# D. Foundation Analysis and Design Examples

*Chapter 3 described the types of loads considered in this manual. This appendix demonstrates how these loads are calculated using a sample building and foundation. The reactions from the loads imposed on the example building are calculated, the loads on the foundation elements are determined, and the total loads are summed and applied to the foundation.*

There is a noteworthy difference between the approach taken for designing the foundations included in this manual and the analyses that a designer may undertake for an individual building. The analyses used for the designs in this manual were based on the "worst case" loading scenario for a "range of building sizes and weights." This approach was used to provide designers and contractors with some flexibility in selecting the home footprint and characteristics for which these pre-engineered foundations would apply. This also simplifies application of the pre-engineered foundations, reducing the number of pre-engineered foundations, and results in conservative designs.

For example, the designs presented were developed to resist uplift and overturning for a relatively light structure with a flat roof (worst case uplift and overturning) while gravity loads were based on a relatively heavy structure to simulate worst case gravity loads. Sliding forces were determined for a relatively deep home with a steep roof to simulate the largest lateral loads. The range of building weights and dimensions applicable to the designs are listed in Appendix C.

Since the designs are inherently conservative for some building dimensions and weights, a local design professional may be consulted to determine if reanalyzing to achieve a more efficient design is warranted. If a reanalysis is determined to be cost-effective, the sample calculations will aid the designer in completing that analysis.

## D.1 Sample Calculations

The sample calculations have been included to show one method of determining and calculating individual loads, and calculating load combinations.

### Type of Building

The sample calculation is based on a two-story home with a 28-foot by 42-foot footprint and a mean roof height of 26 feet above grade. The home is located on Little Bay in Harrison County, Mississippi, approximately 1.5 miles southwest of DeLisle (see Figure D-1). The site is located on the Harrison County Flood Recovery Map in an area with an Advisory Base Flood Elevation (ABFE) of 18 feet, located between the 1.5-foot wave contour and the 3-foot wave contour (Coastal A zone). The local grade elevation is 15 feet North American Vertical Datum (NAVD). The calculations assume short- and long-term erosion will occur and the ground elevation will drop 1 foot during the life of the structure. This places the home in a Coastal A zone with the flood elevation approximately 4 feet above the eroded exterior grade. Based on ASCE 7-05, the 3-second gust design wind speed at the site is 128 miles per hour (mph) Exposure Category C. To reduce possible damage and for greater resistance to high winds, the home is being designed for a 140-mph design wind speed.

The home has a gabled roof with a 3:12 roof pitch. The home is wood-framed, contains no brick or stone veneer, and has an asphalt shingled roof. It has a center wood beam supporting the first floor and a center load bearing wall supporting the second floor. Clear span trusses frame the roof and are designed to provide a 2-foot overhang. No vertical load path continuity is assumed to exist in the center supports, but vertical and lateral load path continuity is assumed to exist elsewhere in the structure.

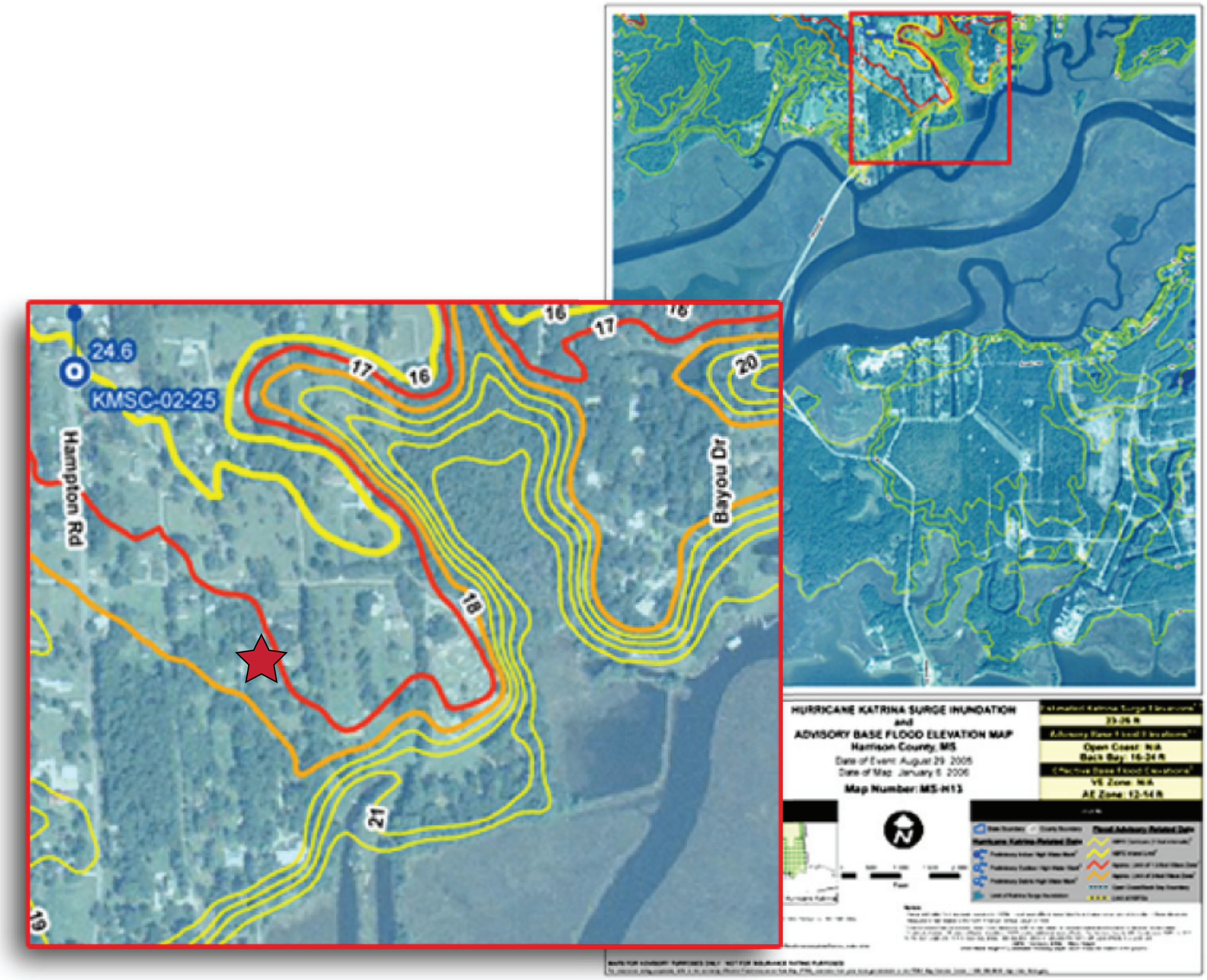



Figure D-1.
Star indicates the location of the sample calculation home on Little Bay in Harrison County, Mississippi, approximately 1.5 miles southwest of DeLisle. Inset on the left (from the map) is enlarged.

The proposed foundation for the home is a system of steel pipe piles, a reinforced concrete grade beam, and concrete columns extending from the grade beam to the elevated structure.

**Methodology**

1. Determine the loads based on the building's parameters (Section D.1.1)

2. Calculate wind and flood loads using ASCE 7-05 (Section D.1.2)

3. Consider the structure as a rigid body, and use force and moment equilibrium equations to determine reactions at the perimeter foundation elements (Section D.2)

## D.1.1 Determining Individual Loads on a Structure

**Building Dimensions and Weights** (pulled from the text of the example problem)

| | | |
|---|---|---|
| B | = 42 | Building width (ft) |
| L | = 28 | Building depth (ft) |
| $F_1$ | = 10 | First floor height (ft) |
| $F_2$ | = 10 | Second floor height (ft) |
| r | = 3 | 3:12 roof pitch |
| $W_{ovhg}$ | = 2 | Width of roof overhang (ft) |

**Dead Loads**

| | | |
|---|---|---|
| $W_{rfDL}$ | = 12 | Roof dead load including upper level ceiling finish (in pounds per square foot [psf]) |
| $W_{1stDL}$ | = 8 | First floor dead load (psf) |
| $W_{2ndDL}$ | = 10 | Second floor dead load, including first floor ceiling finish (psf) |
| $W_{wlDL}$ | = 9 | Exterior wall weight (psf of wall area) |

**Live Loads**

| | | |
|---|---|---|
| $W_{1stLL}$ | = 40 | First floor live load (psf) |
| $W_{2ndLL}$ | = 30 | Second floor live load (psf) |
| $W_{rfLL}$ | = 20 | Roof live load (psf) |

**Wind Loads**

Building Geometry

| | |
|---|---|
| h/L | = 1 |
| L/B | = 0.7 |

### Site Parameters (ASCE 7-05, Chapter 6)

| | | |
|---|---|---|
| $K_{zt}$ | = 1 | Topographic factor (no isolated hills, ridges, or escarpments) |
| $K_d$ | = 0.85 | Directionality factor (for use with ASCE 7-05, Chapter 2, Load Combinations) |
| $K_h$ | = 0.94 | For simplicity, Velocity Pressure Coefficient used at the 26 foot mean roof height was applied at all building surfaces. See ASCE 7-05, Table 6-3, Cases 1 and 2. |
| I | = 1 | Importance factor (residential occupancy) |
| V | = 140 | 3-second gust design wind speed (mph) |
| G | = 0.85 | Gust effect factor (rigid structure, ASCE 7-05, Section 6.5.8.1) |

### External Pressure Coefficients ($C_p$) (ASCE 7-05, Figure 6-6)

| | | |
|---|---|---|
| $C_{pwwrf}$ | = -1.0 | Windward roof (side facing the wind) |
| $C_{plwrf}$ | = -0.6 | Leeward roof |
| $C_{pwwl}$ | = 0.8 | Windward wall |
| $C_{plwwl}$ | = -0.5 | Leeward wall (side away, or sheltered, from the wind) |
| $C_{peave}$ | = -0.8 | Windward eave |

Positive coefficients indicate pressures acting on the surface. Negative coefficients indicate pressures acting away from the surface.

