Coastal Construction: Designing the Foundation

Course No: S04-017
Credit: 4 PDH

Gilbert Gedeon, P.E.

CED CANADA engineering.ca
CPD FOR THE PROFESSIONAL

Continuing Education and Development, Inc.

P: (877) 322-5800
info@cedengineering.ca

10 Designing the Foundation

This chapter provides guidance on designing foundations, including selecting appropriate materials, in coastal areas. It provides general guidance on designing foundations in a coastal environment and is not intended to provide complete guidance on designing foundations in every coastal area. Design professionals should consult other guidance documents, codes, and standards as needed.

> **CROSS REFERENCE**
>
> For resources that augment the guidance and other information in this Manual, see the Residential Coastal Construction Web site (http://www.fema.gov/rebuild/mat/fema55.shtm).

Design considerations for foundations in coastal environments are in many ways similar to those in inland areas. Like all foundations, coastal foundations must support gravity loads, resist uplift and lateral loads, and maintain lateral and vertical load path continuity from the elevated building to the soils below. Foundations in coastal areas are different in that they must generally resist higher winds, function in a corrosive environment, and withstand the environmental aspects that are unique to coastal areas: storm surges, rapidly moving floodwaters, wave action, and scour and erosion. These aspects can make coastal flooding more damaging than inland flooding.

Like many design processes, foundation design is an iterative process. First, the loads on the elevated structure are determined (see Chapter 9). Then a preliminary foundation design is considered, flood loads on the preliminary design are determined, and foundation style is chosen and the respective elements are sized to resist those loads. With information on foundation size, the design professional can accurately determine flood loads on the foundation and can, through iteration, develop an efficient final design.

Because flood loads depend greatly on the foundation design criteria, the discussion of foundation design begins there. The appropriate styles of foundation are then discussed and how the styles can be selected to reduce vulnerability to natural hazards.

The distinction between *code requirements* and *best practices* is described throughout the chapter.

10.1 Foundation Design Criteria

Foundations should be designed in accordance with the latest edition of the 2012 IBC or the 2012 IRC and must address any locally adopted building ordinances. Designers will find that other resources will likely be needed in addition to the building codes in order to properly design a coastal foundation. These resources are listed at the end of this chapter. Properly designed and constructed foundations are expected to:

- Support the elevated building and resist all loads expected to be imposed on the building and its foundation during a design flood, wind, or seismic event
- In SFHAs, prevent flotation, collapse, and lateral movement of the building
- Function after being exposed to scour and erosion

In addition, the foundation must be constructed with flood-resistant materials below the BFE. See Technical Bulletin 2, *Flood Damage-Resistant Materials Requirements* (FEMA 2008a), and Fact Sheet 1.7, *Coastal Building Materials,* in FEMA P-499 (FEMA 2011).

Some coastal areas mapped as Zone A are referred to as "Coastal A Zones." Following Hurricane Katrina (2005), Coastal A Zones have also been referred to as areas with a Limit of Moderate Wave Action (LiMWA). Buildings in Coastal A Zones may be subjected to damaging waves and erosion and, when constructed to minimum NFIP requirements for Zone A, may sustain major damage or be destroyed during the base flood. Therefore, in this Manual, foundations for buildings in Coastal A Zones are strongly recommended to be designed and constructed with foundations that resist the damaging effects of waves.

TERMINOLOGY: LiMWA AND COASTAL A ZONE

Limit of Moderate Wave Action (LiMWA) is an advisory line indicating the limit of the 1.5-foot wave height during the base flood. FEMA requires new flood studies in coastal areas to delineate the LiMWA.

10.2 Foundation Styles

In this Manual, foundations are described as open or closed and shallow or deep. The open and closed descriptions refer to the above-grade portion of the foundation. The shallow and deep descriptions refer to the below-grade portion. Foundations can be open and deep, open and shallow, or closed and shallow. Foundations can also be closed and deep, but these foundations are relatively rare and generally found only in areas where (1) soils near the surface are relatively weak (700 pounds/square foot bearing capacity or less), (2) soils near the surface contain expansive clays (also called shrink/swell soils) that shrink when dry and swell when wet, or (3) other soil conditions exist that necessitate foundations that extend into deep soil strata to provide sufficient strength to resist gravity and lateral loads.

Open, closed, deep, and shallow foundations are described in the following subsections.

10.2.1 Open Foundations

An open foundation allows water to pass through the foundation of an elevated building, reducing the lateral flood loads the foundation must resist. Examples of open foundations are pile, pier, and column foundations. An open foundation is designed and constructed to minimize the amount of vertical surface area that is exposed to damaging flood forces. Open foundations have the added benefit of being less susceptible than closed foundations to damage from flood-borne debris because debris is less likely to be trapped.

Open foundations are required in Zone V and recommended in Coastal A Zone. Table 10-1 shows the recommended practices in Coastal A Zone and Zone V.

Table 10-1. Foundation Styles in Coastal Areas

Foundation Style	Zone V	Coastal A Zone (LiMWA)	Zone A
Open/deep	Acceptable	Acceptable	Acceptable
Open/shallow	Not permitted	Acceptable[(a)]	Acceptable
Closed/shallow	Not permitted	Not recommended	Acceptable
Closed/deep	Not permitted	Not recommended	Acceptable

LiMWA = Limit of Moderate Wave Action

(a) Shallow foundations in Coastal A Zone are acceptable only if the maximum predicted depth of scour and erosion can be accurately predicted and foundations can be constructed to extend below that depth.

10.2.2 Closed Foundations

A closed foundation is typically constructed using continuous perimeter foundation walls. Examples of closed foundations are crawlspace foundations and stem wall foundations,[1] which are usually filled with compacted soil. Slab-on-grade foundations are also considered closed.

A closed foundation does not allow water to pass easily through the foundation elements below an elevated building. Thus, these types of foundations obstruct floodwater flows and present a large surface area upon which waves and flood forces act. Closed foundations are prohibited in Zone V and are not recommended in Coastal A Zones. If perimeter walls enclose space below the DFE, they must be equipped with openings that allow floodwaters to flow in and out of the area enclosed by the walls (see Figure 2-19). The entry and exit of floodwater equalizes the water pressure on both sides of the wall and reduces the likelihood that the wall will fail. See Fact Sheet No. 3.5, *Foundation Walls,* in FEMA P-499, *Home Builder's Guide to Coastal Construction Technical Fact Sheet Series* (FEMA 2010).

Closed foundations also create much larger obstructions to moving floodwaters than open foundations, which significantly increases localized scour. Scour, with and without generalized erosion, can remove soils that support a building and can undermine the foundation and its footings. Once undermined, shallow footings readily fail (see Figure 10-1).

1 Stem wall foundations (in some areas, referred to as chain wall foundations) are similar to crawlspace foundations where the area enclosed by the perimeter walls are filled with compacted soil. Most stem wall foundations use a concrete slab-on-grade for the first floor. The NFIP requires flood vents in crawlspace foundations but not in stem wall foundations (see Section 6.1.1.1 and Section 7.6.1.1.5).

Figure 10-1.
Closed foundation failure due to erosion and scour undermining; photograph on right shows a close-up view of the foundation failure and damaged house wall, Hurricane Dennis (Navarre Beach, FL, 2005)

10.2.3 Deep Foundations

Buildings constructed on deep foundations are supported by soils that are not near grade. Deep foundations include driven timber, concrete or steel piles, and caissons.

