)$
p = **-11.3 lb/ft²** (outward)
$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.3) - (26.0\,\text{lb/ft}^2)(-0.18)$
p = **-2.0 lb/ft²** (outward)

Roof:
Windward, for distance from leading edge:

0 to 20 ft $p = (26.0\,\text{lb/ft}^2)(0.85)(-0.90) - (26.0\,\text{lb/ft}^2)(0.18)$
p = **-24.6 lb/ft²** (outward)
$p = (26.0\,\text{lb/ft}^2)(0.85)(-0.90) - (26.0\,\text{lb/ft}^2)(-0.18)$
p = **-15.2 lb/ft²** (outward)

20 to 40 ft $p = (26.0\,\text{lb/ft}^2)(0.85)(-0.5) - (26.0\,\text{lb/ft}^2)(0.18)$
p = **-15.7 lb/ft²** (outward)

40 to 60 ft $p = (26.0\,\text{lb/ft}^2)(0.85)(-0.3) - (26.0\,\text{lb/ft}^2)(0.18)$
p = **-11.3 lb/ft²** (outward)



EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

Solution for #5: To calculate the quantity and distribution of lateral seismic forces (base shear and vertical distribution):

Since the house is being elevated, the potential for seismic loading/overturning design loads will be increased on the home. Depending upon the amount of elevation, additional bracing of the roof or walls may be necessary.

Reference: ASCE 7-10 per Equivalent Lateral Force (ELF) procedure
Per equation 12.8-1 in ASCE 7-10 and equation 12.8-2 in ASCE 7-10:

$$V = C_s W$$

$$C_S = \frac{S_{DS}}{\left(R/I\right)}$$

Where:

- W = effective seismic weight
- C_S = seismic response coefficient
- I = occupancy importance factor = 1 per ASCE 7-10 Section 11.5.1 and Table 1.5-2
- R = response modification factor
- S_{DS} = design spectral response acceleration parameter in the short period range

Now, per ASCE 7-10 Table 12.14-1 and equations 12.8-11 and 12.8-12 in ASCE 7-10:
R = 2 for ordinary reinforced masonry shear wall foundation
R = 6.54 for framed walls with plywood

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$C_S = \frac{S_{DS}}{\left(R/I\right)}$$

$$C_S = \frac{0.2}{\left(2/1.0\right)}$$

$$C_S = 0.1$$

$$V = C_S W$$

Effective seismic weights:

$$W_1 = w_{roof} = 86{,}669 \text{ lbs at roof/upper ceiling level}$$

EXAMPLE C2. CALCULATE LATERAL LOADS (continued)

Recalculate $W_2 = w_{floor}$ without the effects of buoyancy on footing weight and lower foundation walls for conservative seismic condition.

Lower level masonry wall grouted at 48 in. o.c.: $(180\,\text{ft})(10.5\,\text{ft})(46\,\text{lb/ft}^2) = 86,940\,\text{lbs}$ **lbs** for the walls

Footing:

$(180\,\text{ft})(2\,\text{ft})(1\,\text{ft})(150\,\text{lb/ft}^2) =$ **54,000 lbs** for footing

$W_2 = w_{floor} = 34,200\,\text{lbs} + 4,500\,\text{lbs} + 112,460\,\text{lbs} + 54,000\,\text{lbs} + 89,100\,\text{lbs} = 294,260\,\text{lbs}$ at first floor level (without buoyancy force on components below the slab)

$W = 294,260\,\text{lbs} + 86,669\,\text{lbs} = 380,929\,\text{lbs}$ = total effective seismic weight

$V = 0.1(380,929)\,\text{lbs}$

$V = 38,093\text{lbs}$

$h_{roof} = 18\,\text{ft}$ = height at roof/upper ceiling $Total = 12,960\,\text{lbs} + 12,560\,\text{lbs} + 86,940\,\text{lbs} = 112,460$

$h_{floor} = 10\,\text{ft}$ = height at first floor

$w_{roof}h_{roof} = (86,669\,\text{lbs})(18\,\text{ft})$

$w_{roof}h_{roof} = 1,560,042$ ft-lb

$w_{floor}h_{floor} = (294,260\,\text{lbs})(10\,\text{ft})$

$w_{floor}h_{floor} = 2,942,600$ ft-lb

$$C_{vroof} = \frac{1,560,042\,\text{ft}-\text{lb}}{(1,560,042\,\text{f}\,t-\text{lb}) + (2,942,600\,\text{ft}-\text{lb})}$$

$C_{vroof} = 0.35$

$$C_{vfloor} = \frac{2,942,600\,\text{ft}-\text{lb}}{(1,560,042\,\text{f}\,t-\text{lb}) + (2,942,600\,\text{ft}-\text{lb})}$$

$C_{vroof} = 0.65$

$F_{roof} = (0.35)(38,093\,\text{lbs})$

$C_{vroof} =$ **13,332 lbs**

$F_{floor} = (0.65)(38,093\,\text{lbs})$

$C_{vroof} =$ **24,760 lbs**

EXAMPLE C2. CALCULATE LATERAL LOADS (concluded)

NOTE: For summary, see below:

Lateral Forces Perpendicular to Long Direction

Seismic

Level	Height (ft) h_x	Level Weight (kips) w_x	$(w_x)(h_x)$	Lateral Force (kips) F_x	Level Shear (kips) ΣF_x
2	18	83.97	1,511	13.3	13.3
1	10	303.03	3,030	24.8	38.1
		TOTAL	4,541	38.1	

Wind

Level	Wind Pressure (lb/ft²) P_x	Area (ft²) a_x	Lateral Force (kips) H_x	Level Shear (kips) ΣF_x
2	19.4	300	5.8	5.8
1	37.0	1,080	40.0	45.8

EXAMPLE C3. LOAD COMBINATIONS

Lateral Forces Parallel to Long Direction

Seismic

Level	Height (ft) h_x	Level Weight (kips) w_x	$(w_x)(h_x)$	Lateral Force (kips) F_x	Level Shear (kips) ΣF_x
2	18	83.97	1,511	13.3	13.3
1	10	303.03	3,030	24.8	38.1
		TOTAL	4,541	38.1	

Wind

Level	Wind Pressure (lb/ft^2) P_x	Area(ft^2) a_x	Lateral Force (kips) H_x	Level Shear (kips) ΣF_x
2	32.6	75	2.4	2.4
1	32.6	540	17.6	20.0

Given:

Per original Sample Calculation Statement:

- Solutions to above Example C1 and C2
- Field investigation information and additional information on the site

Find:

1. If the structure can resist sliding using the most appropriate load combination allowable stress design (ASD)
2. If the structure can resist overturning
3. If the structure can resist uplift and buoyancy forces

Solution for #1: Check if the structure can resist sliding as follows:

First, the most appropriate load combination for sliding must be found using ASCE 7-10 Section 2.4.1 for ASD, the load combinations are:

1. D
2. $D + L$
3. $D + (L_r$ or S or $R)$
4. $D + 0.75L + 0.75(L_r$ or S or $R)$
5. $D + (0.6W + 0.7E)$

6a. $D + 0.75L + 0.75(0.6W) + 0.75(L_r$ or S or $R)$

6b. $D + 0.75L + 0.75(0.7E) + 0.75S$

7. $0.6D + 0.6W$
8. $0.6D + 0.7E$

EXAMPLE C3. LOAD COMBINATIONS (continued)

When a structure is located in a flood zone, the following load combinations should be considered in addition to the basic combinations on Section 2.4.1 of ASCE 7-10:

- In Zone V or A Coastal Zone – 1.5 F_a should be added to load combination 5, 6, and 7 and E should be set equal to zero in Nos. 5 and 6
- In non-Coastal A Zones – 0.75 F_a should be added to load combination 5, 6, and 7 and E should be set equal to zero in Nos. 5 and 6

Each possible building failure mode (sliding, overturning, uplift, buoyancy) must be investigated using the most restrictive load combination.