### Velocity Pressure ($q_h$) (ASCE 7-05, Section 6.5.10)

Velocity pressure at mean roof height:

$$q_h = 0.00256\ K_h K_{zt}\ K_d\ V^2\ I$$

$$= 0.00256\ (0.94)\ (1)\ (0.85)\ (140)^2 (1)$$

$$q_h = 40 \text{ psf}$$

### Wind Pressures (P)

Determine external pressure coefficients for the various building surfaces. Internal pressures, which act on all internal surfaces, do not contribute to the foundation reactions. For sign convention, positive pressures act inward on a building surface and negative pressures act outward.

$P_{wwrfV}$ $= q_h \, GC_{pwwrf}$ Windward roof

$= (40)(0.85)(-1)$

$P_{wwrfV}$ $= -34$ psf

Likewise

$P_{lwrfV}$ $= q_h \, GC_{plwrf}$ Leeward roof

$P_{lwrfV}$ $= -20$ psf

$P_{wwwl}$ $= q_h \, GC_{pwwwl}$ Windward wall

$P_{wwwl}$ $= 27$ psf

$P_{lwwl}$ $= q_h \, GC_{plwwl}$ Leeward wall

$P_{lwwl}$ $= -17$ psf

$P_{wweave}$ $= q_h \, GC_{peave}$ Windward roof overhang

$P_{wweave}$ $= -27$ psf - eave

$= -34$ psf - upper surface

$= -61$ psf - total

### Wind Forces (F) (on a 1-foot wide section of the home)

$F_{wwrfV}$ $= P_{wwrfV} \; L/2$ Windward roof vertical force

$= (-34 \text{ psf})(14 \text{ sf/lf})$

$= -476$ lb/lf

$F_{lwrfV}$ $= P_{lwrfV} \; L/2$ Leeward roof vertical force

$= (-20 \text{ psf})(14 \text{ sf/lf})$

$= -280$ lb/lf

$F_{wwrfH}$ $= P_{wwrfH} \; L/2 \; (r/12)$ Windward roof horizontal force

$= (-34 \text{ psf})(14 \text{ sf/lf})(3/12)$

$= -119$ lb/lf

$F_{lwrfH}$ $= P_{lwrfH} \; L/2 \; (r/12)$ Leeward roof horizontal force

$= (-20 \text{ psf})(14 \text{ sf/lf})(3/12)$

$= -70$ lb/lf

$F_{wwwl1st}$ $= P_{wwwl}$ $(F_1)$ Windward wall on first floor

$= (27 \text{ psf}) (10 \text{ sf/lf})$

$= 270 \text{ lb/lf}$

$F_{wwwl2nd}$ $= P_{wwwl}$ $(F_2)$ Windward wall on second floor

$= (27 \text{ psf}) (10 \text{ sf/lf})$

$= 270 \text{ lb/lf}$

$F_{lwwl1st}$ $= P_{lwwl}$ $(F_1)$ Leeward wall on first floor

$= (-17 \text{ psf}) (10 \text{ sf/lf})$

$= -170 \text{ lb/lf}$

$F_{lwwl2nd}$ $= P_{lwwl}$ $(F_2)$ Leeward wall on second floor

$= (-17 \text{ psf}) (10 \text{ sf/lf})$

$= -170 \text{ lb/lf}$

$F_{wweave}$ $= P_{wweave} w_{owhg}$ Eave vertical force (lb)

$= -(61 \text{ psf}) (2 \text{ sf/lf})$ *horizontal projected areas is negligible so horizontal force is neglected*

$= -122 \text{ lb/lf}$

## D.1.2 Calculating Reactions from Wind, and Live and Dead Loads

Sum overturning moments ($M_{wind}$) about the leeward corner of the home. For sign convention, consider overturning moments as negative. Since vertical load path continuity is assumed not to be present above the center support, the center support provides no resistance to overturning (see Figure D-2).

$M_{wind}$ $= (-476 \text{ lb/lf}) (21 \text{ ft}) + (-280 \text{ lb/lf}) (7 \text{ ft}) + (119 \text{ lb/lf}) (21.75 \text{ ft}) + (-70 \text{ lb/lf}) (21.75 \text{ ft}) + (-270 \text{ lb/lf}) (5 \text{ ft}) + (-270 \text{ lb/lf}) (15 \text{ ft}) + (-170 \text{ lb/lf}) (5 \text{ ft}) + (-170 \text{ lb/lf}) (15 \text{ ft}) + (-122 \text{ lb/lf}) (29 \text{ ft})$

$= -23{,}288 \text{ ft-lb/lf}$

Solving for the windward reaction, therefore:

$W_{wind}$ $= M_{wind} \div L$

$W_{wind}$ $= -23{,}228 \text{ ft } (\text{lb/lf} \div 28 \text{ ft})$

$W_{wind}$ $= -830 \text{ lb/lf}$

The leeward reaction is calculated by either summing vertical loads or by summing moments about the windward foundation wall. Leeward reaction equals -48 lb/lf.

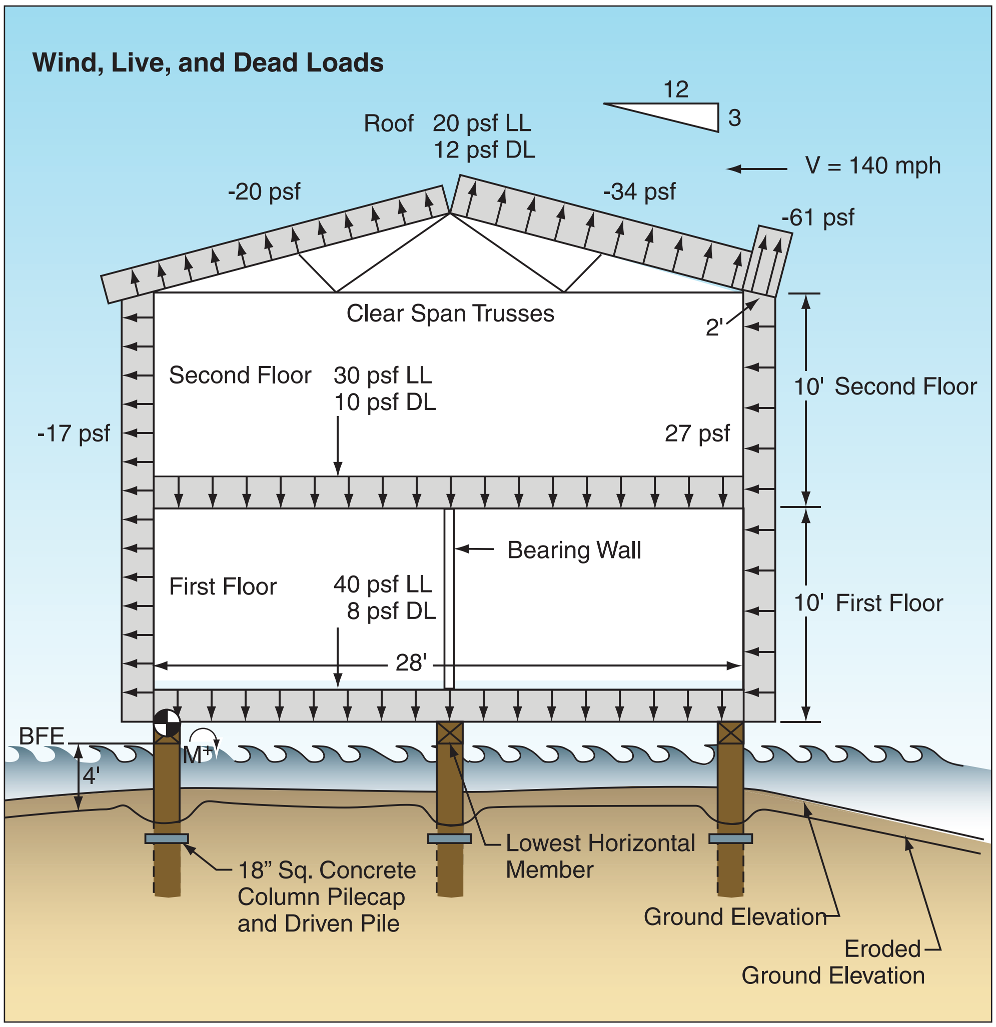
## Lateral Wind Loads

Sum horizontal loads (Flat) on the elevated structure. (Forces to the left are positive. See Figure D-2.)