Deep foundations are much more resistant to the effects of localized scour and generalized erosion than shallow foundations. Because of that, deep foundations are required in Zone V where scour and erosion effects can be extreme. Open/deep foundations are recommended in Coastal A Zones and in some riverine areas where scour and erosion can undermine foundations.

10.2.4 Shallow Foundations

Buildings constructed on shallow foundations are supported by soils that are relatively close to the ground surface. Shallow foundations include perimeter strip footings, monolithic slabs, discrete pad footings, and some mat foundations. Because of their proximity to grade, shallow foundations are vulnerable to damage from scour and erosion, and because of that, they are not allowed in Zone V and are not recommended in Coastal A Zones unless they extend below the maximum predicted scour and erosion depth.

In colder regions, foundations are typically designed to extend below the frost depth, which can exceed several feet below grade. Extending the foundation below the frost depth is done to prevent the foundation from heaving when water in the soils freeze and to provide adequate protection from scour and erosion. However, scour and erosion depths still need to be investigated to ensure that the foundation is not vulnerable to undermining.

10.3 Foundation Design Requirements and Recommendations

Foundations in coastal areas must elevate the home to satisfy NFIP criteria. NFIP criteria vary for Zone V and Zone A. In Zone V, the NFIP requires that the building be elevated so that the bottom of the lowest horizontal structural member is elevated to the BFE. In Zone A, the NFIP requires that the home be constructed such that the top of the lowest floor is elevated to the BFE.

In addition to elevation, the NFIP contains other requirements regarding foundations. Because of the increased flood, wave, flood-borne debris, and erosion hazards in Zone V, the NFIP requires homes to be elevated on open/deep foundations that are designed to withstand flood forces, wind forces, and forces for flood-borne debris impact. They must also resist scour and erosion.

10.3.1 Foundation Style Selection

Many foundation designs can be used to elevate buildings to the DFE. Table 10-1 shows which foundation styles are acceptable, not recommended, or not permitted in Zone V, Coastal A Zone, and Zone A. Additional information concerning foundation performance can be found in Fact Sheet 3.1, *Foundations in Coastal Areas,* in FEMA P-499.

A best practices approach in the design and construction of coastal foundations is warranted because of the extreme environmental conditions in coastal areas, the vulnerability of shallow foundations to scour and erosion, the fact that the flood loads on open foundations are much lower than those on closed foundations, and foundation failures typically result in extensive damage to or total destruction of the elevated building.

Structural fill can also be used to elevate and support stem wall, crawlspace, solid wall, slab-on-grade, pier, and column foundations in areas not subject to damaging wave action, erosion, and scour. The NFIP precludes the use of structural fill in Zone V. For more information, see FEMA Technical Bulletin 5, *Free-of-Obstruction Requirements* (FEMA 2008b).

10.3.2 Site Considerations

The selected foundation design should be based on the characteristics of the building site. A site characteristic study should include the following:

- **Design flood conditions.** Determine which flood zone the site is located in—Zone V, Coastal A Zone, or Zone A. Flood zones have different hazards and design and construction requirements.
- **Site elevation.** The site elevation and DFE determine how far the foundation needs to extend above grade.
- **Long- and short-term erosion.** Erosion patterns (along with scour) dictate whether a deep foundation is required. Erosion depth affects not only foundation design but also flood loads by virtue of its effect on design stillwater depth (see Section 8.5).
- **Site soils.** A soils investigation report determines the soils that exist on the site and whether certain styles of foundations are acceptable.

10.3.3 Soils Data

Accurate soils data are extremely important in the design of flood-resistant foundations in coastal areas. Although many smaller or less complex commercial buildings and most homes in non-coastal areas are designed without the benefit of specific soils data, all buildings in coastal sites, particularly those in Zone V, should have a thorough investigation of the soils at the construction site. Soils data are available in numerous publications and from onsite soils tests.

10.3.3.1 Sources of Published Soils Data

Numerous sources of soil information are available. Section 12.2 of the *Timber Pile Design and Construction Manual* (Collin 2002) lists the following:

- Topographic maps from the U.S. Geologic Survey (USGS)
- Topographic maps from the Army Map Service
- Topographic maps from the U.S. Coast and Geodetic Survey
- Topographic information from the USACE for some rivers and adjacent shores and for the Great Lakes and their connecting waterways
- Nautical and aeronautical charts from the Hydrographic Office of the Department of the Navy
- Geologic information from State and local governmental agencies, the Association of Engineering Geologists, the Geological Society of America, the Geo-Institute of the American Society of Civil Engineers, and local universities
- Soil survey maps from the Soil Conservation Service of the U.S. Department of Agriculture

10.3.3.2 Soils Data from Site Investigations

Site investigations for soils include surface and subsurface investigations. Surface investigations can identify evidence of landslides, areas affected by erosion or scour, and accessibility for equipment needed for subsurface testing and for equipment needed in construction. Surface investigations can also help identify the suitability or unsuitability of particular foundation styles based on the past performance of existing structures. However, caution should be used when basing the selection of a foundation style solely on the performance of existing structures because the structures may not have experienced a design event.

The 2012 IBC requires that geotechnical investigations be conducted by Registered Design Professionals. Section 1803.2 allows building officials to waive geotechnical investigations where satisfactory data are available from adjacent areas and demonstrate that investigations are not required. The 2012 IRC requires building officials to determine whether soils tests are needed where “quantifiable data created by accepted soil science methodologies indicate expansive, compressible, shifting or other questionable soil characteristics are likely to be present.” Because of the hazards in coastal areas, a best practices approach is to follow the 2012 IBC requirements.

Subsurface exploration provides invaluable data on soils at and below grade. The data are both qualitative (e.g., soil classification) and quantitative (e.g., bearing capacity). Although some aspects of subsurface exploration are discussed here, subsurface exploration is too complicated and site-dependent to be covered fully in one document. Consulting with geotechnical engineers familiar with the site is strongly recommended.

Subsurface exploration typically consists of boring or creating test pits, soils sampling, and laboratory tests. The *Timber Pile Design and Construction Manual* (Collin 2002) recommends a minimum of one boring per structure, a minimum of one boring for every 1,000 square feet of building footprint, and a minimum of two borings for structures that are more than 100 feet wide. Areas with varying soil structure and profile dictate more than the minimum number of borings. Again, local geotechnical engineers should be consulted.

The following five types of data from subsurface exploration are discussed in the subsections below: soil classification, bearing capacity, compressive strength, angle of internal friction, and subgrade modulus.

Soil Classification

Soil classification qualifies the types of soils present along the boring depth. ASTM D2487-10 is a consensus standard for soil classification. Soil classification is based on whether soils are cohesive (silts and clays) or non-cohesive (composed of granular soils particles). The degree of cohesiveness affects foundation design. Coupled with other tests such as the plasticity/Atterburg Limits soil classification can identify unsuitable or potentially problematic soils. Table 10-2 contains the soil classifications from ASTM D2487-10. ASTM D2488-09a is a simplified standard for soil classification that may be used when directed by a design professional.

Bearing Capacity

Bearing capacity is a measure of the ability of soil to support gravity loads without soil failure or excessive settlement. Bearing capacity is generally measured in pounds/square foot and occasionally in tons/square foot. Soil bearing capacity typically ranges from 1,000 pounds/square foot (relatively weak soils) to more than 10,000 pounds/square foot (bedrock).

