F. Example Calculations

Design a CMU pier and ground anchor foundation for a manufactured home to be placed in an SFHA Zone AE having a flood velocity of 2 fps. The BFE is 9 feet and existing ground elevation is approximately 7 feet. The flood depth is 2 feet and the freeboard is 1 foot, which yields a DFE depth of 3 feet. The manufactured home dimensions for these example calculations are shown in Figure F-1. The manufactured home is a single unit, 16 feet wide and 60 feet long with a 30-degree gable roof with a 1-foot overhang. Roofing members are spaced 16 inches on center (o.c.). The manufactured home weighs 20 psf. Assume an NFPA 5000 soil classification of soft, sandy clay, or clay (allowable bearing pressure q_a =1,000 psf ; ultimate bearing pressure q_u = 2,000 psf). Use ASCE 7 to calculate loads.

Foundation loads selected for this example of a manufactured home in an SFHA differ from those that may be found in HUD standard 24 CFR 3280. Design loads in this example are in accordance with ASCE 7-05 and other standards.

These example calculations assume transverse wind loads produce the controlling loading. Wind in the direction parallel to the roof ridge may produce greater loads for certain cases and must be evaluated during final design.

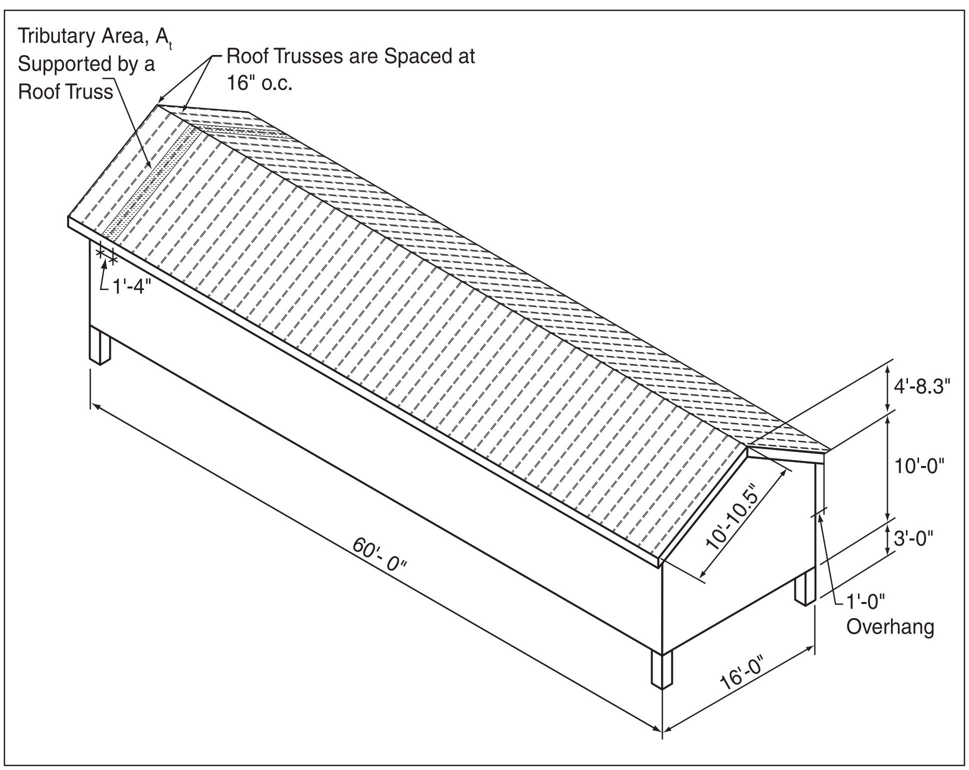


Figure F-1. Manufactured home dimensions.

ASCE 7-05

Step 1: Determine Design Criteria

ASCE 7-05
Table 4-1

NORMAL LOADS

Dead Load (D)

D = 20 psf Given in the example statement

Live Load (L)

L is based on one- and two-family dwellings

L = 40 psf

Roof Live Load (L_r)

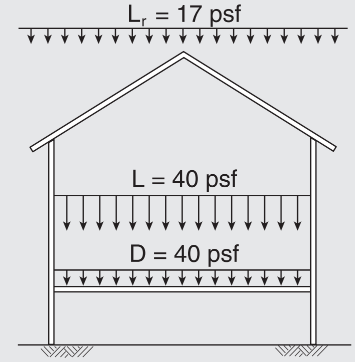
$$L_r = 20R_1R_2 = 20(1)(0.85) = 17 \text{ psf}$$

$$R_1 = 1 \text{ for } A_t \leq 200 \text{ ft}^2$$

$$A_t = 2(9.2\ ft)(16\ in)\left(\frac{1\ ft}{12\ in}\right) = 24.5\ ft^2$$

F = number of inches of rise per foot

$$F = 1ft\left(\frac{12\ in}{1\ ft}\right)\tan 30^\circ = 7\ in$$



Section View

Note that the roof live load falls between the limits given:

$$12 \leq L_r \leq 20$$

ASCE 7-05

ENVIRONMENTAL LOADS

Wind Loading

Section 6.2

Structure is a regular shape, located in a windborne debris region with terrain classification of Exposure C and surrounded by flat terrain.

Mean roof height (h)

h = 3 ft + 10 ft + 0.5(4 ft)

= 15 ft

h < 16 ft (least horizontal dimension)

Calculations are for a foundation system, which is a main wind force resisting system (MWFRS).