$F_{lat}$ = (-119 lb/lf) + (70 lb/lf) + (270 lb/lf) + (270 lb/lf) + (170 lb/lf) + (170 lb/lf)

= 831 lb/lf

**Figure D-2. Paths for wind, live, and dead loads.**




## Dead Loads

In this example, dead load reactions ($W_{dead}$) are determined by summing loads over the tributary areas. Since the roof is framed with clear-span trusses and there is a center support in the home, each exterior foundation wall supports ½ of the roof load, all of the exterior wall load, and ¼ of the first and second floor loads. This approach to analysis is somewhat conservative since it does not consider the entire dead load of the structure to resist overturning. Standard engineering practice often considers the entire weight of the structure (i.e., not just the portion supported by the perimeter foundation walls) available to resist overturning. The closed foundations in this guidance were developed considering only the tributary dead load to resist uplift. The open foundations were developed considering all dead loads to resist uplift.

$$
\begin{aligned}
W_{dead} &= L/2\ (w_{rfDL}) + L/4\ (w_{1stDL} + w_{2ndDL}) + (F_1 + F_2)\ w_{wlDL} \\
&= [14\ \text{sf/lf}\ (12\ \text{psf})] + [7\ \text{sf/lf}\ (8\ \text{psf} + 10\ \text{psf})] + [(10\ \text{sf/lf} + 10\ \text{sf/lf})\ 9\ \text{psf}] \\
&= 474\ \text{lb/lf}
\end{aligned}
$$

### Live Loads

**Floor**

Live loads ($W_{live}$) are calculated in a similar fashion

$$
\begin{aligned}
W_{live} &= L/4(W_{1stLL} + W_{2ndLL}) \\
&= (7\ \text{sq ft/lf})\ (40 + 30)\ \text{psf} \\
&= 490\ \text{lb/lf}
\end{aligned}
$$

**Roof**

$$
\begin{aligned}
W_{liveroof} &= L/2(W_{rfLL}) \\
&= (14\ \text{sq ft/lf})\ (20\ \text{psf}) \\
&= 280\ \text{lb/lf}
\end{aligned}
$$

## D.2 Determining Load Combinations

Combine loads as specified in Chapter 2 of ASCE 7-05. For this example, an allowable stress design approach was used. A strength-based design is equally valid.

Other loads (such as snow, ice, and seismic) are listed in the ASCE 7-05 Load Combinations, but were considered to be too rare in the Gulf Coast of the United States to be considered in the design. ASCE 7-05 also lists rain loads that are appropriate for the Gulf Coast region. Since a minimum roof slope ratio of 3:12 was assumed for the homes, rain loading was not considered. Table D-1 lists foundation wall reactions for each load case. Critical reactions that contain the foundation design are highlighted.

Table D-1. Design Reactions on Base of Elevated Home

| ASCE 7-05 Load Combination | Vertical (lb/lf) | Horizontal (lb/lf) |
|---|---|---|
| #1 D | 474 | -- |
| #2 D + L | 964 | -- |
| #3 D + $L_r$ | 754 | -- |
| #4 D + 0.75(L) + 0.75($L_r$) | **{1,052}** | -- |
| #5 D + W | -356 | 831 |
| #6 D + 0.75(W) + 0.75(L) + 0.75($L_r$ ) | 1,016 | 623 |
| #7 0.6D + W | **{-546}** | **{831}** |
| #8 0.6D | 284 | -- |

Where

D = dead load

L = live load

$L_r$ = roof live load

W = wind load

**Note:** Critical loads are in bold with brackets (**{ }**).

## Flood Load Effects on a Foundation

In this example, since the foundation selected is a system of steel pipe piles, the equations used to calculate flood loads are based on open foundations. Some of the equations used to calculate flood loads will be different if the building has a closed foundation system.

Many flood calculations depend on the stillwater flooding depth ($d_s$). While not listed on FIRMs, $d_s$ can be calculated from the BFE by knowing that the breaking wave height ($H_b$) equals 78 percent of the stillwater depth and that 70 percent of the breaking wave exists above the stillwater depth (see Figure 11-3 of FEMA 55). Stated algebraically:

BFE $= GS + d_s + 0.70\ H_b$

$= GS + d_s + 0.70(0.78\ d_s)$

$= GS + 1.55\ d_s$

GS = 15 ft NAVD (initial elevation) – 1 ft (short- and long- term erosion)

= 14 ft NAVD

$d_s$ = (BFE – GS) ÷ 1.55

= (18 feet NAVD – 14 feet NAVD)÷ 1.55

= 4 ft ÷1.55

= 2.6 ft

## Hydrostatic Loads

Hydrostatic loads act laterally and vertically on all submerged foundation elements. On open foundations, lateral hydrostatic loads cancel and do not need to be considered but vertical hydrodynamic forces (buoyancy) remain. The buoyancy forces reduce the effective weight of the foundation by the weight of the displaced water and must be considered in uplift calculations. For example, normal weight concrete which typically weighs 150 lb/ft$^3$ only weighs 86 lb/ft$^3$ when submerged in saltwater (slightly more in freshwater).

In this example, calculations are based on an 18-inch square normal weight concrete column that extends 4 feet above eroded ground elevation. The column weighs 1,350 pounds dry ((1.5 ft) (1.5 ft) (4 ft) (150 lb/ft$^3$)). Under flood conditions, the column displaces 9 ft$^3$ of saltwater that, at 64 lb/ft$^3$, weighs 576 pounds so the column weighs only 774 pounds when submerged.

## Hydrodynamic Loads

### Flood Velocity

Since a Coastal A zone is close to the flood source, flood velocity is calculated using the ASCE 7-05 Equation C5-2:

$V = (g\,d_s)^{1/2}$ Upper bound flood velocity

**Where**

$g$ = Gravitational acceleration (32.2 ft/sec$^2$)

$d_s$ = Design stillwater depth (ft)

**Hence**

$V = [(32.2)(2.6)]^{1/2}$

= 9.15 feet per second (fps)

## Hydrodynamic Forces

A modified version of ASCE 7-05 Equation C5-4 can be used to calculate the hydrodynamic force on a foundation element as

$$F_{dyn} = \tfrac{1}{2} C_d \rho V^2 A$$

Where

$F_{dyn}$ = Hydrodynamic force (lb) acting on the submerged element

$C_d = 2.0$ Drag coefficient (equals 2.0 for a square or rectangular column)

$\rho = 2$ Mass density of salt water (slugs/cubic foot)

$A = 1.5\ d_s$ Surface area of obstruction normal to flow ($ft^2$)

*For open foundation, "A" is the area of pier or column perpendicular to flood direction (calculated for an 18-inch square column).*

Hence

$F_{dyn} =$ (½) (2)(2)(9.15)2(1.5)(2.6)

= 653 lb/column

The force is assumed to act at a point $d_s/2$ above the eroded ground surface.

The formula can also be used for loads on foundation walls. The drag coefficient, however, is different. For foundation walls, $C_d$ is a function of the ratio between foundation width and foundation height or the ratio between foundation width and stillwater depth. For a building with dimensions equal to those used in this example, $C_d$ for a closed foundation would equal 1.25 for full submersion (42 feet by 4 feet) or 1.3 if submersed only up to the 2.6-foot stillwater depth.

Dynamic loads for submersion to the stillwater depth for a closed foundation are as follows:

$F_{dyn} = \frac{1}{2} C_d \rho V^2 A$

$F_{dyn} = (\frac{1}{2})(1.3)(2)(9.15)^2(1)(2.6)$

= 283 lb/lf of wall

### Floodborne Debris Impacts

In this example, the loads imposed by floodborne debris were approximated using formula 11-9 contained in FEMA 55.

The *Commentary* of ASCE 7-05 contains a more sophisticated approach for determining debris impact loading, which takes into account the natural period of the impacted structure, local debris sources, upstream obstructions that can reduce the velocity of the floodborne debris, etc.

It is suggested that designers of coastal foundations review the later standards to determine if they are more appropriate to use in their particular design.

The FEMA 55 Formula 11.9 estimates debris impact loads as follows:

$F_i = wV \div (g\ \Delta\ t)$

Where

$F_i$ = impact force (lb)

w = weight of the floodborne debris (lb)

V = velocity of floodborne debris (ft/sec)

g = gravitational constant = 32.2 ft/sec$^2$

$\Delta t$ = impact duration (sec)

Floodborne debris velocity is assumed to equal the velocity of the moving floodwaters and acting at the stillwater level. For debris weight, FEMA 55 recommends using 1,000 pounds when no other data are available. The impact duration depends on the relative stiffness of the foundation and FEMA 55 contains suggested impact durations for wood foundations, steel foundations, and reinforced concrete foundations. For this example, the suggested impact duration of 0.1 second was used for the reinforced concrete column foundation.

$F_i = wV \div g\Delta t$

$F_i = [1{,}000 \text{ lb } (9.15 \text{ ft/sec})] \div [(32.2 \text{ ft/sec}^2)(0.1 \text{ sec})]$

$F_i = 2{,}842 \text{ lb}$

### Breaking Wave Loads

When water is exposed to even moderate winds, waves can build quickly. When adequate wind speed and upstream fetch exist, floodwaters can sustain wave heights equal to 78 percent of their stillwater depths. Depending on wind speeds, maximum wave height for the stillwater depth at the site can be reached with as little as 1 to 2 miles of upwind fetch.

Breaking wave forces were calculated in this example using ASCE 7-05 formulae for wave forces on continuous foundation walls (ASCE 7-05 Equations 5-5 and 5-6) and on vertical piles and columns (ASCE 7-05 Equation 5-4).

The equation for vertical piles and columns from ASCE 7-05 is

$F_{brkp} = \frac{1}{2} C_{db} \gamma D H_b^2$

Where

$F_{brkp}$ = Breaking wave force (acting at the stillwater level) (lb)

$C_{db}$ = Drag coefficient (equals 2.25 for square or rectangular piles/columns)

$\gamma$ = Specific weight of water (64 lb/ft$^3$ for saltwater)

D = Pile or column diameter in ft for circular sections, or for a square pile or column, 1.4 times the width of the pile or column (ft). For this example, since the column is 18 inches square, D = (1.4)(1.5 ft) = 2.1 ft.