Bearing capacity has a direct effect on the design of shallow foundations. Soils with lower bearing capacities require proportionately larger foundations to effectively distribute gravity loads to the supporting soils. For deep foundations, like piles, bearing capacity has less effect on the ability of the foundation to support gravity loads because most of the resistance to gravity loads is developed by shear forces along the pile.

Presumptive allowable load bearing values of soils are provided in the 2012 IBC and the 2012 IRC. Frequently, designs are initially prepared based on presumed bearing capacities. The builder's responsibility is to verify that the actual site conditions agree with the presumed bearing capacities. As a ***best practices approach***, the actual soil bearing capacity should be determined to allow the building design to properly account for soil capacities and characteristics.

Compressive Strength

Compressive strength is typically determined by Standard Penetration Tests. Compressive strength controls the design of shallow foundations via bearing capacity and deep foundations via the soil's resistance to lateral loads. Compressive strength is also considered when determining the capacity of piles to resist vertical loads.

Compressive strength is determined by advancing a probe, 2 inches in diameter, into the bottom of the boring by dropping a 140-pound slide hammer a height of 30 inches. The number of drops, or blows, required to advance the probe 6 inches is recorded. Blow counts are then correlated to soil properties.

Table 10-2. ASTM D2487-10 Soil Classifications

<table>
<tr><th colspan="3">Major Divisions</th><th>Group Symbol</th><th>Typical Names</th><th colspan="3">Classification Criteria</th></tr>
<tr><td rowspan="8">Coarse-grained soils more than 50% retained on No. 200 sieve</td><td rowspan="4">Gravels: 50% or more of coarse fraction retained on No. 4 sieve</td><td rowspan="2">Clean gravels</td><td>GW</td><td>Well-graded gravels and gravel-sand mixtures, little or no fines</td><td rowspan="8">Classification on basis of percentage of fines:
• Less than 5% pass No. 200 sieve: GW, GP, SW, SP
• More than 12% pass No. 200 sieve: GM, GC, SM, SC
• 5% to 12% pass No. 200 sieve: borderline classification requiring dual symbols</td><td colspan="2">$C_u = \frac{D_{60}}{C_{10}}$ greater than 4
$C_Z = \frac{(D_{30})^2}{(D_{10})(D_{60})}$
between 1 and 3</td></tr>
<tr><td>GP</td><td>Poorly graded gravels and gravel-sand mixtures, little or no fines</td><td colspan="2">Not meeting both criteria for GW</td></tr>
<tr><td rowspan="2">Gravels with fines</td><td>GM</td><td>Silty gravels, gravel-sand-silt mixtures</td><td>Atterberg limits plot below "A" line or plasticity index less than 4</td><td rowspan="2">Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.</td></tr>
<tr><td>GC</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td>Atterberg limits plot above "A" line or plasticity index less than 7</td></tr>
<tr><td rowspan="4">Sands: More than 50% of coarse fraction passes No. 4 sieve</td><td rowspan="2">Clean sands</td><td>SW</td><td>Well-graded sands and gravelly sands, little or no fines</td><td colspan="2">$C_u = \frac{D_{60}}{C_{10}}$ greater than 6
$C_Z = \frac{(D_{30})^2}{(D_{10})(D_{60})}$
between 1 and 3</td></tr>
<tr><td>SP</td><td>Poorly graded sands and gravelly sands, little or no fines</td><td colspan="2">Not meeting both criteria for SW</td></tr>
<tr><td rowspan="2">Sands with fines</td><td>SM</td><td>Silty sands, sand-silt mixtures</td><td>Atterberg limits plot below "A" line or plasticity index less than 4</td><td rowspan="2">Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.</td></tr>
<tr><td>SC</td><td>Clayey sands, sand-clay mixtures</td><td>Atterberg limits plot above "A" line or plasticity index greater than 7</td></tr>
</table>

Table 10-2. ASTM D2487-10 Soil Classifications (concluded)

Major Divisions		Group Symbol	Typical Names	Classification Criteria
Fine-grained soils: 50% or more passes No. 200 sieve	Silts and clay liquid limit 50% or less	ML	Inorganic silts, very fine sands, rock flout, silty or clayey fine sands	
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
Fine-grained soils: 50% or more passes No. 200 sieve	Silts and clay liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
		CH	Inorganic clays of high plasticity, fat clays	
		OH	Organic clays of medium to high plasticity	
Highly organic soils		PT	Peat, muck, and other highly organic soils	

Adapted, with permission, from ASTM D2487-10 *Standard Practice for Classification of Soils for Engineering Purposes* (Unified Soil Classification System), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. The complete standard is available at ASTM International, http://www.astm.org.

Angle of Internal Friction/Soil Friction Angle

The angle of internal friction is a measure of the soil's ability to resist shear forces without failure. Internal friction depends on soil grain size, grain size distribution, and mineralogy.

The angle of internal friction is used in the design of shallow and deep foundations. It is also used to determine the sliding resistance developed between the bottom of a footing and the foundation at the adjacent soil strata via Equation 10.1.

The following factors should be considered. The normal force includes only the weight of the building (dead load). Live loads should not be considered. Also, ASD load factors in ASCE 7-10 allow only 60 percent of the dead load of a structure to be considered when resisting sliding forces. Foundation materials exert less normal force on a foundation when submerged, so the submerged weight of all foundation materials below the design stillwater depth should be used.

Editions of the IBC contain presumptive coefficients of friction for various soil types (for example, coefficients of friction are contained in Table 1806.2 in the 2009 IBC). Those coefficients can be used in Equation 10.1 by substituting them for the term "tan (φ)."

Subgrade Modulus n_h

The subgrade modulus (n_h) is used primarily in the design of pile foundations. It, along with the pile properties, determines the depth below grade of the point of fixity (point of zero movement and rotation) of a pile under lateral loading.

The inflection point is critical in determining whether piles are strong enough to resist bending moments caused by lateral loads on the foundation and the elevated building. The point of fixity is deep for soft soils (low subgrade modulus) and stiff piles and shallow for stiff soils (high subgrade modulus) and flexible piles.

Subgrade moduli range from 6 to 150 pounds/cubic inch for soft clays to 800 to 1,400 pounds/cubic inch for dense sandy gravel. See Section 10.5.3 for more information on subgrade modulus.

EQUATION 10.1. SLIDING RESISTANCE

$$F = \tan(\varphi)(N)$$

where:

F = resistance to sliding (lb)

φ = angle of internal friction

N = normal force on the footing (lb)

10.4 Design Process

The following are the major steps in foundation analysis and design.

- Determine the flood zone that the building site is in. For a site that spans more than one flood zone (e.g., Zone V and Coastal A Zone, Coastal A Zone and Zone A), design the foundation for the most severe zone (see Chapter 3).
- Determine the design flood elevation and design stillwater elevation (see Chapter 8).
- Determine the projected long- and short-term erosion (see Chapter 8).
- Determine the site elevation and determine design stillwater depths (see Chapter 8).
- Determine flood loads including breaking wave loads, hydrodynamic loads, flood-borne debris loads, and hydrostatic loads. Buoyancy reduces the weight of all submerged materials, so hydrostatic loads need to be considered on all foundations (see Chapter 8).
- Obtain adequate soils data for the site (see Section 10.3.3).
- Determine maximum scour and erosion depths (see Chapter 8).
- Select foundation type (open/deep, open/shallow, closed/deep, or closed/shallow). Use open/deep foundations in Zone V and Coastal A Zone. Use open/shallow foundations in Coastal A Zone only when scour and erosion depths can be accurately predicted and when the foundation can extend beneath the erosion depths. See Sections 10.2 and 10.3.1.
- Determine the basic wind speed, exposure, and wind pressures (see Chapter 8). Determine live and dead loads and calculate all design loads on the elevated building and on the foundation elements (see Chapter 8).