Velocity Pressures

Section 6.5

Velocity pressures are determined using

Method 2: Analytical Procedure

(A simplified alternative is to use ASCE 7, Section 6.4, Method 1. Wind pressures are tabulated for basic conditions. The wind pressure must be adjusted for mean roof height and exposure category.)

Velocity Pressure Coefficient (q_z)

Section 6.5.10
Eq. 6-15
Section 6.5.6
Table 6-3

$$q_z = 0.00256K_zK_{zt}K_dV^2I$$

Velocity pressure exposure coefficient evaluated at height *z* (the height above ground level in feet) (K_z)

Section 6.5.6

$K_z = 0.85$

Section 6.5.4.4
Section 6.5.5
Table 6-1
Section 6.5.4

Topographical factor (K_{zt})

$K_{zt}=1$ (assume a flat surface)

Wind directionality factor (K_d)

$K_d = 0.85$

Basic Wind Speed (V)

Section 6.5.11.1
Figure 6-5

V = 110 mph (3-second gust)

I = 1 (Category II building: Table 1-1 (ASCE 7))

Section 6.5.11.2.1
Figure 6-6

Therefore, $q_z = 0.00256(0.85)(1)(0.85)(110)^2(1)$

= 23 psf

Design Pressures for MWFRS

Internal Pressure Coefficient (GC_{pi})

$GC_{pi} = \pm 0.18$

External Pressure Coefficient (C_p)

h = Mean roof height, in feet

L = Horizontal dimension of building, in feet, measured parallel to wind direction

B = Horizontal dimension of building, in feet, measured normal to wind direction

Figure 6-6

Table F-1 shows the External Pressure Coefficients calculated for the windward, leeward and side walls. Computations of the External Pressure Coefficients for the windward and leeward roof are shown Table F-2.

Table F-1. External Wall Pressure Coefficients

Surface	Wind Direction	L/B	C_p
Windward Wall	n/a	n/a	0.8
Leeward Wall	Perpendicular to roof ridge	$\frac{16\,ft}{60\,ft} = 0.27$	-0.5
Side Wall	n/a	n/a	-0.7

Figure 6-6

Table F-2. External Roof Pressure Coefficients

Surface	Wind Direction	h/L	C_p
Windward Roof	Perpendicular to roof ridge	$\frac{15\,ft}{16\,ft} = 0.94$	-0.3 0.2
Leeward Roof			-0.6

Figure 6-6
Section 6.5.8
Eq. 6-17

Foundation systems are considered rigid, therefore, G = 0.85.

Design Pressure (p)

The basic pressure equation (ASCE 7 6-17), which includes the internal pressure coefficient is as follows:

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

However, this would only be used if designing individual components whose effective tributary area is equal to or greater than 700 sf (ASCE 7-05 6.5.12.1.3 and IBC 2006 1607.11.2.1). When determining loads on the global structure (i.e., shear walls or foundation design), the internal pressure components will act in equal and opposite directions on the roof/floor and the leeward/windward walls, thereby algebraically canceling each other. The resulting simplified form of the pressure equation is:

$$p = q \text{ x } GC_p$$

Table F-3 summarizes the design pressures calculated using this simplified wind design pressure equation. Figure F-2 shows the application of these design pressures on the structure. For foundation design, internal pressures need not be considered since internal pressure on windward walls, leeward walls, floors, and roofs cancel each other. For example, internal pressures acting on a windward wall are equal and opposite to those acting on a leeward wall and the net force on the foundation from internal pressures is zero.

While internal pressures cancel, internal pressures for a partially enclosed building have been included in the example. This is to provide an example of more general wind load calculations.

Table F-3. Design Pressures for Wind Perpendicular to the Roof Ridge

Surface	Design Wind Pressure Calculations	pressure (psf)
Windward Wall	p = 23 psf(0.85)(0.8)	15.7
Leeward Wall	p = 23 psf(0.85)(-0.5)	-9.8
Side Walls	p = 23 psf(0.85)(-0.7)	-13.7
Windward Roof	p = 23 psf(0.85)(-0.3)	-5.9
	p = 23 psf(0.85)(0.2)	4.0
Leeward Roof	p = 23 psf(0.85)(-0.6)	-11.8

MWFRS Roof Overhang Pressures

ASCE 7-05

ASCE 7 only addresses the windward overhang, specifying the use of a positive pressure coefficient of $C_p = 0.8$. Acting on the bottom surface of the overhang in combination with pressures acting on the top surface. For the leeward overhang, the coefficient for the leeward wall ($C_p = -0.5$) could be used, but the coefficient has been conservatively taken as zero.