$H_b$ = Breaking wave height $(0.78\ d_s)$ (ft) = (0.78) (2.6) = 2.03 ft

**Note:** The critical angle of a breaking wave occurs when the wave travels in a direction perpendicular to the surface of the column. Waves traveling at an oblique angle ($\alpha$) to the surface of the waves are attenuated by the factor $\sin^2\alpha$.

$$F_{brkp} = \tfrac{1}{2}\, C_{db}\, \gamma\, D H_b^2$$

$$= \tfrac{1}{2}\,(2.25)\,(64\ \text{lb/ft}^3)\,(2.1\ \text{ft})\,(2.03\ \text{ft})^2$$

$$= 623\ \text{lb}$$

For closed foundations, use equations in Section 5.4.4.2 of ASCE 7-05 to calculate $F_{brkp}$. FEMA 55 contains the following two equations for calculating loads on closed foundations:

$$F_{brkw} = 1.1\, C_p\, \rho\, d_s^2 + 2.4\, \gamma\, d_s^2 \quad \text{Case 1}$$

and

$$F_{brkw} = 1.1\, C_p\, \rho\, d_s^2 + 1.9\, \gamma\, d_s^2 \quad \text{Case 2}$$

Where $\gamma$ and $d_s$ are the specific weights of water and design stillwater depths as before. $C_p$ is the dynamic pressure coefficient that depends on the type of structure. $C_p$ equals 2.8 for residential structures, 3.2 for critical and essential facilities, and 1.6 for accessory structures where there is a low probability of injury to human life. The term $F_{brkw}$ is a distributed line load and equals the breaking wave load per foot of wall length where$F_{brkw}$ is assumed to act at the stillwater elevation.

Case 1 of Formula 11.6 represents a condition where floodwaters are not present on the interior of the wall being designed or analyzed. Case 1 is appropriate for foundation walls that lack flood vents (see Figure D-3). The less stringent Case 2 is appropriate for walls where NFIP required flood vents are in place to equalize hydrostatic loads and reduce forces (see Figure D-4).

In non-Coastal A Zones, the maximum wave height is 1.5 feet. This corresponds to a stillwater depth ($d_s$) of approximately 2 feet (i.e., 1.5 foot/0.78 for a depth limited wave). For closed foundations in coastal areas with flood vents, a 1.5-foot breaking wave creates 1,280 lbs per linear foot of wall and 1,400 lbs per linear foot of wall on foundations that lack flood vents.

### Wind Load on Columns

Wind loads have been calculated per ASCE 7-05, Section 6.5.13 (Wind Loads on Open Buildings and with Monoslope, Pitched, or Troughed Roofs). The velocity pressure ($q_h$) calculated previously was used, although this is a conservative figure based on the 26-foot mean roof height. The force coefficient ($C_f$) was determined from ASCE 7-05, Figure 6-21 (chimneys, tanks, rooftop equipment, and similar structures); ASCE 7-05, Figure 6-20 (walls and solid signs) could have been used as well.

From ASCE 7-05 Equation 6-28:

$F_{wind} = q_z \, G \, C_f \, A_f$

$= 40 \text{ psf } (0.85)(1.33^*)(1.5 \text{ ft})(4 \text{ ft})$

$= 270 \text{ lb}$

*Interpolated $C_f$

Wind loads on the foundation elements are not considered in combination with flood loads since the elements are submerged during those events.

### Flood Load Combinations

Section 11.6.12 of FEMA 55 provides guidance on combining flood loads. In Coastal A zones, FEMA 55 suggests two scenarios for combining flood loads. Case 1 involves analyzing breaking wave loads on all vertical supports and impact loading on one corner or critical support and Case 2 involves analyzing breaking wave loads applied to the front row of supports (row closest to the flood source), and hydrodynamic loads applied to all other supports and impact loads on one corner or critical support.

Depending on the relative values for dynamic and breaking waves, Case 1 often controls for designing individual piers or columns within a home. Case 2 typically controls for the design of the assemblage of piers or columns working together to support a home. Because of the magnitude of the load, it is not always practical to design for impact loads. As an alternative, structural redundancy can be provided in the elevated home to allow one pier or column to be damaged by floodborne debris impact without causing collapse or excessive deflection.

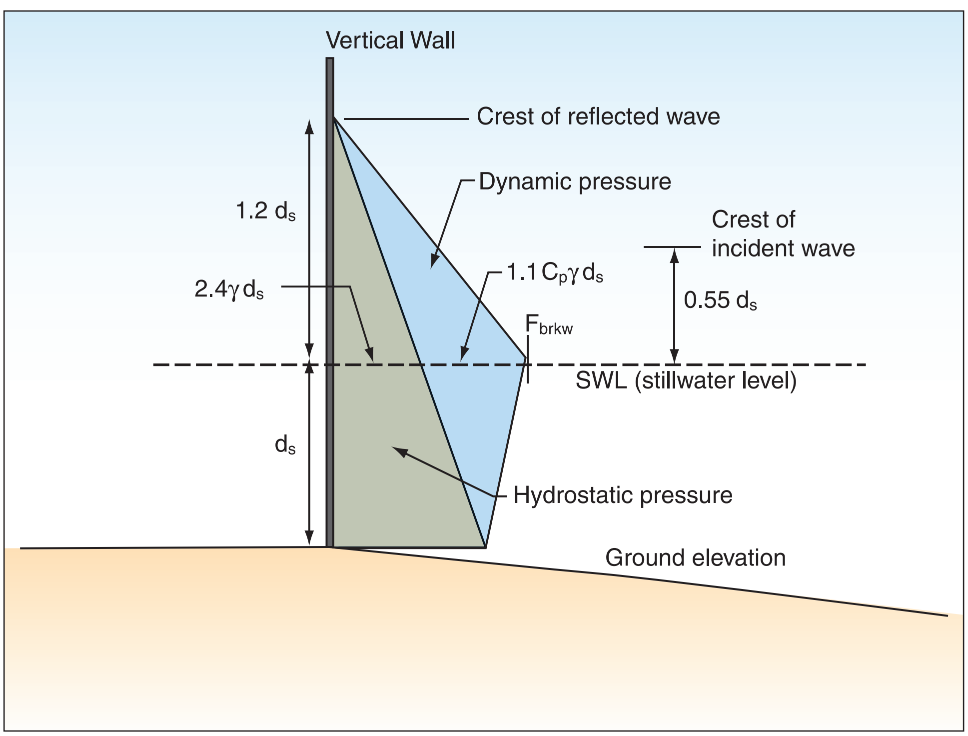
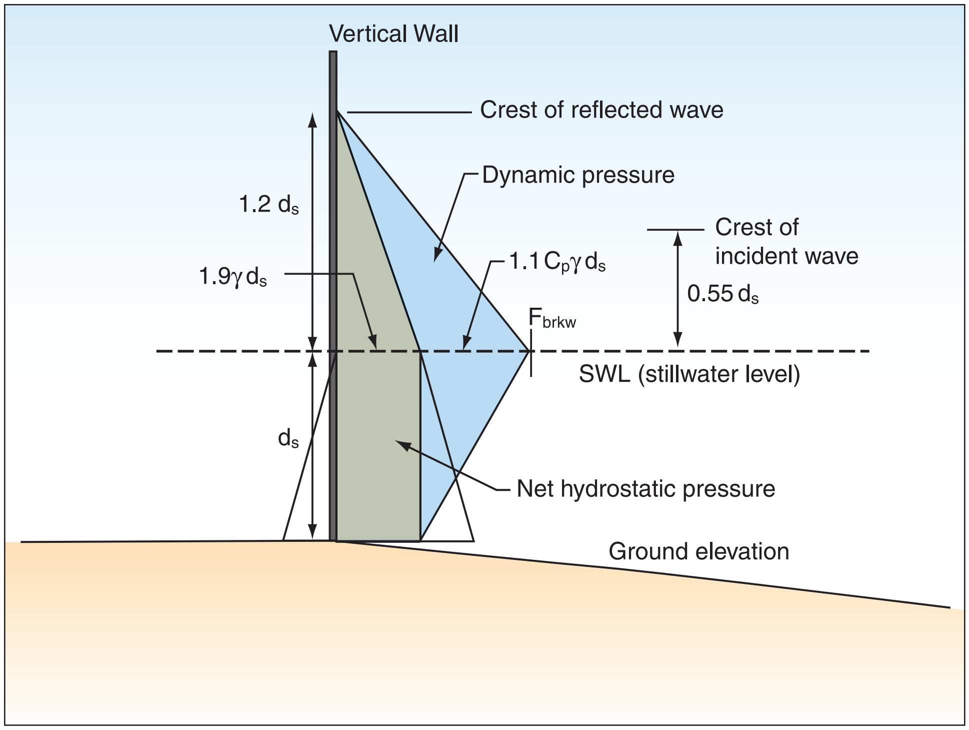



**Figure D-3. Case 1. Normally incident breaking wave pressures against a vertical wall (space behind vertical wall is dry).**

SOURCE: ASCE 7-05

**Figure D-4. Case 2. Normally incident breaking wave pressures against a vertical wall (stillwater level equal on both sides of wall).**

SOURCE: ASCE 7-05




For the sample calculations, Case 1 was used (see Figure D-5) with a breaking wave load of 623 pounds applied to a non-critical column. The loads were then determined and summarized. Since the calculations must combine distributed loads on the elevated structure and discrete loads on the columns themselves, a column spacing of 7 feet is assumed in the calculations. For lateral loads on the structure, calculations are based on three rows of columns sharing lateral loads (Table D-2).