- Determine forces and moments at the top of the foundation elements for all load cases specified in ASCE 7-10. Use load combinations specified in Section 2.3 for strength-based designs or Section 2.4 for stress-based designs. Apply forces and moments to the foundation.

- Design the foundation to resist all design loads and load combinations when exposed to maximum predicted scour and erosion.

10.5 Pile Foundations

Pile foundations are widely used in coastal environments and offer several benefits. Pile foundations are deep and, when properly imbedded, offer resistance to scour and erosion. Piles are often constructed of treated timber, concrete, or steel although other materials are also used.

Treated timber piles are readily available and because they are wood, they can be cut, sawn, and drilled with standard construction tools used for wood framing. ASTM D25-99 contains specifications on round timber piles including quality requirements, straightness, lengths and sizes (circumferences and diameters) as well as limitations on checks, shakes, and knots. The *National Design Specification for Wood Construction (*ANSI/AF&PA 2005) contains design values for timber piles that meet ASTM D25-99 specifications.

Pre-cast (and typically pre-stressed) concrete piles are not readily available in some areas but offer several benefits over treated timber piles. Generally, they can be fabricated in longer lengths than timber piles. For the same cross section, they are stronger than timber piles and are not vulnerable to rot or wood-destroying insects. The strength of concrete piles can allow them to be used without grade beams. Foundations without grade beams are less vulnerable to scour than foundations that rely on grade beams (See Section 10.5.6).

Steel piles are generally not used in residential construction but are common in commercial construction. Field connections are relatively straightforward, and since steel can be field drilled and welded, steel-to-wood and steel-to-concrete connections can be readily constructed. ASTM A36/A36M-08 contains specifications for mild (36 kip/square inch) steels in cast or rolled shapes. ASTM standards for other shapes and steels include:

- For steel pipe, ASTM A53/A53M-10, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless* (ASTM 2010c)

- For structural steel tubing, ASTM A500-10, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes* (ASTM 2010b); and ASTM A501-07, *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing* (ASTM 2007)

- For welded and seamless steel pipe piles, ASTM 252-10, *Standard Specification for Welded and Seamless Steel Pipe Piles* (ASTM 2010d)

Fiber-reinforced polymer (FRP) piles are becoming more commonplace in transportation and marine infrastructure but are rarely used in residential applications. However, the usage of FRP piles in residential applications is expected to increase. New construction materials can offer many benefits such as sustainability, durability, and longevity but like any new construction material, the appropriateness of FRP piles should be thoroughly investigated before being used in new applications. Although FRP is not discussed in the publication, Technical Fact Sheet 1.8, *Non-Traditional Building Materials and Systems,* in FEMA P-499 provides guidance on using new materials and new systems in coastal environments.

Table 10-3 is a summary of the advantages and special considerations for three of the more common pile materials.

Table 10-3. Advantages and Special Considerations of Three Types of Pile Materials

Material	Advantages	Special Considerations
Wood	• Comparatively low initial cost • Readily available in most areas • Easy to cut, saw and drill • Permanently submerged piles resistant to decay • Relatively easy to drive in soft soil • Suitable for friction and end bearing pile	• Difficult to splice • Subject to eventual decay when in soil or intermittently submerged in water • Vulnerable to damage from driving (splitting) • Comparatively low compressive load • Relatively low allowable bending stress
Concrete	• Available in longer lengths than wood piles • Corrosion resistant • Can be driven through some types of hard material • Suitable for friction and end-bearing piles • Reinforced piles have high bending strength • High bending strength allows taller or more heavily loaded pile foundations to be constructed without grade beams	• High initial cost • Not available in all areas • Difficult to make field adjustments for connections • Because of higher weight, require special consideration in high seismic areas
Steel	• High resistance to bending • Easy to splice • Available in many lengths, sections, and sizes • Can be driven through hard subsurface material • Suitable for friction and end-bearing piles • High bending strength, which allows taller or more heavily loaded pile foundations to be constructed without grade beams	• Vulnerable to corrosion • May be permanently deformed if struck by heavy object • High initial cost • Some difficulty with attaching wood framing

The critical aspects of pile foundations include the pile material and size and pile embedment depth. Pile foundations with inadequate embedment do not have the structural capacity to resist sliding and overturning (see Figure 10-2). Inadequate embedment and improperly sized piles greatly increase the probability for structural collapse. However, when properly sized, installed, and braced with adequate embedment into the soil (with consideration for erosion and scour effects), a building's pile foundation performance allows the building to remain standing and intact following a design flood event (see Figure 10-3).

10.5.1 Compression Capacity of Piles – Resistance to Gravity Loads

The compression capacity of piles determines their ability to resist gravity loads from the elevated structure they support. One source that provides an equation for the compression capacity of piles is the *Foundation and Earth Structures*, Design Manual 7.2 (USDN 1986). The manual contains Equation 10.2 for determining

Figure 10-2. Near collapse due to insufficient pile embedment, Hurricane Katrina (Dauphin Island, AL, 2005)

Figure 10-3. Surviving pile foundation, Hurricane Katrina (Dauphin Island, AL, 2005)

the compression capacity of a single pile when placed in granular (non-cohesive) soils. Design Manual 7.2 also contains methods of determining compression capacity of a pile placed in cohesive soils.

The resistance of the pile is the sum of the capacity that results from end bearing and friction. The capacity from end bearing is the first term in Equation 10.2; the capacity from friction is given in the second term.

Equation 10.2 gives the ultimate compression capacity of a pile. The allowable capacity (Q_{allow}) used in ASD depends on a Factor of Safety applied to the ultimate capacity. For ASD, Design Manual 7.2 recommends a Factor of Safety of 3.0; thus, $Q_{allow} = Q_{ult}/3$.

EQUATION 10.2. ULTIMATE COMPRESSION CAPACITY OF A SINGLE PILE

$$Q_{ult} = P_T N_q A_T + \sum K_{HC} P_0 Ds \tan\delta$$

where:

Q_{ult} = ultimate load capacity in compression (lb)

P_T = effective vertical stress at pile tip (lb/ft^2)

N_q = bearing capacity factor (see Table 10-4)

A_T = area of pile tip (ft^2)

K_{HC} = earth pressure in compression (see Table 10-5)

P_0 = effective vertical stress over the depth of embedment, D (lb/ft^2)

δ = friction angle between pile and soil (see Table 10-6)

s = surface area of pile per unit length (ft)

D = depth of embedment (ft)

Table 10-4. Bearing Capacity Factors (N_q)

Parameter	Pile Bearing Capacity Factors												
φ (degrees)[(a)]	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (driven pile displacement)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q (drilled piers)[(b)]	5	8	10	12	14	17	21	25	30	38	43	60	72

N_q = bearing capacity factor

φ = angle of internal friction

(a) Limit φ to 28° if jetting is used

(b) When a bailer or grab bucket is used below the groundwater table, calculate end bearing based on φ not exceeding 28 degrees. For piers larger than 24 inches in diameter, settlement rather than bearing capacity usually controls the design. For estimating settlement, take 50% of the settlement for an equivalent footing resting on the surface of comparable granular soils.