Section 6.5.11.4.1

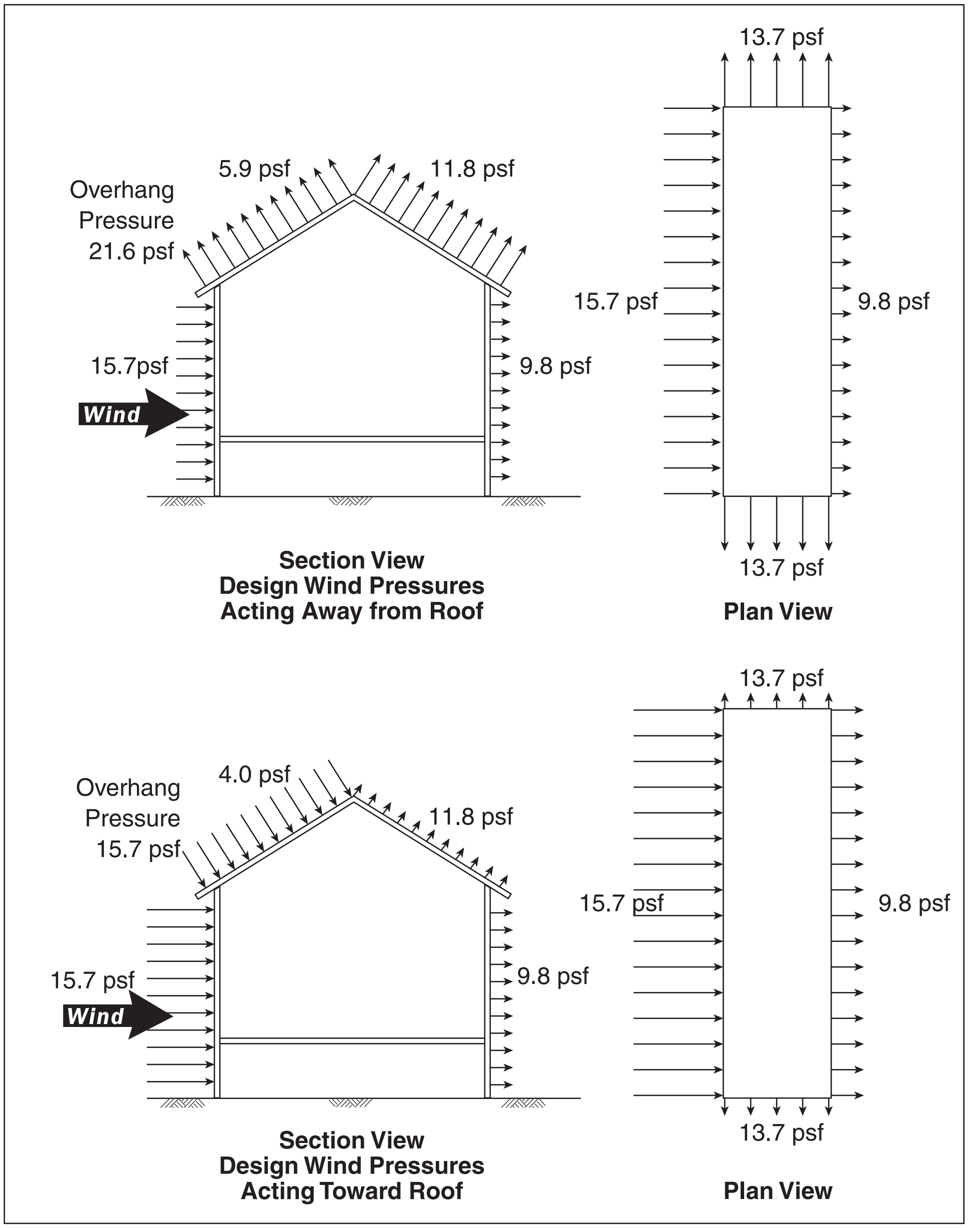
$$p = 23\ \text{psf}(0.85)\,(0.8)$$

$$= 15.7\ \text{psf}$$

$$p_{OH} = -5.9\ \text{psf} - 15.7\ \text{psf} = -21.6\ \text{psf}$$

A conservative simplification is to use the wind pressure acting away from the roof case for uplift roof pressure simultaneously with the wind pressure toward the roof for the lateral roof pressure.

Figure F-2. Maximum uplift and lateral wind loads on roof.



SNOW LOADING

Ground Snow Load (p_g)

p_g = 20 psf

Flat Roof Snow Load (p_f)

$p_f = 0.7C_eC_tIp_g$

$p_f = 0.7(1.0)(1.0)(1.0)(20)$

$= 14$ psf

But not less than $p_f = (I)p_g = 20$ psf

ASCE 7-05

Section 7.2
Figure 7-1

Section 7.3
Eq. 7-1

Exposure Coefficient (C_e)

Section 7.3.1
Table 7-2

C_e = 1.0 (partially exposed roof)

Thermal Factor (C_t)

Section 7.3.2
Table 7-3

C_t = 1.0

Importance Factor (I)

Section 7.3.3
Table 7-4

I = 1.0 (Category II building: Table 7-4 (ASCE 7))

Sloped Roof Snow Load (p_s)

Section 7.4

$p_s = C_s p_f$ (Eq. 7-2)
$= (1.0)(20 \text{ psf})$
$= 20 \text{ psf}$

Warm Roof Slope Factor (C_s)

Section 7.4.1
Figure 7-2

C_s = 1.0 (asphalt shingle not slippery)

Unbalanced Roof Snow Load (p_u)

Section 7.6

Since the roof's eave to ridge distance ≤ 20 ft, unbalanced uniform snow loads shall be applied as follows:

Section 7.6.1

$P_{windward} = 0.3\ p_s$
$= 6 \text{ psf}$

$P_{leeward.1} = p_s$
$= 20 \text{ psf}$

$P_{leeward.2} = (h_d)(\gamma)/\sqrt{(S)}$
$= (1.44 \text{ ft})(16.6 \text{ pcf})/\sqrt{(1.73)}$
$= 18.2 \text{ psf}$

From the ridge toward the leeward eave a distance of:

Figure 7-9
Eq. 7-3

$x = (8/3)(h_d)\sqrt{(S)}$
$= 5.1 \text{ ft}$

$h_d = 1.44 \text{ ft}$

$\gamma = 0.13\ p_g + 14 \leq 30 \text{ pcf}$
$= 16.6 \text{ pcf}$

FLOOD LOADING

Hydrostatic Load (F_h)

If the manufactured home is elevated above the BFE on an enclosed foundation, venting must be provided in all manufactured homes placed in a SFHA; the hydrostatic forces on either side of the foundation wall will cancel. However, the hydrostatic load is calculated because it is used in the hydrodynamic load calculation.

$$F_h = \tfrac{1}{2} P_h H$$

Hydrostatic Pressure (P_h)

$$P_h = \gamma H$$

Specific Weight of Fresh Water (ω)

$$\gamma = 62.4\ pcf$$

Floodproofing Design Depth (H)

H = 2 ft (base flood depth) + 1 ft

Hydrodynamic Load

The hydrodynamic load is calculated by converting it to an equivalent hydrostatic load by increasing the flood depth. The increase in flood depth is referred to as $d_{h.}$

$$d_h = \frac{C_d V^2}{2g} = \frac{2.0(2\,^{ft}\!/_s)^2}{2(32.2\,^{ft}\!/_s)} = 0.13\ ft$$

Drag Coefficient (C_d)

In the above equation, a value of 2.0 was assumed for C_d. This is a conservative estimate; the actual value for C_d could be anywhere between 1.2 and 2.0.

Acceleration Due to Gravity (g)

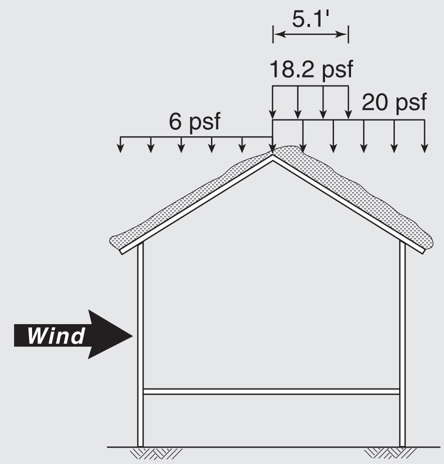
g = 32.2 ft/s^2

with a hydrodynamic pressure of

$$P_{hydr} = \gamma(d_h) = 62.4\ pcf\ (0.13\ ft) = 8.2\ psf$$

The equivalent hydrostatic load ($F_{h/ad}$) taken into consideration the hydrodynamic load is :

$$F_{h/ad} = P_{hydr} \times H = 8.2\ psf \times 3\ ft = 24.6\ plf$$



Section View

Note: A 1-foot freeboard is added to the BFE depth to provide a protection above the BFE; 3 feet becomes the "design" depth or the DFE.

ASCE 7

Eq. 7-5

FEMA's *Coastal Construction Manual* (FEMA 55) recommends a value of 2.0 for square or rectangular piles and 1.2 for round piles.

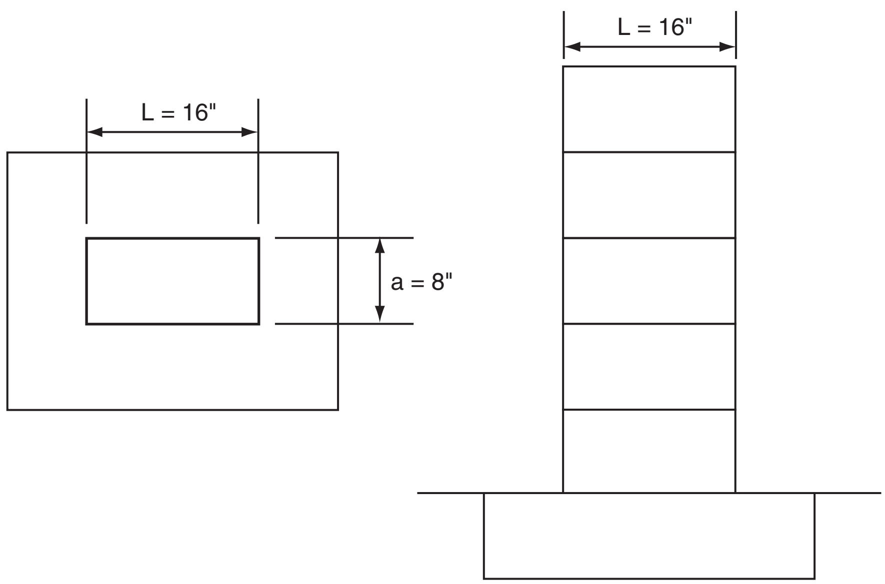
For additional guidance regarding drag coefficients, refer to Volume II of the U.S. Army Corps of Engineers' *Shore Protection Manual* (USACE 1984), FEMA 55, and the *Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures* (FEMA 259).

Since piers are 16 inches wide, the total hydrodynamic force on the pier is

$$= 24.6\,\frac{lb}{ft}\;\;16\ in\;\;\frac{1\ ft}{12\ in}\;\;32\ lbs \text{ per pier}$$

CHECK SCOUR

Reference: Publication No. FHWA NHI 00-001, *Evaluating Scour at Bridges,* 4th Edition, May 2001, Hydraulic Engineering Circular No. 18.



$$\frac{Y_s}{Y_1} = 2.0 \text{ x } (K_1) \text{x } (K_2) \text{ x}(K_3) \text{ x}(K_4) \text{ x } (a/Y_1)^{0.65} \text{ x } F_{r1}{}^{0.43}$$

Where: Y = Scour depth

Y_1 = Flow depth directly upstream of pier

a = Pier width (ft.)