**Figure D-5. Flood loads.**

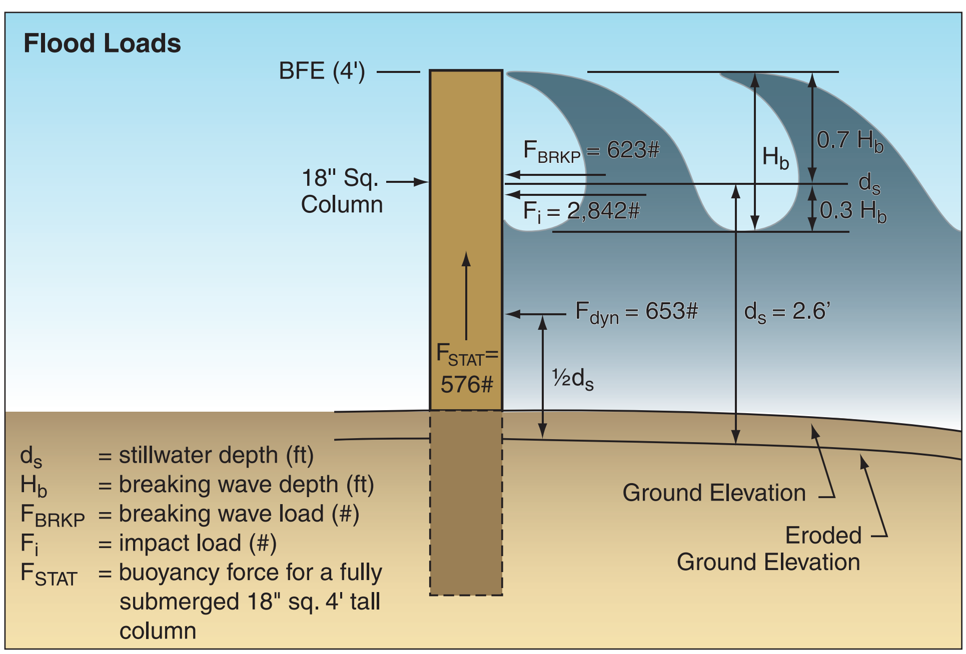


Table D-2. Loads on Columns Spaced 7 Feet On Center (for three rows of columns)

| ASCE 7-05 Load Combination | Vertical (lb) | Horizontal (lb) |
|---|---|---|
| #1 D + F | 1,350 + [7(474)] = 4,668 | -- |
| #2 D + F + L | 1,350 + [7(964)] = 8,098 | -- |
| #3 D + F + $L_r$ | 1,350 + [7(754)] = 6,628 | -- |
| #4 D + F + 0.75(L) + 0.75($L_r$) | 774 + 7(1,052) = **{8,138}** | -- |
| #5 D + F + W + 1.5($F_a$) | 774 + [7(474 - 48)] = 3,756 | [(1/3)7(831)] + [1.5(623)] = **{2,874}** |
| #6 D + F + 0.75(W) + 0.75(L) + 0.75($L_r$) + 1.5($F_a$) | 774 + 7[474 + 0.75 (-48+490+280)] = 7,883 | [(1/3)7(0.75)(831)] + [1.5(623)] = 2,388 |
| #7 0.6D + W + 1.5($F_a$) | 0.6[774 + 7(474)] + [7(-830)] = **{-3,555}** | [(1/3)7(831)] + [1.5(623)] = **{2,874}** |
| #8 0.6D | [0.6(1,350)] + [7(306)] = 3,000 | -- |

Where

D = dead load

F = load due to fluids with well-defined pressures and maximum heights (see Section D.2 for additional information)

$F_a$ = flood load

L = live load

$L_r$ = roof live load

W = wind load

**Note:** Critical loads are in bold with brackets ({ }).

## Results

Each perimeter column needs to support the following loads:

Vertical Load = 8,138 lb

Uplift = 3,555 lb

Lateral Load = 2,874 lb

With the critical loads determined, the foundation elements and their connections to the home can be designed.

The following two examples are to demonstrate designs using information provided in this manual. The first example is based on a closed foundation; the second example is based on an open foundation.

## D.3 Closed Foundation Example

A structure to be supported by the closed foundation is identical to the structure analyzed in the example from Section D.1. The site, however, is different. For the closed foundation design, the structure is to be placed in a non-Coastal A zone where breaking waves are limited to 1.5 feet. The design stillwater depth is 2 feet, and the BFE is 3 feet above exterior grade. While the structure could be placed on a 3-foot foundation, the property owner requested additional protection from flooding and a 4-foot tall foundation is to be built. Since the home elevation is identical to that in the example, the loads and load combinations listed in Table D-1 are identical. However, since the foundation is closed, flood forces must first be analyzed.

Like the previous analysis example, flood forces consist of hydrodynamic loads, debris loads, and breaking wave loads. Since the home is located in a non-Coastal A zone, it is appropriate to use lower bound flood velocities. This will significantly reduce hydrodynamic and debris loads. From FEMA 55, the following equation is used:

$V_{lower} = d_s$

$= 2$ ft/sec

### Hydrodynamic Loads

$F_{dyn} = \frac{1}{2} C_d \rho V^2 A$

$= \frac{1}{2} C_d \rho V^2 (1)\ d_s$

$F_{dyn} = (\frac{1}{2})\ (1.4)(2)(2)^2(1)(2)$

$= 11$ lb/lf of wall

Where $C_d$ of 1.4 is for a (width of wall/$d_s$) ratio of 21 (42 ft/2 ft)

(From FEMA 55, Table 11.2)

The hydrodynamic load can be considered to act at the mid-depth of the stillwater elevation. The hydrodynamic load is less than the 27 psf wind load on the windward wall.

### Debris Loads

$F_i = wV \div gt$

$F_i = [1{,}000 \text{ lb } (2 \text{ ft/sec})] \div [(32.2 \text{ ft/sec}^2)(0.1 \text{ sec})]$

$F_i = 620$ lb

Due to load distribution, the impact load will be resisted by a section of the wall. Horizontal shear reinforcement will increase the width of the section of wall available to resist impact. For this example, a 3-foot section of wall is considered to be available to resist impact. The debris impact load becomes:

$F_{iwall}$ = (1/3) 620 lb

$F_{iwall}$ = 210 lb/ft

### Breaking Wave Loads

The home is to be constructed in an SFHA; hence, the NFIP required flood vents will be installed. The breaking wave load can be calculated using formulae for equalized flood depths (Case 2).

$F_{brkw} = 1.1\ C_p\ \gamma\ d_s^2\ + 1.91\ \gamma\ d_s^2$

$F_{brkw} = \gamma\ d_s^2\ (1.1\ C_p + 1.91)$

$F_{brkw} = (64)(2^2)\{(1.1)(2.8) + 1.91\}$

$F_{brkw}$ = 1,280 lb/lf

The breaking wave load can be considered to act at the 2-foot stillwater depth ($d_s$) above the base of the foundation wall.

The foundation must resist the loads applied to the elevated structure plus those on the foundation itself. Chapter 2 of ASCE 7-05 directs designers to include 75 percent of the flood load in load combinations 5, 6, and 7 for non-Coastal A zones. Table D-1 lists the factored loads on the elevated structure.

Critical loads from Table D-1 include 546 lb/lf uplift, 1,052 lb/lf gravity, and 831 lb/lf lateral from wind loading. The uplift load needs to be considered when designing foundation walls to resist wind and flood loads and when sizing footings to resist uplift; the gravity load must be considered when sizing footings and the lateral wind and flood loads must be considered in designing shear walls.

Extending reinforcing steel from the footings to the walls allows the designer to consider the wall as a propped cantilever fixed at its base and pinned at the top where it connects to the wood framed floor framing system. The foundation wall can also be considered simply supported (pinned at top and bottom). The analysis is somewhat simpler and provides conservative results.

The 1,280 lb/lf breaking wave load is the controlling flood load on the foundation. The probability that floodborne debris will impact the foundation simultaneously with a design breaking wave is low so concurrent wave and impact loading is not considered. Likewise, the dynamic load does not need to be considered concurrently with the breaking wave load and the 27 lb/sf wind load can not occur concurrently on a wall submerged by floodwaters.

The breaking wave load is analyzed as a point load applied at the stillwater level. When subjected to a point load (P), a propped cantilevered beam of length (L) will produce a maximum moment of 0.197 (say 0.2) PL. The maximum moments occur when "P" is applied at a distance 0.43L from the base. For the 4-foot tall wall, maximum moment results when the load is applied near the stillwater level ($d_s$). In this example, the ASCE 7-05 required flood load of 75 percent of the breaking wave load will create a bending moment of:

$M \quad = (0.2)\ f_{brkw}\ (L)$

$= (0.2)(0.75)(1{,}280\ lb/ft)\ (4\ ft)$

$= 768\ ft\text{-}lb/lf$ or

$= 9{,}200\ in\text{-}lb/lf$

The reinforced masonry wall is analyzed as a tension-compression couple with moment arm "jd," where "d" is the distance from the extreme compression fiber to the centroid of the reinforcing steel, and "j" is a factor that depends on the reinforcement ratio of the masonry wall. While placing reinforcing steel off center in the wall can increase the distance (d) (and reduce the amount of steel required), the complexity of off-center placement and the inspections required to verify proper placement make it disadvantageous to do so. For this design example, steel is considered to be placed in the center of the wall and "d" is taken as one half of the wall thickness. For initial approximation, "j" is taken as 0.85 and a nominal 8-inch wall with a thickness of 7-5/8 inches is assumed.

Solving the moment equation is as follows:

$M \quad = T\ (jd)$

$T \quad = M/(jd) =$ Tension force

$= M/(j)(t/2) \qquad (t =$ thickness of wall$)$

$T \quad = (9{,}200\ in\text{-}lb/lf) \div \{(0.85)(7.63\ in)(0.5)\}$

$= 2{,}837\ lb/lf$

For each linear foot of wall, steel must be provided to resist 2,837 pounds of bending stress and 546 pounds of uplift.