Table 10-5. Earth Pressure Coefficients

Pile Type	K_{HC}	K_{HT}
Driven single H-pile	0.5 – 1.0	0.3 – 0.5
Driven single displacement pile	1.0 – 1.5	0.6 – 1.0
Driven single displacement tapered pile	1.5 – 2.0	1.0 – 1.3
Driven jetted pile	0.4 – 0.9	0.3 – 0.6
Drilled pile (less than 24-inch diameter)	0.7	0.4

K_{HC} = earth pressure compression coefficient

K_{HT} = earth pressure tension coefficient

Table 10-6. Friction Angle Between Soil and Pile (δ)

Pile Type	δ
Timber	$\frac{3}{4}\varphi$
Concrete	$\frac{3}{4}\varphi$
Steel	20 degrees

φ = angle of internal friction

10.5.2 Tension Capacity of Piles

The tension capacity of piles determines their ability to resist uplift and overturning loads on the elevated structure. One source that provides pile capacity in tension load is the Design Manual 7.2, which is also a reference on compression capacity. Equation 10.3 determines the tension capacity in a single pile.

EQUATION 10.3. ULTIMATE TENSION CAPACITY OF A SINGLE PILE

$$T_{ult} = \sum K_{HT} P_0 D s \tan\delta$$

where:

T_{ult} = ultimate load capacity in tension (lb)

K_{HT} = earth pressure in tension (see Table 10-5)

P_0 = effective vertical stress over the depth of embedment, D (lb/ft^2)

δ = friction angle between pile and soil (see Table 10-6)

s = surface area of pile per unit length (ft^2/ft or ft)

D = depth of embedment (ft)

Note: *With the recommended Factor of Safety of 3.0, the allowable tension capacity,* $T_{allow} = T_{ult}/3$.

The Design Manual 7.2 provides tables to identify bearing capacity factors (N_q), earth pressure coefficients (K_{HC} and K_{HT}), and friction angle between pile and soil (δ) based on pile type and the angle of internal friction (φ) of the soil.

Example 10.1 illustrates compression and tension capacity calculations for a single pile not affected by scour or erosion.

Table 10-7 contains example calculations using Equations 10.2 and 10.3 for the allowable compression (gravity loading) and tension (uplift) capacities of wood piles for varying embedments, pile diameters, and installation methods. The table also illustrates the effect of scour around the pile on the allowable compression and tension loads. Scour (and erosion) reduces pile embedment and therefore pile capacity. For this table, a scour depth of twice the pile diameter ($2d$) with no generalized erosion is considered.

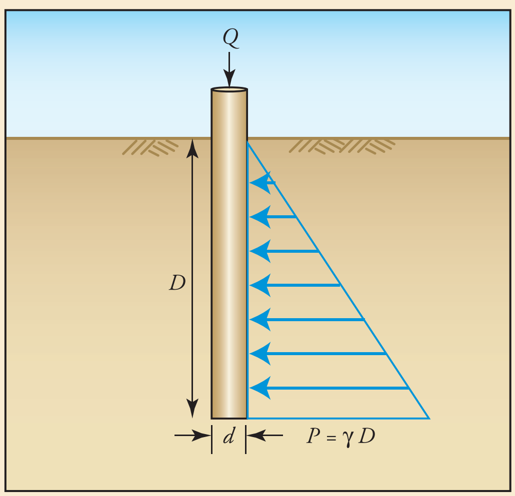
EXAMPLE 10.1. CALCULATION FOR ALLOWABLE CAPACITIES OF WOOD PILES

Given:

- Closed end, driven timber pile
- Diameter *(d)* = 1 ft
- Depth of embedment *(D)* = 15 ft
- Soil density (γ) = 65 lb/ft^3
- Angle of internal friction (φ) = 30 K_{HC} = 1.0 (applicable coefficient from Table 10-5)
- Earth pressure in tension (K_{HT}) = 0.6 (applicable coefficient from Table 10-5)
- Bearing capacity factor (N_q) = 21 (applicable coefficient from Table 10-4)
- Factor of Safety = 3.0

Find:

1. Allowable tension and compression capacities of wood piles embedded in soil



Q = load
D = length of pile
d = diameter of pile
P = pressure
γ = soil density

Illustration A.
Pile schematic and pressure diagram

EXAMPLE 10.1. CALCULATION FOR ALLOWABLE CAPACITIES OF WOOD PILES (concluded)

Solution for #1: Find the allowable tension and compression capacity of the wood pile embedded in soil as follows:

- To determine the resultant pressure from the soil on the pile:

$$\delta = \frac{3}{4}(\varphi) = \frac{3}{4}(30^\circ) = 22.5^\circ$$

$$P_0 = P_t = \gamma D = (65\ \text{lb/ft}^3)(15\ \text{ft}) = 975\ \text{lb/ft}^2$$

- Geometrical properties of the pile surfaces upon which pressure from the soil is applied to the pile are:

$$A_t = (\pi)(\frac{1}{2}d)^2 = (3.14)[(0.5)(1\ \text{ft})]^2 = 0.785\ \text{ft}^2$$

$$P_0 = P_t = \gamma D = (65\ \text{lb/ft}^3)(15\ \text{ft}) = 975\ \text{lb/ft}^2$$

Allowable compression capacity:

$$Q_{ult} = (975\ \text{lb / ft}^2)(21)(0.785\ \text{ft}^2) + (1.0)(975\ \text{lb/ft}^2)(\tan 22.5^\circ)(3.14\ \text{ft}^2/\text{ft})(15\ \text{ft})$$

$$Q_{ult} = 35{,}095\ \text{lb}$$

$$Q_{all} = \frac{Q_{ult}}{3} = \frac{35{,}095\ \text{lb}}{3} = \mathbf{11{,}698\ lb}$$

Allowable tension capacity:

$$T_{ult} = (0.6)(975\ \text{lb/ft}^2)(\tan 22.5^\circ)(3.14\ \text{ft}^2/\text{ft})(15\ \text{ft}) = 11{,}413\ \text{lb}$$

$$T_{all} = \frac{T_{ult}}{3} = \frac{11{,}413\ \text{lb}}{3} = \mathbf{3{,}804\ lb}$$

The purpose of Table 10-7 is to illustrate the effects of varying diameters, depths of embedment, and installation methods on allowable capacities. See Section 10.5.4 for information of installation methods. Example calculations used to determine the values in Table 10-7 are used in Example 10.1. The values in Table 10-7 are not intended to be used for design purposes.

Table 10-7. Allowable Compression and Tension of Wood Piles Based on Varying Diameters, Embedments, and Installation Methods

Diameter and Embedment	Installation Method	Compression (pounds)		Tension (pounds)	
		No Scour	2d Scour	No Scour	2d Scour
d = 12 inches *D* = 15 feet	Driven	11,698	9,406	3,804	2,857
	Jetted	7,894	6,548	1,902	1,429
	Augered	6,990	5,545	2,536	1,905
d = 12 inches *D* = 20 feet	Driven	18,416	15,560	6,763	5,478
	Jetted	11,652	10,081	3,382	2,739
	Augered	11,292	9,453	4,509	3,652
d =10 inches *D* = 15 feet	Driven	9,004	7,482	3,170	2,505
	Jetted	5,834	4,977	1,585	1,252
	Augered	5,470	4,497	2,114	1,670

d = diameter
D = depth of embedment

10.5.3 Lateral Capacity of Piles

The lateral capacity of piles is dictated by the piles and the pile/soil interface. The ability of the pile to resist lateral loads depends on the pile size and material, the soil properties, and on presence or absence of pile bracing.