L = Pier length (ft)

F_{r1} = Froude number

$F_{r1} = V_1/(gY_1)^{1/2}$

Where V_1 = Mean velocity of flow directly upstream of pier

g = acceleration due to gravity (32.2 feet/sec^2)

K_1 = Factor for pier nose shape. For square nose

K_2 = Factor for Angle of attack . $K_2 = (\text{cosine } \varphi + (L/a) \text{ x sine } \varphi)^{0.65}$

K_3 = Factor for bed condition/. $K_3 = 1.1$

K_4 = Factor for armoring by bed material size.

Project parameters:

Flood low = 2 fps

Flood depth = 3 ft

Assume flood angle of attack = 0º

So that:

$K_1 = 1.1$ (Table 6.1)

$K_2 = [\text{cosine } 0^o + (L/a) \text{x sine } 0^0]^{0.65} = [1.00 + (1.33'/0.67'') \text{ x } 0]^{0.65} = 1.00$

K_3 = Factor for bed condition/. $K_3 = 1.1$ (Table 6.3)

K_4 = Factor for armoring by bed material size. $K_4 = 1.0$ unarmored

$F_{r1} = V_1/(gY_1)^{1/2} = 2/[32.2 \text{ x } 3]^{1/2} = 2/9.84 = 0.203$

And

$$\frac{Y_s}{Y_1} = (2) \text{ x } (1.1) \text{ x } (1.0) \text{ x } (1.1) \text{ x } (1.0) \text{ x } (0.67/2)^{0.65} \text{ x } (0.203)^{0.43}$$

$$\frac{Y_s}{Y_1} = (2.42) \text{ x } (0.491) \text{ x } (.504) = 0.6$$

$$Y_s = (0.6) \text{ x } (Y_1) = (0.6)\text{x}(3) = 1.8 \text{ ft}$$

Scour protection or increased footing embedment required.

Step 2: Select a Design Methodology and Assess Load Combinations and Failure Modes

Figure F-3 illustrates the loads applied to the manufactured home. Table F-4 lists the nomenclature of the applied loads shown in Figure F-3.

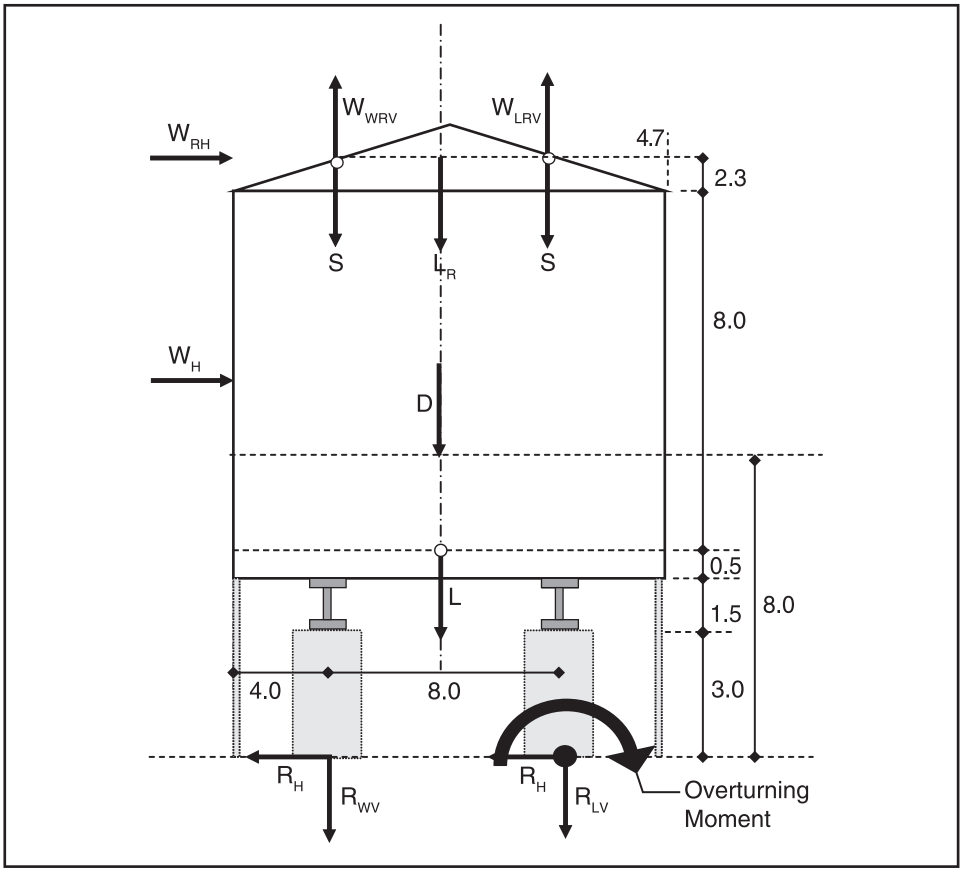


Figure F-3. Loading on a manufactured home.

Table F-4. Load Nomenclature

Nomenclature	Load Description
D	dead load
L	live load
L_R	roof live load
R_H	horizontal reaction
R_{LV}	leeward vertical reaction
R_{WV}	windward vertical reaction
S_B	balanced snow load
W_H	horizontal wall wind pressure
W_{RH}	roof horizontal wind pressure

Table F-4. Load Nomenclature (continued)

Nomenclature	Load Description
W_{LRV}	leeward roof vertical wind pressure
W_{WRV}	windward roof vertical wind pressure

Note that snow load governs over roof live load and wind downward load, and wind lateral load governs over earthquake lateral load. Load combinations for non-governing cases are not shown.