$F_{steel} \quad = 2{,}837\ lb/lf\ (bending) + 546\ lb/lf\ (uplift)$

$= 3{,}383\ lb/lf$

The American Concrete Institute (ACI) 530 allows 60 kips per square inch (ksi) steel to be stressed to 24 ksi so the reinforcement needed to resist breaking wave loads and uplift is as follows:

$A_{steel} \quad = 3{,}383\ lb/lf \div 24{,}000\ lb/in^2$

$= 0.14\ in^2/lf$

Placing #5 bars (at 0.31 $in^2$/bar) at 24 inches on centers will provide the required reinforcement. To complete the analysis, the reinforcement ratio must be calculated to determine the actual "j" factor and the stresses in the reinforcing steel need to be checked to ensure the limits dictated in ACI 530 are not exceeded. The wall design also needs to be checked for its ability to resist the lateral forces from flood and wind.

### Footing Sizing

The foundation walls and footings must be sized to prevent overturning and resist the 546 lb/lf uplift. ASCE 7-05 load combination 6 allows 60 percent of the dead load to be considered in resisting uplift. Medium weight 8-inch masonry cores grouted at 24 inches on center weigh 50 lb/sf or, for a 4-foot tall wall, 200 lb/lf. Sixty percent of the wall weight (120 lb/lf) reduces the amount of uplift the footing must resist to 426 lb/lf. At 90 lb/ft$^3$ (60 percent of 150 lb/ft$^3$ for normal weight concrete), the footing would need to have a cross-sectional area of 4.7 square feet. Grouting all cores increases the dead load to 68 lb/sf and reduces the required footing area to 4.25 square feet. The bearing capacity of the soils will control footing dimensions. Stronger soils can allow narrower footing dimensions to be constructed; weaker soils will require wider footing dimensions.

The design also needs to be checked to confirm that the footings are adequate to prevent sliding under the simultaneous action of wind and flood forces. If marginal friction resistance exists, footings can be placed deeper to benefit from passive soil pressures.

## D.4 Open Foundation Example

For this example, the calculations are based on a two-story home raised 8 feet above grade with an integral slab-grade beam, mat-type foundation and a 28-foot by 42-foot footprint. The home is sited approximately 800 feet from the shore in a Coastal A zone. Subtracting the elevation of the site (determined from a topographic map or preferably from a survey) from the ABFE and adding estimated erosion (in feet) determines that the floodwaters during a design event (including wave effects and runup) will extend 6 feet above the eroded exterior grade. It is important to note, however, that submittal of an elevation certificate and construction plans to local building code and floodplain officials in many jurisdictions will require that the elevation be confirmed by a licensed surveyor referencing an established benchmark elevation.

The wood framed home has a 3:12 roof pitch with a mean roof height of 30 feet, a center wood beam supporting the first floor, and a center load bearing wall supporting the second floor. Clear span trusses frame the asphalt-shingled roof and are designed to provide a 2-foot overhang. This home is a relatively light structure that contains no brick or stone veneers.

The surrounding site is flat, gently sloping approximately 1 foot in 150 feet. The site and surrounding property have substantial vegetation, hardwood trees, concrete sidewalks, and streets. A four-lane highway and a massive concrete seawall run parallel to the beach and the established residential area where the site is located. The beach has been replenished several times in the last 50 years. Areas to the west of the site that have not been replenished have experienced beach erosion to the face of the seawall. The ASCE 7-05, 3-second gust design wind speed is 140 mph and the site is in an Exposure Category C.

The proposed foundation for the home incorporates a monolithic carport slab placed integrally with a system of grade beams along all column lines (see Figure D-6). The dimensions of the grade beam were selected to provide adequate bearing support for gravity loads, resistance to overturning and sliding, and mitigate the potential of undermining of the grade beams and slab due to scour action. The home is supported by concrete columns, extending from the top of the slab to the lowest member of the elevated structure, spaced at 14 feet on center (see Figure D-7).

**Figure D-6.**
**Layout of Open Foundation Example.**

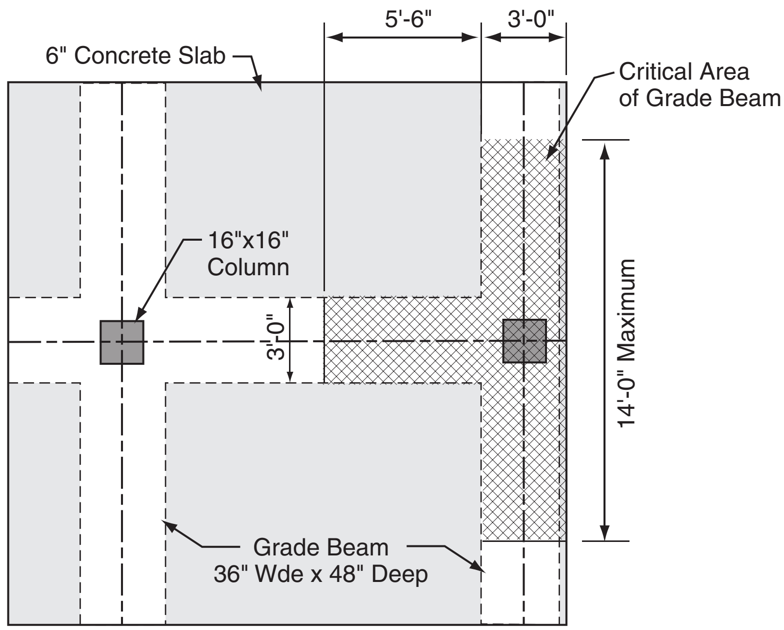
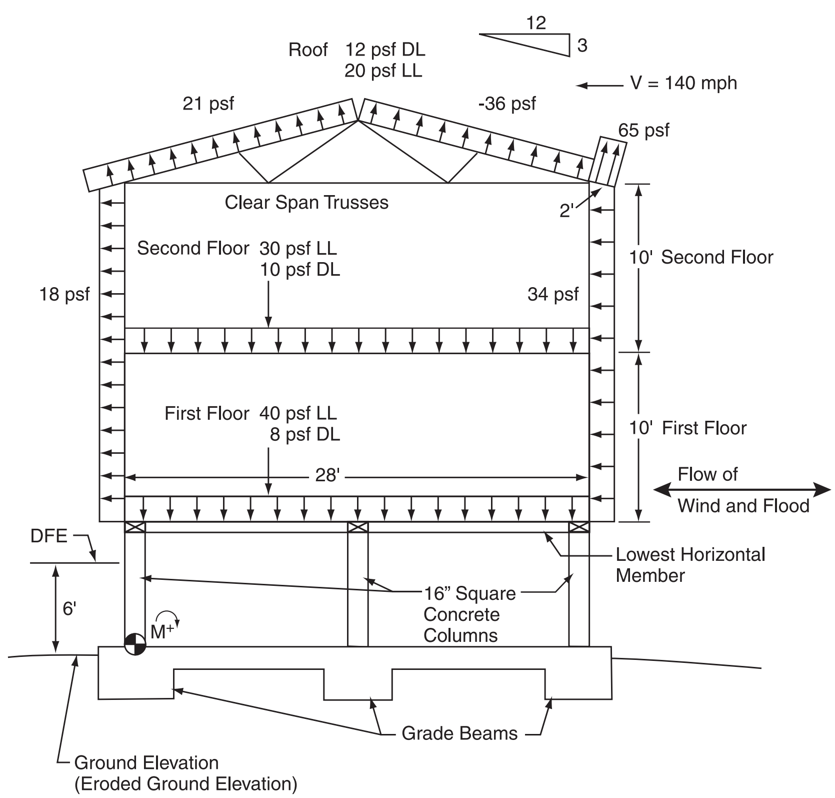



**Figure D-7.**
**Loading Diagram for Open Foundation Example.**



### Lateral Wind Loads

Sum horizontal loads ($F_{lat}$) on the elevated structure (forces to the left are positive)

$F_{lat}$ = (-126 lb/lf) + (74 lb/lf) + (280 lb/lf) + (280 lb/lf) + (180 lb/lf) + (180 lb/lf)

= 868 lb/lf

### Dead Loads

Dead load reactions ($W_{dead}$) are determined by summing loads over the tributary areas. For the anterior columns:

$W_{dead}$ = $L/2\ (w_{rfDL}) + L/4\ (w_{1stDL} + w_{2ndDL}) + (F_1 + F_2)\ w_{wlDL}$

= [14 sf/lf (12 psf)] + [7 sf/lf (8 psf + 10 psf)] + [(10 sf/lf + 10 sf/lf) 9 psf]

= 474 lb/lf

### Live Loads

**Floor**

Live loads ($W_{live}$) are calculated in a similar fashion

$W_{live}$ = $L/4(W_{1stLL} + W_{2ndLL})$

= (7 sq ft/lf) (40 + 30) psf

= 490 lb/lf

**Roof**

$W_{liveroof}$ = $L/2(W_{rfLL})$

= (14 sq ft/lf) (20 psf)

= 280 lb/lf

A minimum roof slope of 3:12 was assumed for the homes; rain loading was not considered.

### Flood Effects

Since the foundation selected is a system of concrete columns, the equations used to calculate flood loads are based on open foundation. The stillwater flooding depth ($d_s$) is as follows:

$d_s$ = DFE ÷ 1.55

= 6 ft ÷ 1.55

= 3.9 ft

### Hydrostatic Loads

Calculations are based on a 16-inch square normal weight concrete column that extends 8 feet above the concrete slab.

The column weighs 2,123 pounds dry ((1.33 ft)(1.33 ft)(8 ft)(150 lb/ft$^3$).