- Controlling condition for sliding: The sum of the forces in the horizontal direction must be LESS than the sliding resistance provided by the soil in order for building to not slide. By inspection, load combination 7 is most restrictive sliding condition because dead load is reduced and flood and wind loads are included for the direction parallel to the ridge. By inspection, load combination 7 is also the most restrictive load combination equation for the direction perpendicular to the ridge, even though no flood load is involved in this direction.
- Weight of the structure used in sliding and overturning calculations must incorporate the use of flood vents. The buoyancy force for the slab and 5 ft of wall must be subtracted from the total weight of the structure:

$$D = 350\,\text{kips} + (0.33\,\text{ft})(1{,}800\,\text{ft}^2)(-62.4\,\text{lb/ft}^3) + (\frac{8}{12}\,\text{ft})(5\,\text{ft})(180\,\text{ft})(-62.4\,\text{lb/ft}^3) = 275\,\text{kips}$$

Parallel to ridge direction: The coefficient of soil friction is 0.3. Using load combination 7:

$0.6W + 0.75F_a < 0.6D$

$0.6(20.0\,\text{kips}) + 0.75(6.5\,\text{kips} + 2.2\,\text{kips}) = 18.5\,\text{kips}$ for the sliding force

$0.6(D)(0.3) = (0.6)(275\,\text{kips})(0.3) = 50\,\text{kips}$ for the sliding resistance providing by the soil

$19\,\text{kips} < 50\,\text{kips}$ OK in parallel to ridge direction

Perpendicular to ridge direction: The coefficient of soil friction is 0.3. Using load combination 7:

$0.6W + 0.75F_a < 0.6D$ where $F_a = 0$ (flow and flood-borne debris directions are parallel to this direction) and using the coefficient of soil friction 0.3

$0.6W < 0.6(D)(0.3)$

$0.6(45.8\,\text{kips}) = 27.5\,\text{kips}$ for the sliding force

$0.6(D)(0.3) = (0.6)(275\,\text{kips})(0.3) = 50\,\text{kips}$ for the sliding resistance providing by the soil

$27\,\text{kips} < 50\,\text{kips}$ OK in perpendicular to ridge direction

Therefore, the structure can resist sliding.

Solution for #2: Check if the structure can resist overturning as follows:

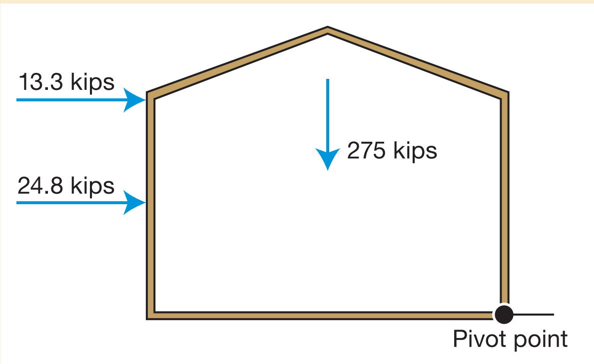
- Controlling condition for overturning: The moments caused by the lateral forces must be less than the counter-moment provided by the building weight.

EXAMPLE C3. LOAD COMBINATIONS (concluded)

The most likely overturning direction is along the short dimension of the building caused by seismic forces. The ASD factor for the seismic force is 0.7 and the ASD factor for the dead weight of the building is 0.6 per load combination 8.

$\sum M_{pivot} < 0$ for building to resist overturning

Summing the moments about the pivot point:



$$\sum M_{pivot} = 0.7(13.3\,\text{kips})(18\,\text{ft}) + 0.7(24.8\,\text{kips})(10\,\text{ft}) - 0.6(275\,\text{kips})(15\,\text{ft}) = -2134\,\text{kips}-\text{ft}$$

OK, building weight keeps building from overturning.

Therefore, the structure can resist overturning.

Solution for #3: Check if the structure can resist uplift and buoyancy forces as follows:

- Controlling condition for uplift and buoyancy: The dead weight of the building must resist the uplift forces imposed by wind and buoyancy on the building. The sum of the forces in the vertical direction must be greater than the buoyancy force in order for the building to stay in the ground. load combination 7 is most conservative. Buoyancy factored as flood load.

$0.6D + 0.6W + 0.75F_b < 0$ to resist uplift and buoyancy.

Per Example C1, the elevated structure is incapable of resisting buoyancy forces and flood openings are required.

NOTE: For this example analysis, the existing structural components were assumed to be adequate for the loading conditions. However, the designer should check the existing truss-to-wall-connections, plywood roof diaphragm, upper level walls, and floor diaphragm for their ability to resist increased loadings.

EXAMPLE C4. CHECK OF EXISTING FOUNDATION DESIGN

Given:

Per original Sample Calculation Statement:

- Solutions to above Examples C1, C2, and C3
- Field investigation information and additional information on the site

Find:

1. Adequacy of existing foundation design

Solution for #1: The adequacy of the existing foundation design can be determined as follows:

- All of the loads on the foundation must be calculated and tabulated using ASCE 7-10. Calculate loads along 60-foot exterior load bearing walls for highest footing loading condition.

Snow load = $S = (20\,\text{lb/ft}^2)(1\,\text{ft})(15\,\text{ft} + 2\,\text{ft}_{Overhang}) = 340\,\text{lb/lf}$

First flood live load= $L = (40\,\text{lb/ft}^2)(1\,\text{ft})(15\,\text{ft}/2) = 300\,\text{lb/lf}$

Dead loads:

Roof:
Shingles: $(15.81\,\text{ft} + 2\,\text{ft})(2\,\text{lb/ft}^2)(1\,\text{ft}) = 35.6\,\text{lb/lf}$
Felt: $(15.81\,\text{ft} + 2\,\text{ft})(0.7\,\text{lb/ft}^2)(1\,\text{ft}) = 12.5\,\text{lb/lf}$
Plywood: $(15.81\,\text{ft} + 2\,\text{ft})(1.5\,\text{lb/ft}^2)(1\,\text{ft}) = 26.7\,\text{lb/lf}$
Truss: $(15\,\text{ft} + 2\,\text{ft}\left(\dfrac{15\,\text{ft}}{15.81\,\text{ft}}\right))(5\,\text{lb/ft}^2)(1\,\text{ft}) = 84.5\,\text{lb/lf}$
$Total = 35.6\,\text{lb/lf} + 12.5\,\text{lb/lf} + 26.7\,\text{lb/lf} + 84.5\,\text{lb/lf} = 159.3\,\text{lb/lf}$