One of the critical aspects of pile design is the distance between the lateral load application point and the point of fixity of the pile. That distance constitutes a moment arm and governs how much bending moment develops when a pile is exposed to lateral loads. For a foundation to perform adequately, that moment must be resisted by the pile without pile failure.

Equation 10.4 determines the distance between the point where the lateral load is applied and the point of fixity for an unbraced pile. Note that in Equation 10.4, "*d*" is the depth below grade of the point of fixity, not the diameter of the pile. Also, see Figure 10-4 for the deflected shape of a laterally loaded pile.

Table 10-8 lists recommended values for n_h, modulus of subgrade reaction, for a variety of soils (Bowles 1996). For wood pilings, the depths to points of fixity range from approximately 1 foot in stiff soils to approximately 5 feet in soft soils.

The ability of site soils to resist lateral loads is a function of the soil characteristics, their location on the site, and their compressive strength. Chapter 7 of the *Timber Pile Design and Construction Manual* (Collin 2002) contains methods of determining the lateral resistance of timber piles for both fixed pile head conditions (i.e., piles used with grade beams or pile caps) and free pile head conditions (i.e., piles free to rotate at their top). The manual also contains methods of approximating lateral capacity and predicting pile capacity when detailed soils data are known.

EQUATION 10.4. LOAD APPLICATION DISTANCE FOR AN UNBRACED PILE

$$L = H + \frac{d}{12}$$

where:

L = distance between the location where the lateral force in applied and the point of fixity (i.e., moment arm) (ft)

d = depth from grade to inflection point (inches); $d = 1.8\left(\frac{EI}{n_h}\right)^{\frac{1}{5}}$

H = distance above eroded ground surface (including localized scour) where lateral load is applied (ft)

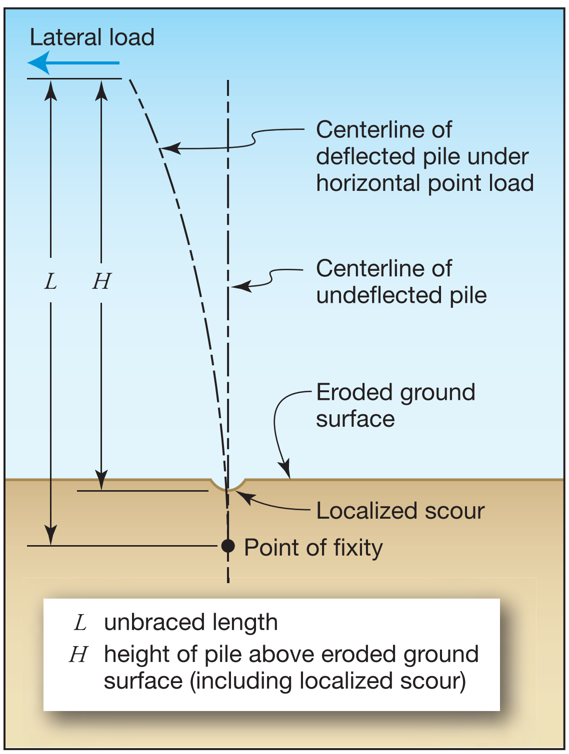


Figure 10-4.
Deflected pile shape for an unbraced pile

Table 10-8. Values of n_h Modulus of Subgrade Reaction

Soil Type	n_h Modulus of Subgrade Reaction (pound/cubic inch)
Dense sandy gravel	800 to 1,400
Medium dense coarse sand	600 to 1,200
Medium sand	400 to 1,000
Fine to silty fine sand	290 to 700
Medium clay (wet)	150 to 500
Soft clay	6 to 150

10.5.4 Pile Installation

Methods for installing piles include driving, augering, and jetting. A combination of methods may also be used. For example, piles may be placed in augered holes and then driven to their final depth. Combining installation methods can increase the achievable embedment depth. With increased depths, a pile's resistance to lateral and vertical loads can be increased, and its vulnerability to scour and erosion will be reduced.

- ***Driving*** involves hitting the top of the pile with a pile driver or hammer until the pile reaches the desired depth or it is driven to refusal. Piles can be driven with vibratory hammers. Vibratory hammers generate vertical oscillating movements that reduce the soil stress against the pile and which makes the piles easier to drive. Ultimate load resistance is achieved by a combination of end bearing of the pile and frictional resistance between the pile and the soil. A record of the blow counts from the pile driver can be used with a number of empirical equations to determine capacity.

- ***Augering*** involves placing the pile into a pre-drilled hole typically made with an auger. The augered hole can be the full diameter of the pile or a smaller diameter than the pile. Pre-drilling is completed to a predetermined depth, which often is adjusted for the soils found on the site. After placing the pile into the pre-drilled hole, the pile is then driven to its final desired depth or until it reaches refusal.

- ***Jetting*** is similar to augering but instead of using a soils auger, jetting involves using a jet of water (or air) to remove soils beneath and around the pile. Like augering, jetting is used in conjunction with pile driving.

Both augering and jetting remove natural, undisturbed soil along the side of the pile. Load resistance for both of these methods is achieved by a combination of end bearing and frictional resistance, although the frictional resistance is much less than that provided by driven piles.

Figure 10-5 illustrates the three pile installation methods. Table 10-9 lists advantages and special considerations for each method.

Figure 10-5.
Pier installation methods

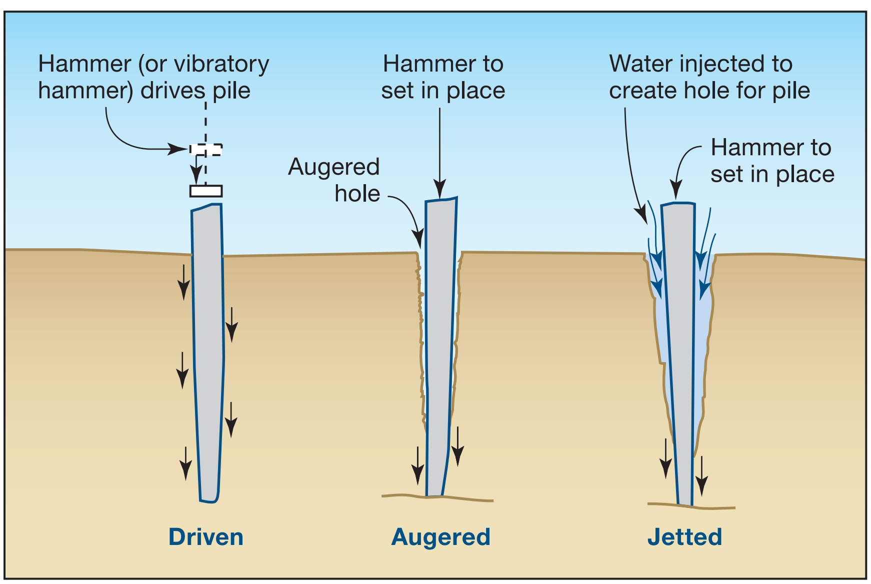


Table 10-9. Advantages and Special Considerations of Pile Installation Methods

Installation Method	Advantages	Special Considerations
Driving	• Well-suited for friction piles • Common construction practice • Pile capacity can be determined empirically	• Requires subsurface investigation • May be difficult to reach terminating soil strata if piles are only driven • Difficult to maintain plumb during driving and thus maintain column lines
Augering	• Economical • Minimal driving vibration to adjacent structures • Well-suited for end bearing • Visual inspection of some soil stratum possible • Convenient for low headroom situations • Easier to maintain column lines	• Requires subsurface investigation • Not suitable for highly compressed material • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients K_{HC} and K_{HT} to 40 percent of that driven for piles • Capacity must be determined by engineering judgment or load test
Jetting	• Minimal driving vibration to adjacent structures • Well-suited for end bearing piles • Easier to maintain column lines	• Requires subsurface investigation • Disturbs soil adjacent to pile, thus reducing earth pressure coefficients K_{HC} and K_{HT} to 40 percent of that driven for piles • Capacity must be determined by engineering judgment or load test