Under flood conditions, the column displaces 10.6 ft$^3$ of saltwater which, at 64 lb/ft$^3$, weighs 679 pounds so the column weighs 1,444 pounds when submerged.

## Hydrodynamic Loads

### Flood Velocity

Since a Coastal A zone is close to the flood source, flood velocity is calculated using the ASCE 7-05 Equation C5-2 as follows:

$$V = [(32.2 \text{ ft/sec}^2)(3.9 \text{ ft})]^{1/2}$$

$$= 11.21 \text{ feet per second (fps)}$$

### Flood Force

ASCE 7-05 Equation C5-4 is as follows:

$$F_{dyn} = ½ C_d \rho V^2 A$$

$$= (½)\ (2)(2)(11.21 \text{ fps})^2(1.33 \text{ ft})(3.9 \text{ ft})$$

$$= 1{,}303 \text{ lb/column}$$

### Floodborne Debris Impact

The flood debris impact can be estimated, per FEMA 55 Formula 11.9, as follows:

$$F_i = wV \div gt$$

$$= [1{,}000 \text{ lb } (11.21 \text{ ft/sec})] \div [(32.2 \text{ ft/sec}^2)(0.1 \text{ sec})]$$

$$= 3{,}478 \text{ lb}$$

### Breaking Wave Load

The equation for vertical piles and columns from ASCE 7-05 is as follows:

$$F_{brkp} = ½ C_{db}\, \gamma\, D H_b^2$$

$$= ½\ (2.25)(64 \text{ lb/ft}^3)(1.82 \text{ ft})(3.04 \text{ ft}^*)^2$$

$$= 1{,}211 \text{ lb}$$

### Wind Load on Columns

For a load case combining both wind and flood forces, the column would be almost completely submerged; therefore, the wind load on the column shall not be included.

---

* A wave height of 3.04 ft suggests a V zone but, in this example, the depth of water is increased by erosion, which is not considered in mapping A zones. The deeper water supports a bigger wave, which in this case exceeds the V-zone wave height minimum.

### Calculating Reactions from Wind, Live, and Dead Loads

Sum overturning moments ($M_{wind}$) and ($M_{flood}$) about the leeward corner of the mat foundation. For sign convention, consider overturning moments as negative. Note in this example the home is slightly higher above grade and hence the wind loads are slightly higher.

$M_{wind}$ = (-504 lb/lf)(21 ft) + (-294 lb/lf)(7 ft) + (126 lb/lf)(21.75 ft) + (-74 lb/lf)(21.75 ft) + (-280 lb/lf)(13 ft) + (-280 lb/lf)(23 ft) + (-180 lb/lf)(13 ft) + (-180 lb/lf)(23 ft) + (-130 lb/lf)(29 ft)

= -31,841 ft-lb/lf

The vertical components of the reaction caused by the wind overturning moment is:

$R_x$ = 31,841 lb ÷ 28 ft = +/- 1,137 lb/ft

$M_{flood}$ = 1.5[((-1,211 lb)(3.9 ft)) + (2(-1,303 lb)(3.9 ft/2)) + ((-3,478 lb)(3.9 ft))] = 35,053 ft-lb/ft

The vertical component of the reaction caused by the flood overturning moment is:

$R_x$ = 35,053 lb ÷ 28 ft = +/- 1,252 lb outboard columns

### Load Combinations

Table D-3 summarizes loads on the open foundation example. Loads are listed for the eight load combinations and critical loads are highlighted.

**Table D-3. Loads at Base of Columns Spaced 14 Feet On Center (for three rows of columns per bay)**

| ASCE 7-05 Load Combination | Vertical (lb) | Horizontal (lb) |
|---|---|---|
| #1 D + F | 1,444 + 14(474) = 8,080 | – |
| #2 D + F + L | 1,444 + 14(964) = 14,940 | – |
| #3 D + F + $L_r$ | 1,444 + 14(754) = 12,000 | – |
| #4 D + F + 0.75(L) + 0.75($L_r$) | 8,080 + (0.75)[(14)(490+280)] = 16,165 | – |
| #5 D + F + W + 1.5($F_a$) | 8,080 +/- 14(1,137) +/- 1,252 = 25,876; -9,716 windward; leeward | wind + (1.5)[$F_{dyn}$ + $F_i$] [(14(868)(1/3)] + (1.5) [(1,303+3,478)] = 11,222 |
| #6 D + F + 0.75(W) + 0.75(L) + 0.75($L_r$) + 1.5($F_a$) | 8,080 +/- (0.75)(14)(1,137) +(0.75)(14)([(490+280)] +/- 1,252 = 2,348; **29,982** windward; leeward | (0.75) wind + (1.5)[$F_{dyn}$ + $F_i$] [(0.75)(14)(868)(1/3)] + (1.5)[(1,303+3,478)] = 10,210 |
| #7 0.6D + W + 1.5($F_a$) | 0.6 [2,123+14(474)] +/- 14(1,137) +/- (1.5)1,252 = **-12,541**; 23,051 windward; leeward | wind + (1.5)[$F_{dyn}$ + $F_i$] [(14(868)(1/3)] + (1.5) [(1,303+3,478)] = **11,222** |
| #8 0.6D | [0.6((2,123) + 14(474))] = 5,255 | – |

Critical loads are in bold.

Where

| | |
|---|---|
| D | = dead load |
| F | = fluid (buoyancy) load |
| L | = live load |
| $L_r$ | = roof live load |
| W | = wind load |
| ww | = windward |
| lw | = leeward |

**Results**

Each perimeter column needs to support the following loads:

| | |
|---|---|
| Vertical Load | = 29,982 lb |
| Uplift | = 12,541 lb |
| Lateral Load | = 11,222 lb |
| Moment wind + $f_{dyn}$ | = [(1/3)(14)(1,314)(8) + (1,303)(3.9/2)] ÷ 1,000 lb/kip |
| | = 51.6 ft-kip |
| Moment wind + $f_{brkp}$ | = [(1/3)(14)(1,314)(8) + (1,113)(3.9)] ÷ 1,000 lb/kip |
| | = 96.9 ft-kip |
| Moment debris | = (3,478)(3.9) ÷ 1,000 lb/kip |
| | = 13.6 ft-kip |
| Moment wind + $f_{dyn}$+ debris | = 51.6 ft-kip + 13.6 ft-kip |
| | = 65.1 ft-kip |

The force is assumed to act at a point $d_s/2$ above the eroded ground surface. For concrete design, we use load factors per ASCE 7-05.

| | |
|---|---|
| Ultimate Moment wind + $f_{dyn}$ | = (48.6)(1.2)+(2.5)(2.0) |
| | = 63.4 ft-kip |
| Ultimate Moment wind + $f_{brkp}$ | = (48.6)(1.2)+(4.3)(2.0) |
| | = 66.9 ft-kip |
| Ultimate Moment wind + $f_{dyn}$ + debris | = (63.4)+13.6(2) |
| | = 90.6 ft-kip |

## Foundation Design

### Overturning

The overturning moment due to wind with a typical bay of 14 feet wide is as follows:

$M_{wind}$ = (-31,841 ft-lb/lf) (14 ft)

= -445,774 ft-lb

$M_{Fa}$ (1.5) = (1.5) [ (1,211 lb) (3.9 ft) + (2) (1,303 lb) (3.9 ft/2) ]

= -14,707 ft-lb

$M_o$ = -445,774 ft-lb - 14,707 ft-lb

= -460,481 ft-lb

In this example, it is assumed that the home and the foundation slab are reasonably symmetrical and uniform; therefore, it is assumed the center of gravity for the dead loads is at the center of the bay.

Dead load at perimeter columns

$D_{ext}$ = (474 lb/ft) (14 ft) (2 columns)

= 13,272 lb

Dead load at an interior column:

$D_{int}$ = (14 ft) (14 ft) (8 psf)

= 1,568 lb

Dead load of 3 columns: (3) (8 ft) (1.33 ft x 1.33 ft) (150)lb cubic ft

= 6,368 lb

Assume that only the grade beams are sufficiently reinforced to resist overturning (neglect weight of slab)

Dead load of the grade beams (area) (depth) (density of concrete)
[ (28 ft x 3 ft) + (3) ( (11 ft) (3 ft) ] (4 ft) (150 lb cubic ft) = 109,800 lb

Summing the dead loads = 13,272 + 1,568 + 6,368 + 109,800 = 131,008 lb

Allowable dead load moment of 60 percent

$M_d$ = (0.6) ( 131,008 lb) (14 ft)

= 1,100,476 ft-lb

Since $M_{ot}$ = 460,481 ft-lb is very much less than 0.6 $M_d$ = 1,100,476 ft-lb, the foundation can be assumed to resist overturning.

**Sliding**

The maximum total lateral load of wind and flood acting on the entire typical bay is as follows:

$L_{wfa}$ = W + 1.5 $F_a$

= [(14 ft (868 lb/ ft) + 1.5(7,203 lb)]

= 23,375 lb

Sliding Resistance = (tanΦ)N + Passive Resistance at Vertical Foundation Surfaces

Φ = internal angle of soil friction, assume Φ = 25 degrees

N = net normal force (building weight - uplift forces)

N = (131,008) + (14 ft)[((16 ft)(-21 psf )+(14 ft)(-36 psf)+(2 ft)(-65 psf)]

= 117,428 lb

Ignore passive soil pressure

Dead Load Sliding Resistance = (tan 25)(117,428 lb)

= 54,758 lb

Since $L_{wfa}$ = 23,371 lb is less than 60 percent of Dead Load Sliding Resistance = 54,758, the foundation can be assumed to resist sliding.