Ceiling:
Insulation: $(15\,\text{ft})(1\,\text{ft})(8\,\text{lb/ft}^2) = 120\,\text{lb/lf}$
Plywood: $(15\,\text{ft})(1\,\text{ft})(1.5\,\text{lb/ft}^2) = 22.5\,\text{lb/lf}$
Plaster: $(15\,\text{ft})(1\,\text{ft})(10\,\text{lb/ft}^2) = 150\,\text{lb/lf}$
Miscellaneous: $(15\,\text{ft})(1\,\text{ft})(2\,\text{lb/ft}^2) = 30\,\text{lb/lf}$
Wall (exterior): $(4\,\text{ft})(1\,\text{ft})(18\,\text{lb/ft}^2) = 72\,\text{lb/lf}$
Wall (interior): $\left(\dfrac{15\,\text{ft}}{2}\right)(1\,\text{ft})(20\,\text{lb/ft}^2) = 150\,\text{lb/lf}$
$Total = 120\,\text{lb/lf} + 22.5\,\text{lb/lfl} + 150\ \text{b/lf} + 30\,\text{lb/lfl} + 72\ \text{b/lf} + 150\,\text{lb/lf} = 544.5\,\text{lb/lf}$

First Floor:
Flooring: $\left(\dfrac{15\,\text{ft}}{2}\right)(1\,\text{ft})(4\,\text{lb/ft}^2) = 30\,\text{lb/lf}$

EXAMPLE C4. CHECK OF EXISTING FOUNDATION DESIGN (concluded)

Subfloor: $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(3\,\text{lb/ft}^2) = 22.5\,\text{lb/lf}$

Joists: $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(4\,\text{lb/ft}^2) = 30\,\text{lb/lf}$

Insulation: $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(5\,\text{lb/ft}^2) = 37.5\,\text{lb/lf}$

Miscellaneous: $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(3\,\text{lb/ft}^2) = 22.5\,\text{lb/lf}$

Ceiling: $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(2.5\,\text{lb/ft}^2) = 18.8\,\text{lb/lf}$

Wall (exterior): $(4\,\text{ft})(1\,\text{ft})(18\,\text{lb/ft}^2) = 72\,\text{lp/lf}$

Footing: $(2\,\text{ft})(1\,\text{ft})(150\,\text{lb/ft}^3) = 300\,\text{lb/lf}$

Wall (interior): $\left(\frac{15\,\text{ft}}{2}\right)(1\,\text{ft})(20\,\text{lb/ft}^2) = 150\,\text{lb/lf}$

New lower level wall: $(10\,\text{ft})(1\,\text{ft})(46\,\text{lb/ft}^2) = 460\,\text{lb/lf}$

$Total = 30\,\text{lb/lf} + 22.5\,\text{lb/lf} + 30\,\text{lb/lf} + 37.5\,\text{lb/lf} + 22.5\,\text{lb/lf} + 18.8\,\text{lb/lf} + 72\,\text{lb/lf} + 300\,\text{lb/lf} +$
$150\,\text{lb/lf} + 460\,\text{lb/lf} = 1{,}143.3\,\text{lb/lf}$

$Total_{DL} = 159.3\,\text{lb/lf} + 544.5\,\text{lb/lf} + 1{,}143.3\,\text{lb/lf} = 1{,}847.1\,\text{lb/lf}$

NOTE: A 20 lb/ft² partition load is applied here; this approach is conservative due to the amount of interior walls in this building.

From our field investigation it was determined that an allowable bearing pressure of 2,000 lb/ft² was acceptable.

- The total load on the foundation must be determined

Total load on foundation:
Most restrictive load combination is Eq.6a.

$TL = D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S \text{ or } R)$

$TL_{foundation} = 1{,}847.1\,\text{lb/lf} + 0.75(300\,\text{lb/lf}) + 0.75(340\,\text{lb/lf}) = 2{,}327\,\text{lb/lf}$

- The total load on the foundation must be compared with the capacity of the existing foundation to determine adequacy

The existing foundation is 2 ft wide; thus, the bearing pressure for total loads is

$\left[\frac{2{,}327\,\text{lb/lf}}{2\,\text{ft}}\right] = 1{,}164\,\text{lb/ft}^2 < 2{,}000\,\text{lb/ft}^2$ Allowable, OK.

Therefore the existing foundation design is adequate.

EXAMPLE C5. NEW FOUNDATION WALL DESIGN

Given:

Per original Sample Calculation Statement:

- Solutions to above Examples C1, C2, C3, and C4
- Assume seismic load condition controls design
- Field investigation information and additional information on-site as noted in previous sections
- Assume #4 reinforcing bars at 48 in. o.c. in solid grouted cores
- G =0.4*E* where *E* = modulus of elasticity per American Concrete Institute (ACI) 530-08 Section 1.8.2.2
- Assume compressive strength of masonry, f'_m = 2,000 lb/in.^{2}
- Type M or S mortar per ACI 530-08 Section 1.8.2.2 $E_m = 900 f'_m = 900$ (2,000) = 1.8 x 10^6 lb/in.^{2}
- With #4 @ 48 in. o.c. the equivalent solid thickness is 4.6 in. = 0.38 ft
- *M* = 120 kips-ft from Example C2 (maximum moment split in half due to 2 walls)
- *V* = 19.2 kips from Example C2 (maximum shear split in half due to 2 walls)

Find:

1. Design the foundation wall and connection to the footing
2. Design top of wall connection. (Checking anchor bolts for pullout from masonry

Solution for #1: To design the foundation wall connection to the footing:

New Wall Design:

- Minimum wall reinforcement is #4 @ 48 in. A_s = 0.20/48 in.
- Next determine V_m (shear strength provided by masonry) per ACI 530-08 Equation 3-22
- Moment is calculated from Seismic Lateral Forces. Refer to the figure below for indicated forces with associated moment arms.

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd}\right)\right] A_n \sqrt{f'_m} + 0.25P$$

$$M = (\frac{24.8\,\text{kips}}{2})(10\,\text{ft}) + (\frac{13.3\,\text{kips}}{2})(18\,\text{ft}) = 243.7\,\text{kips-ft} = 2{,}924{,}000\,\text{in.} - \text{lb}$$

$$V = 19.1\,\text{kips}$$

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (continued)

$d = (30\,\text{ft})(12) = 360\,\text{in.}$

$A_n = 120\,\text{in.}^2$

$f'_m = 2{,}000\,\text{lb/in.}^2$

$P = (189\,\text{lb/ft})(30\,\text{ft}) = 5{,}670\,\text{lbs} = 5.67\,\text{kips}$

Where P is equal to the average weight of the 30 ft long framed wall with gable end wall above. So,

$$V_m = \left[4.0 - 1.75\left(\frac{2{,}924\,\text{kips-in.}}{(19.1\,\text{kips})(360\,\text{in.})}\right)\right](120\,\text{in.}^2)\sqrt{2{,}000\,\text{lb/in.}^2} + 0.25\,(5{,}670\,\text{lbs})$$

$= (17{,}473 + 1{,}418)\,\text{lbs} = 18.9\,\text{kips}$

- Finding the resulting shear force per ACI-530-08 Equation 3-23:

$$V_s = 0.5\left(\frac{A_V}{s}\right)f_v d_v = 0.5\left(\frac{0.20}{48}\right)(60\,\text{kips})(360) = 45\,\text{kips}$$

$$\frac{M}{Vd} = \frac{243.7\,\text{kips-ft}}{(19.1\,\text{kips})(30\,\text{ft})} \le 1 \text{ which is acceptable}$$

The resulting shear force per ACI 530-08 Equation 2-25 is:

$$F_v = \left[\frac{1}{3}\left(4 - \frac{M}{Vd}\right)\right]{}_n\sqrt{f'_m} = \left[\frac{1}{3}(4 - 0.43)\right]{}_n\sqrt{2{,}000} = 53.2\,\text{lb/in.}^2$$

- Determining the nominal shear strength and comparing to the in-plane shear capacity

Nominal Shear Strength $V_n = V_m + V_s = 18.9\,\text{kips} + 45\,\text{kips} = 63.9\,\text{kips}$

63.9 kips > 53.2 kips OK #4 @ 48 in. o.c. in-plane shear

- Investigate long (60 ft) wall for out-of-plane bending because axial load is also supported by this wall. **Neglect impact load in this analysis**.