K_{HC} = earth pressure compression coefficient
K_{HT} = earth pressure tension coefficient

10.5.5 Scour and Erosion Effects on Pile Foundations

Coastal homes are often exposed to scour and erosion, and because moving floodwaters cause both scour and erosion, it is rare for an event to produce one and not the other. As Figure 10-6 illustrates, scour and erosion have a cumulative effect on pile foundations. They both reduce piling embedment.

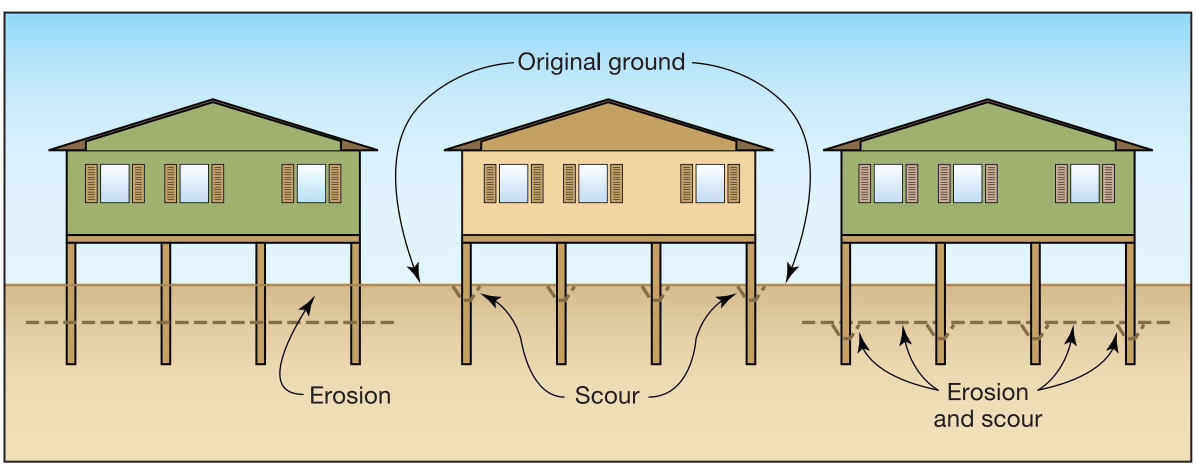


Figure 10-6.
Scour and erosion effects on piling embedment

A properly designed pile foundation must include a consideration of the effects of scour and erosion on the foundation system. Scour washes away soils around the piling, reducing pile embedment, and increases stresses within the pile when the pile is loaded. The reduced embedment can cause the foundation to fail at the pile/soil interface. The increased stresses can cause the pile itself to fracture and fail.

Erosion is even more damaging. In addition to reducing pile embedment depths and increasing stresses on piles, erosion increases the flood forces the foundation must resist by increasing the stillwater depth at the foundation that the flood produces. Pile foundations that are adequate to resist flood and wind forces without being undermined by scour and erosion can fail when exposed to even minor amounts of scour and erosion.

An example analysis of the effects of scour and erosion on a foundation is provided in *Erosion, Scour, and Foundation Design* (FEMA 2009a), published as part of Hurricane Ike Recovery Advisories and available at http://www.fema.gov/library/viewRecord.do?id=3539.

The structure in the example is a two-story house with 10-foot story heights and a 32-foot by 32-foot foundation. The house is away from the shoreline and elevated 8 feet above grade on 25 square timber piles spaced 8 feet apart. Soils are medium dense sands. The house is subjected to a design wind event with a 130-mph (3-second gust) wind speed and a 4-foot stillwater depth above the uneroded grade, with storm surge and broken waves passing under the elevated building.

Lateral wind and flood loads were calculated in accordance with ASCE 7-05. Although the wind loads in ASCE 7-10 vary from ASCE 7-05 somewhat, the results of the analyses do not change significantly. Piles were analyzed under lateral wind and flood loads only; dead, live, and wind uplift loads were neglected. If the neglected loads are included, deeper pile embedment and possibly larger piles than the results of the analysis indicated may be needed. Three timber pile sizes (8-inch square, 10-inch square, and 12-inch square) were evaluated using pre-storm embedment depths of 10 feet, 15 feet, and 20 feet and five erosion and scour conditions (erosion = 0 or 1 foot; scour = 2.0 times the pile diameter to 4.0 times the pile diameter).

The results of the analysis are shown in Table 10-10. A shaded cell indicates that the combination of pile size, pre-storm embedment, and erosion/scour does not provide the bending resistance and/or embedment required to resist lateral loads. The reason for foundation failure is indicated in each shaded cell ("P" for failure due to bending and overstress within the pile and "E" for an embedment failure from the pile/soil interaction). "OK" indicates that the bending and foundation embedment criteria are both satisfied by the particular pile size/pile embedment/erosion-scour combination.

The key points from the example analysis are as follows:

- Scour and erosion can cause pile foundations to fail and must be considered when designing pile foundations.
- Failures can result from either overloading the pile itself or from overloading at the pile/soil interface.
- Increasing a pile's embedment depth does not offset a pile with a cross section that is too small or pile material that is too weak.
- Increasing a pile's cross section (or its material strength) does not compensate for inadequate pile embedment.

Table 10-10. Example Analysis of the Effects of Scour and Erosion on a Foundation

Pile Embedment Before Erosion and Scour	Erosion and Scour Conditions	Pile Diameter (a) 8 inches	10 inches	12 inches
		Reason for Failure		
10 feet	Erosion = 0, Scour = 0	P, E	E	OK
	Erosion = 1 foot, Scour = 2.0a	P, E	E	E
	Erosion = 1 foot, Scour = 2.5a	P, E	E	E
	Erosion = 1 foot, Scour = 3.0a	P, E	E	E
	Erosion = 1 foot, Scour = 4.0a	P, E	P, E	E
15 feet	Erosion = 0, Scour = 0	P	OK	OK
	Erosion = 1 foot, Scour = 2.0a	P	OK	OK
	Erosion = 1 foot, Scour = 2.5a	P	OK	OK
	Erosion = 1 foot, Scour = 3.0a	P	OK	OK
	Erosion = 1 foot, Scour = 4.0a	P, E	P, E	E
20 feet	Erosion = 0, Scour = 0	P	OK	OK
	Erosion = 1 foot, Scour = 2.0a	P	OK	OK
	Erosion = 1 foot, Scour = 2.5a	P	OK	OK
	Erosion = 1 foot, Scour = 3.0a	P	OK	OK
	Erosion = 1 foot, Scour = 4.0a	P	P	OK

Two-story house supported on square timber piles and located away from the shoreline, storm surge and broken waves passing under the building, 130-mph wind zone, soil = medium dense sand.

a = pile diameter

E = foundation fails to meet embedment requirements

OK = bending and foundation embedment criteria are both satisfied by the particular pile size/pile embedment/erosion-scour combination

P = foundation fails to meet bending

10.5.6 Grade Beams for Pile Foundations

Piles can be used with or without grade beams or pile caps. Grade beams create resistance to rotation (also called "fixity") at the top of the piles and provide a method to accommodate misalignment in piling placement. When used with grade beams, the piles and foundation elements above the grade beams work together to elevate the structure, provide vertical and lateral support for the elevated home, and transfer loads imposed on the elevated home and the foundation to the ground below.