For the purposes of these calculations, the worst case wind load is taken to be perpendicular to the roof ridge for all failure modes. Wind in the direction parallel to the roof ridge may produce greater loads for certain failure modes.

Uplift and Downward Failure Mode

Uplift failure is a vertical force phenomenon. The loads that act vertically are wind, snow, dead, and live loads. Table F-5 summarizes the loads that influence uplift and downward failure mode. Table F-6 assesses uplift and downward failure load combinations. Note that uplift is based on MWFRS pressures for the global foundation design. Design of the connections to the foundation may require components and cladding (C&C) pressures to be used.

Table F-5. Vertical Load Values

Load Type	Total load acting on the structure and, therefore, must be supported by the foundation
D	D = [dead load per square foot][width of the manufactured home] D = [20 psf][16 ft] D = 320 lbs per linear ft of manufactured home length
L	L = [live load per square foot][width of the manufactured home] L = [40 psf][16 ft] L = 640 lbs per linear ft of manufactured home length
L_r	L_r = [roof live load per square foot][width of the manufactured home] L_r = [17 psf][18 ft] L_r = 306 lbs per linear ft of manufactured home length

Table F-5. Vertical Load Values (continued)

Load Type	Total load acting on the structure and, therefore, must be supported by the foundation
W	Maximum wind uplift loads occur for winds parallel to the roof ridge at the windward end. $W = W_{WRV}+W_{LRV}$ = [(vertical component roof wind pressures)(area roof)]/manufactured home length W = [-17.6 psf][(9 ft)(15 ft)(2)] 0 ft to 15 ft + [-9.8 psf][(9 ft)(15 ft)(2)] 15 ft to 30 ft + [-5.9 psf][(9 ft)(30 ft)(2)] 30 ft to 60 ft W = -10,584 lbs/60 ft = -176 lbs per linear foot of manufactured home length (average) In this case, vertical uplift loads are low and so this simplification is acceptable. However, to account for the unbalanced uplift if wind loads were higher, either overturning in this direction would need to be considered, or the windward uplifts conservatively made symmetrical about the middle. Maximum wind downward loads occur for wind perpendicular to the roof ridge; however, they are much less than, and do not govern over, roof live or snow loads.
S	S = [snow pressure][horizontal projected roof area] S = [20 psf][(9 ft)]SW + [20 psf][(9 ft)]SL S = 360 lbs per linear ft of manufactured home length

Table F-6. Vertical Failure Mode ASD Load Combinations

Load Combinations	
4	D + 0.75L + 0.75S 320 lbs + 0.75(640 lbs) + 0.75(360 lbs) = 1,070 lbs per linear ft of manufactured home length
7	0.6D + W 0.6(320 lbs) - 176 lbs = 16 lbs per linear ft of manufactured home length acting downward

Note that, for load combination 7, the 0.6 load factor should be applied to the dead load that would actually be present over the whole structure. Additions to the dead load tabulation such as mechanical and miscellaneous or shingles should not be included in this value as they may not be present in all areas or during a high-wind event and their inclusion would not be conservative.

Sliding or Shearing Failure Mode

Sliding failure is a lateral force phenomenon. The loads that act laterally are wind and flood loads. Table F-7 summarizes the lateral loads and their values. Maximum lateral wind loads occur when the wind is perpendicular to the roof ridge. Note that lateral wind loads act on the overall structure (i.e., foundation), whereas flood loads act on the individual piers. Table F-8 gives the load combinations for sliding failure. Once the number of piers is defined, the hydrodynamic forces on these piers are to be added to load combination 4, and the foundation design will have to be checked to make sure it can resist the added hydrodynamic loads.

Table F-7. Lateral Load Values

Load Type	Total load acting on the structure and, therefore, must be supported by the foundation
W	Maximum lateral wind loads occur for winds ⊥ to the roof ridge $W = W_{RH} + W_H =$ [lateral roof pressures][roof height] + [wall pressures][wall height] W = [4 psf – (-11.8 psf)] (4.7 ft) [(15.7 psf + 9.8 psf)(10 ft)] W = 329.3 lbs per linear ft of manufactured home length
F_a	Hydrodynamic load per pier F_a = [hydrodynamic force][pier length] F_a = 32.8 lbs per pier Assume total of 9 piers x 2 rows for 1st iteration F_a = (32.8 lbs per pier)(9 piers per row)(2 rows) F_a = 590.4 lbs / 60 ft = 9.84 lbs per linear ft of manufactured home length

Table F-8. Sliding Load Combinations

Load Combinations	
5	$W + 1.5F_a$ 329.3 lbs + 1.5(9.84 lbs) = 344 lbs per linear ft of manufactured home length

Note: The vertical gravity loads are not considered to be conservative. Thus, the frictional resistance of the footings under the piers has been neglected. This component may be used in borderline situations at the discretion of the engineer.

Overturning Failure Mode

Overturning failure results from loads that act on the whole structure and pivot about the bottom of the leeward pier. Dead, live, wind, and snow loads can all influence the overturning moment. Table F-9 summarizes the moments that affect overturning due to wind in this case. Table F-10 assesses the moment load combinations. Only the portions of the roof and floor live loads that are over the part that cantilevers out past the leeward pier will contribute to the overturning. Since this is the worst overturning case for each, only these conditions will be calculated.