Treat wall as simple T-beam 4 ft wide and calculate axial pressure per load determined in Example C4. Section Properties ACI 580-08 Section 1.9 and

$$f_a = \frac{(23{,}000\,\text{lbs})(4)}{(48\,\text{in.})(7.5\,\text{in.})} = 25.6\,\text{lb/in.}^2$$

for $\frac{h}{r} \le 99$ h/r ≤ 99 h/r = 120/2.16 = 55.6 < 99

$$= \frac{120}{2.16} \le 99 = 55.6 \le 99 \text{ OK}$$

where: h = $(10)(12) = 120\,\text{in.}$

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (continued)

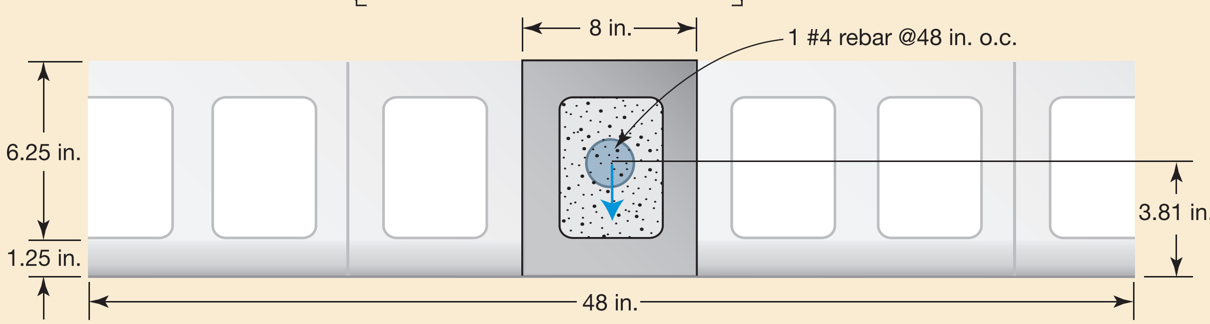
$$r = \sqrt{\frac{t^2}{12}} = \sqrt{\frac{7.5^2}{12}} = 2.16$$

Axial Compressive Strength of Masonry per ACI-530-08 Equation 2-15. Maximum moment determined from Simple Beam Load Combinations #1 and #5 from Part 4 of *Steel Construction Manual, Volume 1* (2nd Edition) using wind and hydrodynamic flood pressures found in Example C2 and shown in the figure below:

$$F_a = \frac{1}{4} f'_m \left[1 - \left(\frac{h}{(140)r}\right)^2\right] = \frac{1}{4}(2{,}000)\left[1 - \left(\frac{120}{(140)(2.16)}\right)^2\right] = 421\,\text{lb/ in.}^2$$

$$M_{max} = \left[\frac{wl^2}{8}\right] + \frac{[(\frac{w_p a}{2l})(2l - a)]^2}{2w_p}$$

$$M_{max} = \left[(4)\frac{(21.3)(10)^2}{8}\right] + \left[(4)\frac{[(\frac{(22.4)(5)}{2(10)})((2)(10) - 5)]^2}{(2)(22.4)}\right] = 1{,}065 + 630 = 1{,}695\,\text{ft} - \text{lbs}$$



	Area (in.²)	δ	Aδ	Aδ²	I	$(I_x)_i$ (in.⁴)
	(1.25)(48)=60	0.625	37.5	23.44	7.1	31.25
	(8)(6.75)=54	3.81	205.74	783.87	205.03	988.90
Sum	**114**		**243.24**			**1,020.15**

Calculate section modulus for T-beam section of wall

$$y = \frac{243.24}{114} = 2.13\,\text{in.}$$

$$I_{NIA1} = I_x - Ay^2 = 1{,}020.15\,\text{in}^4 - (114\,\text{in.}^2)(2.13\,\text{in.})^2 = 502.94\,\text{in.}^4$$

$$S = \frac{I_{N1A1}}{y} = \frac{502.94}{2.13} = 236.12\,\text{in.}^3$$

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (continued)

$$f_b = \frac{M}{S} = \frac{1{,}695\,\text{ft}-\text{lbs}(12\,\text{in./ft})}{236.12\,\text{in.}^3} = 86\,\text{lb/in.}^2$$

$$F_b = \frac{1}{3} f'_m = 667\,\text{lb/in.}^2$$

- Check combined bending and axial load using interaction equation per ACI 530-08 Equation 2-13. Maximum Shear determined from End Reactions per Simple Beam Load Combinations #1 and #5 from Part 4 of *Steel Construction Manual, Volume 1* (Second Edition) using the wind and hydrodynamic flood pressures found in Example C2. Masonry shear stress *(f_v)* per ACI 530-08 Equation 2-23 and Equation 2-24:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$$

$$\frac{25.6}{421} + \frac{86}{667} \le 1 \text{ OK}$$

$$R_{Top} = (4)\left(\frac{(21.3)(10)}{2} + \frac{(22.4)(5)^2}{(2)(10)}\right) = 538\,\text{lbs}$$

$$R_{Bottom} = (4)\left(\frac{(21.3)(10)}{2} + \frac{(22.4)(5)}{(2)(10)}[(2)(5)]\right) = 762\,\text{lbs}$$

$$f_{v\_WallBase} = \frac{V}{bd} = \frac{762}{(7.5)(12)} = 8.5\,\text{lb/in.}^2$$

$$F_v = 50\,\text{lb/in.}^2 > f_v \text{ OK}$$

- Reaction R_{Top} must be resisted by attachment of floor diaphragm to wall by bolts and wood sill plate.
- Investigate pure bending of wall with maximum moment as determined at beginning of Section C5 and Section Properties per ACI 580-08 Section 1.9.