## Soil Bearing Pressure

The simplified approach for this mat foundation assumes that only the grade beams carry loads to the soil; the slab between grade beams is not considered to contribute support. It is further assumed that the bearing pressure is uniform in the absence of wind and flood loading. The areas of the grade beams along the outboard column lines, in the direction of the flow of wind and flood, are considered the "critical areas" of the grade beam. The load combination table below indicates the bearing pressures for the ASCE 7- 05 load combinations for the critical grade beam area. These load combinations are calculated to ensure that downward forces of the wind and flood moment couple do not overstress the soil. The factored dead load moment that resists overturning is of a magnitude such that there is no net uplift along critical grade beams. Table D-4 presents foundation bearing pressures for typical bays.

Self Weight of 1 square of foot Grade Beam = (4 ft)(150 lb/cubic ft) = 650 psf

Area of Critical Grade Beam = [(3ft)(14 ft)] + [(3 ft)(5.5 ft)]

= 58.5 $ft^2$

Weight of Critical Grade Beam = (58.5 $ft^2$)(4 ft)(150 lb/cubic ft)

= 35,100 lb

Critical Column Uplift = 12,541 lb (load combination 7)

Verification of Uplift Resistance = [35,100 lb(0.6)] – 12,541 lb

= 8,519 lb (positive load, no uplift)

Presumptive Allowable Bearing Pressure = 1,500 psf

Table D-4. Foundation Bearing Pressures for Typical Bays (for three rows of columns per bay)

| ASCE 7-05 Load Combination | Combined Loads (lb) | Bearing Pressures (psf) |
|---|---|---|
| #1 D + F | 8,080 | 8,090 ÷ 58.5 + 650 = 788 |
| #2 D + F + L | 14,940 | 14,950 ÷ 58.5 + 650 = 905 |
| #3 D + F + $L_r$ | 12,000 | 12,000 ÷ 58.5 + 650 = 855 |
| #4 D + F + 0.75(L) + 0.75($L_r$) | 16,165 | 16,165 ÷ 58.5 + 650 = 926 |
| #5 D + F + W + 1.5($F_a$) | 28,104 | 28,104 ÷ 58.5 + 650 = 1,130 |
| #6 D + F + 0.75(W) + 0.75(L) + 0.75($L_r$) + 1.5($F_a$) | 29,982 | 29,982 ÷ 58.5 + 650 = **1,163** |
| #7 0.6D + W + 1.5($F_a$) | 23,051 | 23,051 ÷ 58.5 + 650 = 1,044 |
| #8 0.6D | 5,255 | 5,255 ÷ 58.5 + 650 = 740 |

Where

D = dead load

F = load due to fluids with well-defined pressures and maximum heights (See Section D.2 for additional information)

$F_a$ = flood load

L = live load

$L_r$ = roof live load

W = wind load

**Note:** The maximum bearing pressure is in bold (1,163 psf) and is less than the assumed 1,500 psf bearing pressure.

**Design of Concrete members per ACI-318-02 Code and ASCE 7-05; Sections 2.3.2 and 2.3.3-1**

## Column Design

Verify that 16-inch x 16-inch column design is adequate.

Concrete strength = 4,000 psi

Reinforced with (4) #8 bars, grade 60 reinforcing, with 2½-inch clear cover

Note: 1,000 lb = 1.0 kip

Assume that the total wind load distributed through the floor uniformly to 3 columns.

Check combined axial and bending strength:

Ultimate Moment wind + $f_{dyn}$ = (1.6) [(8 ft)((14 ft)(868 lb/ft)/(3))] + (2.0) [(3.9 ft/2)(1,303) lb)]

= 51,849 ft-lb + 5,082 ft-lb

= 56,931 ft-lb ÷ 1,000 lb/kip = 56.9 ft-kip

Ultimate Moment wind + $f_{brkp}$ = (51,849 ft-lb) + (2.0) [(1,211 lb) (3.9 ft)]

= 61,295 ft-lb ÷ 1,000 lb/kip = 61.3 ft-kip

Ultimate Moment wind + $f_{brkp}$ + debris = 61.3 ft-kip + (2.0) (3.5 kip) (3.9 ft)

= 88,6 ft-kip

Maximum Factored Moment = 88.6 ft-kip = 1,063 in kip

Refer to Table D-3, conservatively assume flood load factor of 2.0 for all axial loads

Maximum factored Axial Compression = (2.0) (30.0 kip) = 60.0 kip

Maximum factored Axial Tension = (2.0) (12.5 kip) = 25.0 kip

Based on a chart published by the Concrete Reinforced Steel Institute (CRSI), the maximum allowable moment for the column = 1,092 in kip for 0 axial load and 1,407 in kip for 102 kip axial load; therefore, the column is adequate.

Check shear strength:

Critical Shear = wind + $F_{dyn}$ + $F_i$

Ultimate Shear = $V_u$ = [(14 ft) (0.868 kip) (1/3)] (1.6) + [(1.3 kip)+(3.5 kip] (2.0)

= $V_u$ = 16.0 kip

As the maximum unit tension stress is only 25.0 kip/16 in x 16 in = 0.098 kip/in$^2$ and the maximum axial compression stress is only 60.0 kip/16 in x 16 in = 0.234 kip/in$^2$, we can conservatively treat the column as a flexural member or beam. The allowable shear of the concrete section then, per ACI-318-02 11.3.1.1, 11.3.1.3, and 11.5.5.1 with minimum shear reinforcing (tie/stirrup), would be as follows.

Allowable Shear = $V_c$ = (0.75) (16 in-2.5 in) (16 in ) (2) (4,000 psi)$^{1/2}$ (1/1,000) = 20.5 kip

The shear strength of the column is adequate with minimum shear reinforcement.

The minimum area of shear, $A_v$, per ACI 318-02, 11.5.5.3 would be:

| | | |
|---|---|---|
| $A_v$ | = (50) (width of member) (spacing of reinforcing)/yield strength of reinforcing | |
| | = (50) (16 in) (16 in)/(60,000 psi) | = 0.21 $in^2$ |
| 2 pieces of #4 bar $A_s$ | = (2) (0.2) | = 0.40 $in^2$ |

Use of #4 bar for column ties (shear reinforcement) is adequate.

Check spacing per ACI -318-02, 7.10.4

| | | |
|---|---|---|
| 16 diameter of vertical reinforcing bar | = (16) (1 in) | = 16 in |
| 48 diameter of column tie bar | = (48) (1/2 in) | = 24 in |
| Least horizontal dimension | = 16 in | |

Therefore, #4 ties at 16 inches on center are adequate.

The column design is adequate.

## Grade Beam Design

The size of the grade beam was configured to provide adequate bearing area, resistance to uplift, a reasonable measure of protection from damaging scour, and to provide a factor of redundancy and reserve strength should the foundation be undermined. A grade beam 36 inches wide and 48 inches deep was selected.

Maximum Bearing Pressure = 1,163 psf = 1.2 ksf (kip/square foot)

Assume a combined load factor of 2.0 (for flood)

Check Shear strength:

Maximum Factored Uniform Bearing Pressure = $w_u$ = (2.0) (3.0 ft ) (1.2 ksf) = 7.2 kip/ft

Maximum Factored Shear = $V_u$ = (7.2 kip/ft) (14 ft/2) = 50.0 kip

Allowable Shear without minimum shear reinforcing (stirrups) = $V_c/2$

$V_c/2$= (0.75) (36 in) (48 - 3.5 in) (63 psi) (1/1,000) = 75.7 kip

Use nominal #4 two leg stirrups at 24 inches on center

Check flexural strength: Assume simple span condition

Maximum Factored Moment = $M_u$ = (7.2 kip/ft) $(14\ ft)^2$ (1/8) = 176 ft-kip

Concrete strength = 4,000 psi

Reinforcement grade = 60,000 psi

Try (4) #6 reinforcing bar continuous top and bottom

$A_s$ = (4) (0.44 $in^2$) = 1.76 $in^2$ top or bottom

total $A_s$ = (2) (1.76) = 3.52 $in^2$

Reinforcement ratio ($\rho$)

$\rho$ = $A_s$ ÷ [ (section width) (section depth- clear cover- ½ bar diameter) ]

$\rho$ = $A_s$ ÷ [ ($b_w$) (d) ]

$\rho$ = (1.76 $in^2$) ÷ [ (36) (48 - 3 - 0.375) ]

$\rho$ = 0.01096

One method of calculating the moment strength of a rectangular beam, for a given section and reinforcement, is illustrated in the 2002 edition of the *CRSI Design Handbook*. Referencing page 5-3 of the handbook, the formula for calculating the moment strength can be written as follows:

$\Phi M_n$ = $(\Phi)[((A_s)(f_y)(d)) - (((A_s)(f_y)) \div ((0.85)(f'c)(\text{width of member})(2)))]$

$\Phi M_n$ = $(0.9)[((1.76)(60{,}000)(44.63)) - (((1.76)(60{,}000)) \div ((0.85)(4{,}000)(36)(2)))]$

= 4,241,634 in lb ÷ 12,000 = 353 ft-kip

$\Phi$ = 0.9

Area of reinforcing steel ($A_s$) minimum flexural

= (0.0033) (24) (44.63)

= 3.6 $in^2$

or

(1.33) ($A_s$ required by analysis) = $\Phi M_n$ is much greater than $M_u$

Area of steel for shrinkage and temperature required = (0.0018) (48) (36) = 3.1 $in^2$

Total area of steel provided = (8) #6 = (8) (0.44) = 3.52 $in^2$ adequate

Therefore, the grade beam design is adequate, use (4) #6 reinforcing bar continuous on the top and bottom with #4 stirrups at 24-inch spacing.