Table F-9. Moment Load Values

Moment Type	Total moment about the bottom of the leeward foundation support (positive moment is counter clockwise)
D	D = [dead load per square foot][home width][moment arm] D = [20 psf][(16 ft)(4 ft)] D = +1,280 ft-lbs per linear ft of manufactured home length
L	L_1 = [live load per square foot][home width][moment arm] L_1 = [40 psf][(16 ft)(4 ft)] L_1 = 2,560 ft-lbs per linear ft of manufactured home length L_2 = [live load per square foot][cantilever width][moment arm] L_2 = [40psf][4 ft][-1 ft] L_2 = -160 ft lbs per linear ft of manufactured home length
L_r	L_r = [roof live load per square foot][roof width][moment arm] L_r = [17 psf][4 ft][1 ft] L_r = -68 ft-lbs per linear ft of manufactured home length
W	**WIND PERPENDICULAR TO THE ROOF RIDGE** W_{WRV} = [vertical component roof wind pressures][roof width][moment arm] W_{WRV} (-21.6 psf)(1 ft)(12.5 ft) + (-5.9 psf)(8 ft)(8 ft) W_{WRV} = -648 ft-lbs per linear ft of manufactured home length W_{LRV} = [vertical component roof wind pressures][roof width][moment arm] W_{LRV} (-11.8 psf)(9 ft)(-0.5 ft) W_{LRV} = +53 ft-lbs per linear ft of manufactured home length W_{RH} = [horizontal component roof wind pressures][roof height][moment arm] W_{RH} = [4 psf – (-11.8 psf)](-4.67 ft)](15.3 ft) W_{RH} = -1,129 ft-lbs per linear ft of manufactured home length W_{W+L} = [windward wall pressure + leeward wall pressure][home's height from ground to roof eave][moment arm] W_{W+L} = [15.7 psf + 9.8 psf](10 ft)(-8 ft) W_{W+L} = -2,040 ft-lbs per linear ft of manufactured home length
F_a	Hydrodynamic load on piers F_a = [horizontal component][moment arm] = (9.84 plf)(-3 ft/2) = -15 ft-lbs per linear foot of manufactured home length
S	S_B = [balanced snow pressure][horizontal projected roof area][moment arm] S_B = [20 psf][18 ft][-4 ft] S_B = 1,440 ft-lbs per linear ft of manufactured home length

Table F-10. Overturning Load Combinations

Moment Load Combinations (positive moment is counter clockwise)	
6	$D + 0.75W + 0.75L + 0.75L_r + 1.5F_a$ (Partial live loading to produce max OT) (1,280 ft-lbs) + (0.75)(-648 ft-lbs + 53 ft-lbs – 1,129 ft-lbs – 2,040 ft-lbs) + (0.75)(-160 ft-lbs)+ (0.75) (-68 ft-lbs) + (1.5)(-15 ft-lbs) = -1,737 ft-lbs per linear ft of manufactured home length $D + 0.75W + 0.75L + 0.75S + 1.5F_a$ (Full live and snow to produce max downward reaction) (1,280 ft-lbs) + (0.75)(-648 ft-lbs + 53 ft-lbs – 1,129 ft-lbs – 2,040 ft-lbs) + (0.75)(-2,560 ft-lbs) + (0.75)(1,440 ft-lbs) + (1.5)(-15 ft-lbs) = 1,435 ft-lbs per linear ft of manufactured home length
7	$0.6D + W + 1.5F_a$ (0.6)(1,280 ft-lbs) + (-648 ft-lbs + 53 ft-lbs – 1,129 ft-lbs – 2,040 ft-lbs) + (1.5)(-15 ft-lbs) = -3,019 ft-lbs per linear ft of manufactured home length

Table F-11 summarizes the load combinations that govern for each of the three failure modes. The maximum roof vertical and lateral load cases are assumed to act simultaneously as a conservative simplification.

Table F-11. ASD Load Combinations for Example Problem (loads are in pounds per linear foot)

Failure Modes	Load Combinations		
	4	5	7
Uplift	1,070 lbs	n/a[1]	15 lbs
Sliding	n/a[1]	313 lbs	n/a[1]
Overturning	n/a[1]	n/a[1]	-979 ft-lbs

[1] Load combination does not govern.

Load combinations 1-3 do not govern. Load combination 6 does not comply with HUD 24 CFR 3280.

Step 3: Select Foundation Type and Materials

The example statement specified a CMU pier and ground anchor foundation type. Since the flood velocity is 2 fps, CMU piers must have surface bonded mortar that meets ASTM C887-79a (2001) and ASTM C946-91 (2001) and maintain bonding between CMUs.

Step 4: Determine Forces at Connections and on Foundation Components

CMU piers transfer the compressive loads from the manufactured home into footings and then into the ground. The masonry piers are not considered to provide any lateral or uplift resistance. The governing load combination for downward forces is the vertical failure mode (load combination 4), which produces a total downward force from the manufactured home equal to

(downward force) (length of manufactured home)

Therefore, (1,070 lbs) (60 ft) = 64,200 lbs

This downward force governs the number of footings and, therefore, piers needed to transfer the downward load into the ground.