$M = kbd^2$ where M = 1,695 ft-lbs or 20,340 in.-lbs, b = 48 in., d = 3.81 in.

$$k = pf_s j = \left(\frac{A_s}{bd}\right) f'_s j \text{ and } k = \frac{n}{n+r} \text{ where } n = \frac{E_s}{E_m} \text{ and } r = \frac{f_s}{f_m}$$

$$r = \frac{24{,}000}{2{,}000} = 12$$

$$k = \frac{16.1}{(16.1+12)} = 0.573$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.573}{3} = 0.809$$

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (continued)

So, the required A_s for M = 1,695 ft-lb and 48 in. wide T-beam is:

$$A_{s\_required} = \left(\frac{(k)(b)(d)}{f_s j}\right) = \left(\frac{29.2(48)(3.81)}{(24{,}000)(0.809)}\right) = 0.275\,\text{in.}^2 \text{ N.G., -1 -\#4 BAR=0.20 in.}^2$$

Solution: Increase reinforcement to 5/8-in. diameter with $A_s = 0.307\,\text{in.}^2 > 0.275\,\text{in.}^2$

0.31 in.^{2} (1 -#5) OK

Therefore, use 1-#5 @ 48 in. o.c. to resist bending out-of-plane.

- Bolt Design

Shear at top of wall = $\dfrac{(538/2)\,\text{lbs}}{4\,\text{ft}} = 67\,\text{lb/ft}$

Check shear capacity of 0.5 in. φ A307 A.B. in Southern Pine #2 sill plate. Use National Design Specification for Wood (2005), Table 11E. Try sill plate 2 in. x 6 in. so side member = 1-1/2 in., main member = 3 in., shear perpendicular to grain = 410 lbs. Modify value with adjustment factors for connections.

So $Z' = ZC_dC_mC_tC_gC_A = (410)(1.29)(1.0)(1.0)(1.0)(1.0) = 529\,\text{lbs}$

And,

Bolt spacing = $\dfrac{529\,\text{bs}}{67\,\text{lbs/ft}} = 7.9\,\text{ft} = 95\,\text{in.}$, use 72 in. max. as required by IRC 2012 Section R403.1.6

Edge distance of bolt 4D = $4(0.5) = 2\,\text{in.}$

Check bending of sill plate with bolts @ 6 ft o.c.

$$M = \frac{67\,\text{lb/ft}(6\,\text{ft})^2}{8} = 302\,\text{ft-lb}$$

$$S_{2x6} = 7.56\,\text{in.}^3$$

$$f_b = \frac{M}{S} = \frac{302\,\text{ft-lb}(12)\,\text{in./ft}}{7.56\,\text{in.}^3} = 479\,\text{lb/in.}^2$$

F_b for Southern Pine #2 = 1,500 lb/in.^{2}, $f_b < F_b$ OK

Check pullout of A.B. Uplift Force = $\dfrac{(13.3\,\text{kips})(9\,\text{ft})}{(60\,\text{ft})(30\,\text{ft})} = 67\,\text{lb/ft}$

For anchors spaced 6 ft o.c., uplift force = 402 lb/bolt

Therefore, an 8 in. CMU wall with #5 @ 48 in. o.c. centered on grouted cell - 2,000 lb/in.^{2} masonry *(f'_m)* is acceptable.

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (continued)

Solution for #2: To design the top of wall connection, one must check anchor bolts for pullout from masonry as follows:

Try ½ in. φ A307 anchor bolts @ 6 ft o.c.

Uplift on bolt =402 lb/bolt

Try ½ in. φ A307 anchor bolt, area of bolt, A_b = 0.2 in.^{2}

Edge distance, $l_{be} = 0.5(7.625 - 0.5) = 3.56\,\text{in}$.

Embedment, l_b = 7 in. Required minimum embedment for reinforced masonry IRC 2012 Section R403.1.6

Reference: ACI 530-08 Section 2.1.4 and ACI-530-08 Equation 2-10

$$A_{pt} = \pi l_b^{\,2} = \pi(15)^2 = 707\,\text{in}^2$$

$$A_{pv} = \frac{1}{2}\pi l_{be}^{\,2} = (0.5)\pi(3.56)^2 = 19.9\,\text{in.}^2$$

Allowable load in tension:

$$B_a = \min\,(1.25(A_{pt})\sqrt{f'_m}, 0.6A_b f_y)$$

Where:

A_b = Area of Anchor Bolt

$A_{pt} = \pi l_b^{\,2} = \pi(15)^2 = 707\,\text{in.}^2$

f'_m = Compressive Strength of Masonry

f_y = Yield Strength of Anchor Bolt

For this anchor bolt pattern,

$B_a = \min(((1.25)(707)(2{,}000)^{0.5}), (0.6)(.20)(30{,}000)) = \min(39{,}500,\ 3{,}600) = 3{,}600\,\text{lbs} > 402\,\text{lbs}$ OK

Allowable load in shear,

$$B_v = \min(1.25A_{pv}\sqrt{f'_m}, 350((f'_m)A_b)^{0.25}, 2.5A_{pt}\sqrt{f'_m}, .36A_b(f_y))$$

$$B_v = \min(((1.25)(19.9)(2{,}000)^{0.5}), 350((2{,}000)(.20))^{0.25}, (2.5)(707)\sqrt{2{,}000}, (.36)(.20)(30{,}000))$$

$$B_v = \min(1{,}100,\ 1{,}565,\ 79{,}000,\ 2{,}160) = 1{,}110\,\text{lbs} > 269\,\text{lbs OK}$$

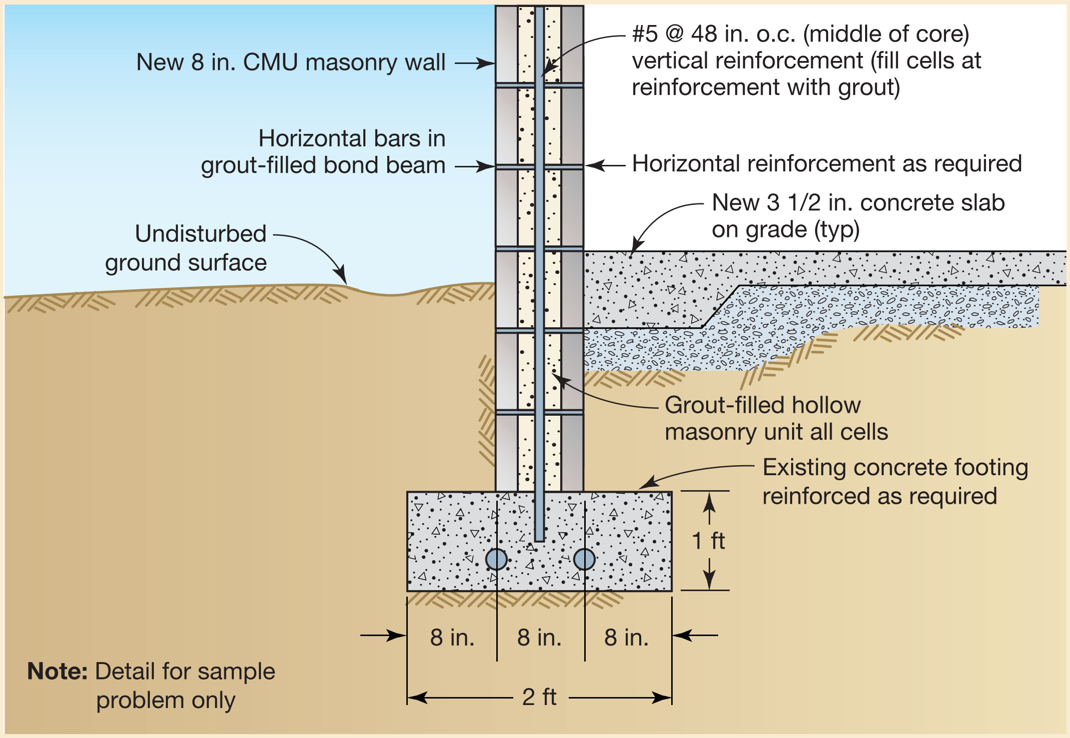
Per ACI 530-08 Section 2.1.4.3.3, the combined ratio is

$$\frac{b_a}{B_a} + \frac{b_v}{B_v} = \frac{402}{3{,}600} + \frac{269}{1{,}100} \le 1$$

EXAMPLE C5. NEW FOUNDATION WALL DESIGN (concluded)

Therefore, a ½ in. φ A307 anchor bolt with an edge distance, l_{be} = 3.56 in. and embedment, l_b = 15 in. is adequate.

Sample Bearing Wall Detail



Dry Floodproofing

EXAMPLE C6. SAMPLE CALCULATION FOR SUMP PUMP

Sump Pump Sizing Calculations

Given:

- The drain line is 1.5 in. diameter steel pipe, which is 30 ft long and includes one elbow, one gate valve, and one check valve (based on hydraulic engineering handbooks)
- Q = the flow rate of the sump pump = 20 gal/min
- V_{fps} = Velocity of flow through the pipe in ft/sec
- A_{pipe} = Area of the discharge pipe cross-section in ft
- Z = the elevation difference between the bottom of the sump and the point of discharge, in ft = 10 ft
- $h_{f\text{-}pipe}$ = head loss due to pipe friction (for this example use 2.92 ft per 100 ft)
- $h_{f\text{-}fittings}$ = head loss through fittings, in ft
- $h_{f\text{-}trans}$ = head loss though transitions, in ft
- K_b (elbow) = 0.63
- K_b (gate valve) = 0.15
- K_e (pipe entrance) = 0.5
- K_o (sudden enlargement/outlet) = 1.0
- g = weight of gravity, 32.2 ft/sec
- TH = total head, in ft

Find:

1. Velocity converted from gal/min to ft/sec
2. Calculate the losses from pipe fittings and pipe transitions
3. Calculate the losses over the length of pipe
4. Calculate the total dynamic head for the sump discharge

Solution for #1: To convert the velocity from gal/min to ft/sec:

$$V_{fps} = \frac{Q}{(7.48\ {}^{\text{gal}}/_{\text{ft}^3})(60\ {}^{\text{sec}}/_{\text{min}})A_{pipe}}$$

EXAMPLE C6. SAMPLE CALCULATION FOR SUMP PUMP (concluded)

$$V_{fps} = \frac{20\,{}^{gal}\!/\!_{min}}{(7.48\,{}^{gal}\!/\!_{ft^3})(60\,{}^{sec}\!/\!_{min})(3.14)\left(\dfrac{0.75\,\text{in.}}{12\,{}^{in.}\!/\!_{ft}}\right)^2}$$

V_{fps} = **3.63 ft/sec**

Solution for #2: The losses from pipe fittings and pipes transitions can be calculated as follows:

$$h_{f-fittings} + h_{f-trans} = (k_b + k_e + k_o)\left(\frac{V_{fps}}{2g}\right)$$

$$h_{f-fittings} + h_{f-trans} = (0.63 + 0.15 + 2.1 + 0.5 + 1.0)\left(\frac{(3.63\,{}^{ft}\!/\!_{sec})^2}{2(32.2\,{}^{ft}\!/\!_{sec})}\right)$$

$h_{f-fittings} + h_{f-trans}$ = **0.25 ft**

Solution for #3: To calculate the losses over the length of the pipe:

$$h_{pipe} = 2.92\,\text{ft}\left(\frac{30\,\text{ft}}{100\,\text{ft}}\right) = \mathbf{0.876\ ft}$$

Solution for #4: To calculate the total dynamic head:

$$TH = Z + h_{f-pipe} + h_{f-fittings} + h_{f-trans}$$

TH = **11.772 ft**

NOTE: Therefore select a pump capable of pumping 20 gal/min at 11.77 ft of total head.

Wet Floodproofing

EXAMPLE C7. NET BUOYANCY FORCE ON A LIQUID PROPANE TANK

Given:

- F_b = the net buoyancy force of the tank, in lbs
- V_t = the volume of the tank in gallons = 250 gal
- 0.134 is a factor to convert gal to ft^3
- γ = the specific weight of flood water surrounding the tank (generally 62.4 lb/ft^3 for fresh water and 64.1 lb/ft^3 for salt water)
- FS = the factor of safety to be applied to the computation, typically 1.3 for tanks
- W_t = the weight of the tank = 670 lbs (empty – worst case scenario)
- V_c = the volume of concrete required, in ft^3
- S_c = the effective weight of concrete, typically 150 lb/ft^3

Find:

1. The net buoyancy force of the tank in pounds
2. The volume of concrete required to offset the buoyant force

Solution for #1: The net buoyancy force of the liquid propane tank can be found as follows:

$$F_b = \left[0.134 V_t \gamma FS\right] - W_t$$

$$F_b = 0.134(250\,\text{gal})(62.4\,\text{lbs/ ft}^3)(1.3) - 670\,\text{lbs}$$

F_b = **2,048 lb**

Solution for #2: The volume of concrete required to offset the buoyant force can be determined as follows:

$$V_c = \frac{F_b}{S_c - \gamma}$$

$$V_c = \frac{2{,}048\,\text{lbs}}{(150\,\text{lb/ ft}^3 - 62.4\,\text{lb/ ft}^3)}$$

Where:
V_c = **23.4 ft³**

EXAMPLE C7. NET BUOYANCY FORCE ON A LIQUID PROPANE TANK (concluded)

NOTE: To resist this buoyant force, a slab of concrete with a volume, V_c, is usually strapped to the tank to resist the buoyant load.

Floodwalls and Levees

EXAMPLE C8. FLOODWALL DESIGN

Given:
Site soil conditions based on clean dense sand:

- γ_{soil} (unit weight of soil) = 120 lbs/ft^3
- S_a (allowable soil bearing capacity) = 2,000 lbs/ft^2
- S (equivalent fluid pressure of soil) = 78 lbs/ft^3
- C_f (coefficient of friction) = 0.47
- k_p (passive soil pressure coefficient) = 3.69
- C_s (cohesion) = 0

Local flood conditions:

- Fresh water (γ_w = 62.4 lbs/ft^3)
- Area of potential normal impact loading, C_B = 0.2 (moderate upstream blocking), C_{Str} = 0.8
- Expected flood velocity, V (ft/sec)

Dimensional information

- H = 7.0 ft
- D_t = 4.0 ft
- $D = D_h$ = 5.0 ft
- t_{ftg} = *1.0* ft
- B = *5.0* ft
- A_h = *2.5* ft

EXAMPLE C8. FLOODWALL DESIGN (continued)

- C = 1.5 ft
- t_{wall} = 1.0 ft

Find:

1. Design a cantilever floodwall to protect a residence subject to 3 ft of flooding.

Solution for #1: To design a cantilever floodwall to protect a residence subject to 3 ft of flooding, follow the eight step process for Floodwall Design in Chapter 5F:

Step 1: Assume wall height and footing depth (see Figure 5F-15 in Chapter 5F)

- H = 7.0 ft
- D_t = 4.0 ft
- D = D_h = 5.0 ft
- t_{ftg} = 1.0 ft

Step 2: Determine dimensions (see Figure 5F-15 in Chapter 5F)

- B = 5.0 ft
- A_h = 2.5 ft
- C = 1.5 ft
- t_{wall} = 1.0 ft

Wall and footing to be reinforced concrete having unit weight (S_g) of 150 lbs/ft³.