Following the braced masonry pre-engineered foundation design for flood velocities over 2 fps specification given in Chapter 10 of the use of a dry-stack 16-inch by 8-inch block pier with a minimum of an 1/4-inch thick surface bonded mortar and a 24-inch square, 10-inch deep footing, calculate the number of footings needed to adequately transfer the downward loads to the ground.

$$\text{Required footing area} = \frac{\textit{comprehensive load}}{\textit{allowable soil bearing capacity}}$$

Consult the geotechnical engineer for the ultimate soil bearing capacity value. An approximate method to calculate the ultimate soil bearing capacity is to multiply the allowable soil bearing capacity by a safety factor. The maximum pressure given in the NFPA 5000 Soil Classification Table can also be used as the ultimate soil bearing capacity.

$$\text{Required footing area} = \frac{64{,}200\ lbs}{1{,}000\ psf} = 64\ ft^2$$

$$\text{Individual footing area} = (24\text{ in x }24\text{ in})\left(\frac{1\ ft^2}{144\ in^2}\right) = 4\text{ ft}^2$$

$$\text{Number of footings} = \frac{\textit{total required footing area}}{\textit{individual footing area}}$$

$$= \frac{64\ ft^2}{4\ ft^2} = 16\text{ footings/piers}$$

Therefore, provide 8 piers per side of the manufactured home

$$\text{Pier spacing} = \frac{\textit{manufactured home length}}{\textit{(number piers per side-1)}}$$

$$= \frac{60\ ft}{8-1} = 8.6\text{ ft}$$

The maximum spacing of the piers is set to 8 feet to provide effective floodborne debris protection. To protect against floodborne debris, it is assumed that 1 pier will be lost due to floodborne debris.

Minimum number of piers = $\frac{60\ ft^2}{8\ ft}$ +1= 8.5 piers, say 9 piers per side (i.e., 8 spaces at 7.5 feet). Therefore, the home will be supported by a total of 18 piers (9 piers on each side) spaced at 7.5 feet.

Lateral wind loads are resisted by the strapping and ground anchors. The final number of piers equals the initial guess; therefore, the lateral load on the piers does not have to be updated.

Calculate the number of anchors needed to resist sliding failure.

The recommended design stiffness of the anchors in Table 7-5 in this guide is given for 5-foot anchors installed at 45 degrees and axially loaded is 1,200 lb/in (Figure 7-4). The horizontal component of the ground anchors strength is equal to

$$(1{,}200 \text{ lbs/in})(\cos 45^\circ) = 848 \text{ lb/in}$$

The manufactured home industry gives an allowable lateral manufactured home movement of 3 inches. So the total lateral strength of a ground anchor is (3 in) (848 lbs) = 2,544 lbs.

$$\text{Number of ground anchors needed} = \frac{\textit{lateral load}}{\textit{anchors lateral capacity}}$$

$$\text{Number of ground anchors needed} = \frac{(313\ lb)(60\ ft)}{2.544\ lbs} = 8 \text{ anchors per side}$$

$$\text{Calculate ground anchor spacing} = \frac{60\ ft}{(8\text{-}1)} = 8.5 \text{ ft}$$

The anchor strapping should attach into a wall stud; therefore, anchor spacing must be adjusted to 16-inch increments.

Both uplift and overturning failure modes are resisted by the vertical strength of ground anchors. The uplift forces will be resisted by all the ground anchors and the overturning moment will be resisted only by the windward ground anchors.

For the worst uplift of the vertical failure mode, load combination 7 (refer to Table F-6) governs. However, the maximum net uplift is 16 plf downward, which means that overturning will govern the uplift requirements.

For the overturning failure mode, load combination 7 (refer to Table F-10) governs for wind perpendicular to the roof ridge. Overturning moment is only resisted by the windward anchors. Therefore, the total vertical load each anchor will have to resist is

$$\frac{\dfrac{\textit{(overturning moment)(length of home)}}{\textit{moment arm}}}{\textit{number of anchors per side}} = \frac{\dfrac{(979\ ft\text{-}lbs)(60\ ft)}{12\ ft}}{8\ \textit{anchors}}$$

$$= 612 \text{ lbs per anchor}$$

The vertical component of the anchor stiffness equals

$$(1{,}200 \text{ lbs/in})(\cos 45) = 848 \text{ lbs/in}$$

The manufactured home industry gives an allowable vertical movement of 2 inches. This results in a vertical strength per anchor equal to

$$(2 \text{ in})(848 \text{ lbs/in}) = 1{,}697 \text{ lbs per anchor}$$

This is more than the strength needed by each anchor to resist the overturning moment.

The anchor strapping should attach into a wall stud and, therefore, anchor spacing must be adjusted to 16-inch increments. Place anchors at each end of the home and space at 72 inches.

For the overturning case, the connection of the straps to the stud and the ground anchor embedment is based on MWFRS pressures. However, although it would likely not govern, to be thorough, the uplift only condition using C&C pressures should be checked for these two anchorages (straps to studs and anchors in ground).

Step 5: Specify Connections and Framing Methods Along with Component Dimensions to Satisfy Load Conditions

The CMU pier and ground anchor foundation will consist of 16 dry-stack, 16-inch by 8-inch block piers with a minimum of a 1/4-inch thick surface bonded mortar and 24-inch square, 10-inch deep footings. Ground anchors will be placed at 45-degree angles at each end of the manufactured home and spaced at 72 inches.

Step 6: Note All Design Assumptions and Details on Drawings

Refer to pre-engineering foundation design drawings contained in Appendix H and specifications presented in Chapter 10 herein as to how to adequately document assumptions and detail drawings.