Step 3: Calculate forces.
Determine Lateral Forces:

Equation 4-4: $f_{sta} = \dfrac{1}{2}\gamma_w H^2 = (0.5)(62.4)(7)^2 = 1{,}528.8\text{ lbs/ lf}$

Equation 4-5: $f_{dif} = \dfrac{1}{2}(S - \gamma_w)D^2 = (0.5)(78 - 62.4)(5)^2 = 195.0\text{ lbs/ lf}$

Equation 4-7: $dh = \dfrac{C_d V^2}{2g} = dh = \dfrac{(1.25)(5)^2}{2(32.2)} = 0.49\text{ ft}$

Equation 4-8: $f_{dh} = \gamma_w (dh) H = (62.4)(0.49)(7) = 214.0\text{ lbs/ lf}$

$F_n = WVC_D C_B C_{Str} = (1{,}000)(5)(0.5)(0.2)(0.8) = 400\text{ lbs/ lf}$

Equation 5F-9: $F_{sta} = f_{sta} + f_{dif} + f_{dh} = 1{,}528.8 + 195 + 214 = 1{,}937.8\text{ lbs/ lf}$

EXAMPLE C8. FLOODWALL DESIGN (continued)

Since F_n acts only at a single point, we will not include loading into the uniform lateral floodwall loading. Once the floodwall is sized, we will evaluate the wall perpendicular to flow to determine ability to resist the impact loading. If necessary this wall will be redesigned to resist impact loads. This process will avoid overdesigning of the entire floodwall.

Equation 5F-12: $F_p = \frac{1}{2}\left[k_p(\gamma_{soil} - \gamma_w) + \gamma_w\right]D_t^2 =$

$0.5\left[3.69(120 - 62.4) + (62.4)\right](4)^2 = 2{,}199.6 \text{ lb/ lf}$

Determine Vertical Forces:

Equation 5F-1: $f_{buoy} = \frac{1}{2}\gamma_w HB = (0.5)(62.4)(7)(5) = 1{,}092 \text{ lbs/ lf}$

Equation 5F-1: $f_{buoy2} = \frac{1}{2}\gamma_w DtB(0.5)(62.4)(4)(5) = 624 \text{ lbs/ lf}$

Equation 5F-1: $f_{buoy} = f_{buoy1} + f_{buoy2} = 1{,}092 + 624 = 1{,}716 \text{ lbs / lf}$

Equation 5F-2: $w_{wall} = (H - t_{ftg})t_{wall}S_g = [7 - 1](1)(150) = 900 \text{ lbs/ lf}$

Equation 5F-3: $w_{ftg} = Bt_{ftg}S_g = (5)(1)(150) = 750 \text{ lbs/ lf}$

Equation 5F-4: $w_{st} = C(D_t - t_{ftg})(\gamma_{soil}) = 1.5(4 - 1)(120) = 540 \text{ lbs/ lf}$

Equation 5F-5: $w_{sh} = A_h(D_h - t_{ftg})(\gamma_{soil} - \gamma_w) = 2.5(5 - 1)(120 - 62.4) = 576 \text{ lbs/ lf}$

Equation 5F-6: $w_{wh} = A_h(H - t_{ftg})(\gamma_w) = 2.5(7 - 1)(62.4) = 936 \text{ lbs/ lf}$

Equation 5F-7: $w_G = w_{wall} + w_{ftg} + w_{st} + w_{sh} + w_{wh} = 900 + 750 + 540 + 576 + 936 = 3{,}702 \text{ lbs/ lf}$

Equation 5F-8: $F_v = w_G - f_{buoy} = 3{,}072 - 1{,}702 = 1{,}986 \text{ lbs/ lf} > 0$

Step 4: Check sliding.

Equation 5F-10: $F_{fr} = C_f F_v = 0.47(1{,}986) = 933.4 \text{ lbs/ lf}$

Equation 5F-11: $F_c = C_s B = 0(5) = 0$

Equation 5F-13: $F_R = F_{fr} + F_c + F_p = 933.4 + 0 + 2{,}199.6 = 3{,}133 \text{ lbs/ lf}$

Equation 5F-14: $FS_{(SL)} = \frac{F_R}{F_{sta}} = \frac{(3{,}133)}{(1{,}937.8)} = 1.6 > 1.5$ (recommended) OK for sliding

Step 5: Check overturning.

EXAMPLE C8. FLOODWALL DESIGN (continued)

Equation 5F-15:

$$M_O = F_{sta}\left(\frac{H}{3}\right) + f_{dif}\left(\frac{D}{3}\right) + f_{buoy1}\left(\frac{2B}{3}\right) + \left[f_{dh}\left(\frac{H}{2}\right) \ or\ F_d\left(H - \frac{D_h}{2} + D_h\right)\right] + F_n H \ or\ F_s H + f_{buoy2}\left(\frac{B}{3}\right)$$

$$= (1,937.8)\left(\frac{7}{3}\right) + (195)\left(\frac{5}{3}\right) + (1,092)\left(\frac{2(5)}{3}\right) + \left[(214)\left(\frac{7}{2}\right)\right] + (624)\left(\frac{5}{3}\right)$$

= 10,276 ft-lbs/lf

Equation 5F-16: $M_R = w_{wall}\left(C + \frac{t_{wall}}{2}\right) + w_{ftg}\left(\frac{B}{2}\right) + w_{st}\left(\frac{C}{2}\right) + w_{sh}\left(B - \frac{A_h}{2}\right) + w_{wh}\left(B - \frac{A_h}{2}\right) + F_p\left(\frac{D_t}{3}\right)$

$$= (900)\left((1.5) + \frac{1}{2}\right) + (750)\left(\frac{5}{2}\right) + (540)\left(\frac{1.5}{2}\right) + (576)\left((5) - \frac{2.5}{2}\right) + (936)\left((5) - \frac{2.5}{2}\right) + (2,199.6)\left(\frac{4}{3}\right)$$

= 12,683 ft-lbs/lf

Equation 5F-17: $FS_{(OT)} = \frac{M_R}{M_O} = \frac{(12,683)}{(10,276)}$ = 1.2 < 1.5 (recommended) No Good

Try increasing the footing size to overcome the overturning moment. Assume B = 7.0 ft; A_h = 4.0 ft; and C = 2.0 ft. This requires revision of Steps 3 and 4 for which the results are shown below. f_{sta}, f_{dif}, f_{dh}, F_{sta}, F_p, w_{wall} will not change. Recompute vertical forces.

Equation 5F-1: $f_{buoy1} = \frac{1}{2}\gamma_w HB = (0.5)(62.4)(7)(7) = 1,528.8$ lbs/ lf

Equation 5F-1: $f_{buoy2} = \frac{1}{2}\gamma_w D_t B = (0.5)(62.4)(4)(7) = 873.6$ lbs/ lf

Equation 5F-1: $f_{buoy} = f_{buoy1} + f_{buoy2} = 1,528.8 + 873.6 = 2,402.4$ lbs/ lf

Equation 5F-2: $w_{wall} = (H - t_{ftg})t_{wall}S_g = [7 - 1](1)(150) = 900$ lbs/ lf

Equation 5F-3: $w_{ftg} = Bt_{ftg}S_g = (7)(1)(150) = 1,050$ lbs/ lf

Equation 5F-4: $w_{st} = C(D_t - t_{ftg})(\gamma_{soil}) = 2(4-1)(120) = 720$ lbs/ lf

Equation 5F-5: $w_{sh} = A_h(D_h - t_{ftg})(\gamma_{soil} - \gamma_w) = 4(5-1)(120 - 62.4) = 921.6$ lbs/ lf

Equation 5F-6: $w_{wh} = A_h(H - t_{ftg})(\gamma_w) = 4(7-1)(62.4) = 1,497.6$ lbs/ lf

Equation 5F-7: $w_G = w_{wall} + w_{ftg} + w_{st} + w_{sh} + w_{wh} =$
$900 + 1,050 + 720 + 921.6 + 1,497.6 = 5,089.2$ lbs/ lf

Equation 5F-8: $F_v = w_G - f_{buoy} = 5,089.2 - 2,402.4 = 2,686.8$ lbs/ lf > 0

Recheck Sliding

Equation 5F-10: $F_{fr} = C_f F_v = 0.47(2,686.8) = 1,262.8$ lbs/ lf

Equation 5F-11: $F_c = C_s B = 0(7) = 0$

EXAMPLE C8. FLOODWALL DESIGN (continued)

Equation 5F-13: $F_R = F_{fr} + F_c + F_p = 1,262.8 + 0 + 2,199.6 = 3,462.4$ lbs/ lf

Equation 5F-14: $FS_{(SL)} = \frac{F_R}{F_{sta}} = \frac{(3,462.4)}{(1,937.8)}$ = 1.79 > 1.5 (recommended) OK for sliding

Recheck Overturning

Equation 5F-15:

$$M_O = F_{sta}\left(\frac{H}{3}\right) + f_{dif}\left(\frac{D}{3}\right) + f_{buoy1}\left(\frac{2B}{3}\right) + \left[f_{dh}\left(\frac{H}{2}\right) \text{ or } F_d\left(H - \frac{D_h}{2} + D_h\right)\right] + F_nH \text{ or } F_sH + f_{buoy2}\left(\frac{B}{3}\right)$$

$$= (1,937.8)\left(\frac{7}{3}\right) + (195)\left(\frac{5}{3}\right) + (1,528.8)\left(\frac{2(7)}{3}\right) + \left[(214)\left(\frac{7}{2}\right)\right] + (873.6)\left(\frac{7}{3}\right)$$

= 14,748 ft-lbs/lf

Equation 5F-16: $M_R = w_{wall}\left(C + \frac{t_{wall}}{2}\right) + w_{ftg}\left(\frac{B}{2}\right) + w_{st}\left(\frac{C}{2}\right) + w_{sh}\left(B - \frac{A_h}{2}\right) + w_{wh}\left(B - \frac{A_h}{2}\right) + F_p\left(\frac{D_t}{3}\right)$

$$= (900)\left((2) + \frac{1}{2}\right) + (1,050)\left(\frac{7}{2}\right) + (720)\left(\frac{2}{2}\right) + (921.6)\left((7) - \frac{4}{2}\right) + (1,497.6)\left((7) - \frac{4}{2}\right) + (2,199.6)\left(\frac{4}{3}\right)$$

= 21,674 ft-lbs/lf

Equation 5F-17: $FS_{(OT)} = \frac{M_R}{M_O} = \frac{(21,674)}{(14,748)}$ = 1.5 = 1.5 (recommended) OK

Step 6: Determine eccentricity.

Equation 5F-18: $e = \left(\frac{B}{2}\right) - \left(\frac{(M_R - M_O)}{F_v}\right) = \left(\frac{7}{2}\right) - \left(\frac{(21,674 - 14,748)}{2,686.8}\right) = 0.92 < \frac{7}{6}$ OK

Step 7: Check soil pressure.

Equation 5F-19: $q = \left(\frac{F_v}{B}\right)\left(1 \pm \left(\frac{6e}{B}\right)\right)$

$$q_{min} = \left(\frac{2,686.8}{7}\right)\left(1 - \left(\frac{6(0.88)}{7}\right)\right) = 94.3 \text{ lbs/ft}^2$$

$$q_{max} = \left(\frac{2,686.8}{7}\right)\left(1 + \left(\frac{6(0.88)}{7}\right)\right) = 673.3 \text{ lbs/ft}^2 < 2,000 \text{ lbs/ft}^2 \text{ OK}$$

Step 8: Select reinforcing steel.

For steel in the vertical floodwall section:

Equation 5F-20 (note): $M_b = F_{sta}\left(\frac{H}{3} - t_{ftg}\right) = (1,937.8)\left(\frac{7}{3} - (1)\right)$ = 2,583.7 ft-lbs/lf

Equation 5F-20: $A_s = \left(\frac{\left(\frac{M_b}{1,000}\right)}{1.76d_f}\right) = \left(\frac{\left(\frac{2,583.7}{1,000}\right)}{1.76(8.5)}\right)$ = 0.17 in.^{2}/lf

EXAMPLE C8. FLOODWALL DESIGN (concluded)

For top steel in the footing section:

Equation 5F-20 (note): $M_b = (w_{sh} + w_{wh})\left(\frac{A_h}{2}\right) = (921.6 + 1,497.6)\left(\frac{4}{2}\right)$ = 4,838.4 ft-lbs/lf

Equation 5F-20: $A_s = \left(\frac{\left(\frac{M_b}{1,000}\right)}{1.76d_f}\right) = \left(\frac{\left(\frac{4,838.4}{1,000}\right)}{1.76(8.5)}\right)$ = 0.32 in.^{2}/lf

For bottom steel in the footing section:

Equation 5F-20 (note): $= 673.3 - \left(\frac{1.5}{8}\right)(673.3 - 94.3)$ = 564.7 lb/in.^{2}

Equation 5F-20 (note): $M_b = (q + 2q_{max})\left(\frac{C^2}{6}\right) = [564.7 + 2(673.3)]\left(\frac{(2)^2}{6}\right)$ = 1,274.2 ft-lb/lf

Equation 5F-20: $A_s = \left(\frac{\left(\frac{M_b}{1,000}\right)}{1.76d_f}\right) = \left(\frac{\left(\frac{1,274.2}{1000}\right)}{1.76(8.5)}\right)$ = 0.09 in.^{2}/lf

From American Concrete Institute Reinforced Concrete Design Handbook Table 9a: **Use #4 bars on 14-inch centers in the vertical floodwall section, use #6 bars on 14-inch centers for the top steel in the footing section, and use #3 bars on 14-inch centers for the bottom steel in the footing section for B = 7.0 ft; Ah = 4.0 ft; and C = 2.0 ft.**

NOTE: Other ACI documents have similar information.