Pile and grade beam foundations should be designed and constructed so that the grade beams act only to provide fixity to the foundation system and not to support the lowest elevated floor. If grade beams support the lowest elevated floor of the home, they become the lowest horizontal structural member and significantly higher flood insurance premiums would result. Grade beams must also be designed to span between adjacent piles, and the piles must be capable of resisting both the weight of the grade beams when undermined by erosion and scour and the loads imposed on them by forces acting on the structure.

Pile foundations with grade beams must be constructed with adequate strength to resist all lateral and vertical loads. Failures during Hurricane Katrina often resulted from inadequate connections between the columns and footings or grade beams below (see Figure 10-7).

If grade beams are used with wood piles, the potential for rot must be considered when designing the connection between the grade beam and the pile. The connection must not encourage water retention. The maximum bending moment in the piles occurs at the grade beams, and decay caused by water retention at critical points in the piles could induce failure under high-wind or flood forces.

While offering some advantages, grade beams can become exposed by moving floodwaters if they are not placed deeply enough. Once exposed, the grade beams create large horizontal obstructions in the flood path that significantly increase scour. Extensive scour was observed after Hurricane Ike in 2008 around scores of homes constructed with grade beams (see Figure 10-8).

Although not possible for all piling materials, foundations should be constructed without grade beams whenever possible. For treated timber piles, this can limit elevations to approximately 8 feet above grade. The actual limit depends greatly on flood forces, number of piles, availability of piles long enough to be driven to the required depth and extend above grade enough to adequately elevate the home, and wind speed and geometry of the elevated structure. For steel and concrete piles, foundations without grade beams are practical in many instances, even for taller foundations. Without grade beams to account for pile placement, additional attention is needed for piling alignment, and soils test are needed for design because pile performance depends on the soils present, and presumptive piling capacities may not adequately predict pile performance.

Figure 10-7.
Column connection failure, Hurricane Katrina (Belle Fontaine Point, Jackson County, MS, 2005)

Figure 10-8.
Scour around grade beam, Hurricane Ike (Galveston Island, TX, 2008)

10.6 Open/Deep Foundations

In this section, some of the more common types of open/deep foundation styles are discussed. Treated timber pile foundations are discussed in Section 10.6.1, and other types of open/deep pile foundations are discussed in Section 10.6.2.

10.6.1 Treated Timber Pile Foundations

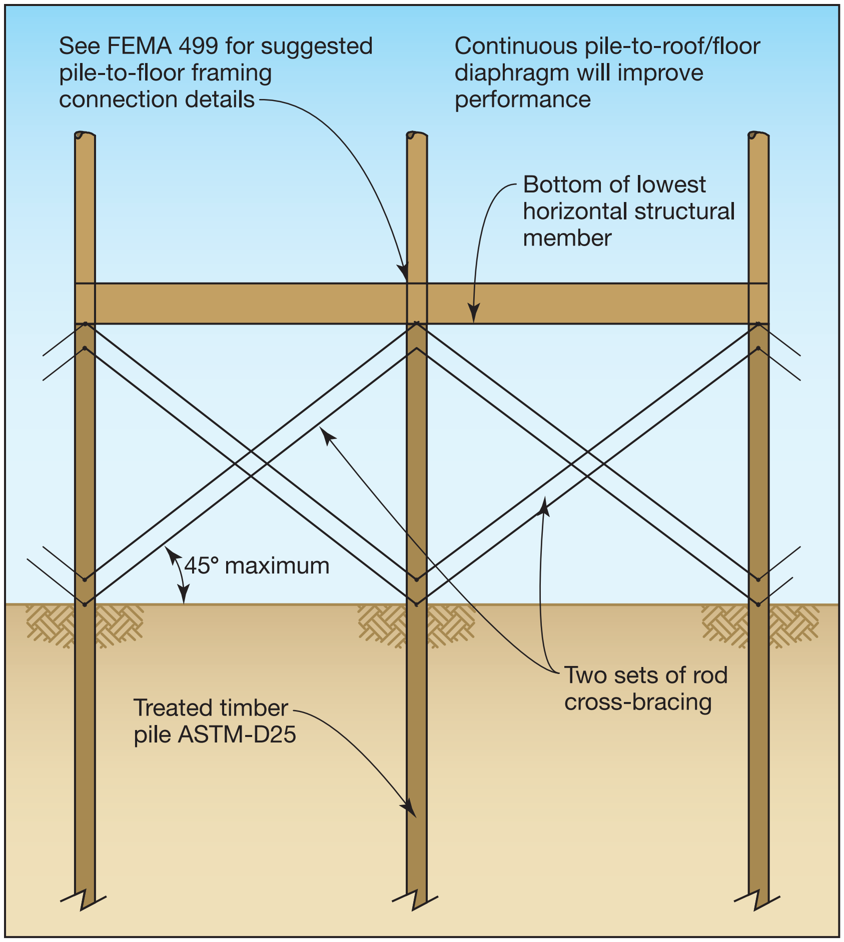
In many coastal areas, treated timber piles are the most common type of an open/deep style foundation. Timber piles are the first choice of many builders because they are relatively inexpensive, readily available, and relatively easy to install. The driven timber pile system (see Figure 10-9) is suitable for moderate elevations. Home elevations greater than 10 feet may not be practical because of pile length availability, the pile strength required to resist lateral forces (particularly when considering erosion and scour), and the pile embedment required to resist lateral loads after being undermined by scour and erosion.

When used without grade beams, timber piles typically extend from the pile tip to the lowest floor of the elevated structure. With timber piles and wood floor framing, the connection of the elevated structure to the piling is essentially a pinned connection because moment resisting connections in wood framing are difficult to achieve. Pinned connections do not provide fixity and require stronger piles to resist the same loads as piles that benefit from moment resisting connections at their tops.

Improved performance can be achieved if the piles extend beyond the lowest floor to the roof (or an upper floor level). Doing so provides resistance to rotation where the pile passes through the first floor. This not only reduces stresses within the piles but also increases the stiffness of the pile foundation and reduces movement under lateral forces. Extending piles in this fashion improves survivability of the building.

The timber pile system is vulnerable to flood-borne debris. During a hurricane event, individual piles can be damaged or destroyed by large, floating debris. Two ways of reducing this vulnerability are (1) using piles with diameters that are larger than those called for in the foundation design and (2) using more piles and continuous beams that can redistribute loads around a damaged pile. Using more piles and continuous beams increases structural redundancy and can improve building performance.

Figure 10-9.
Profile of timber pile foundation type



FEMA P-550, *Recommended Residential Construction for Coastal Areas* (FEMA 2006), contains a foundation design using driven timber piles. The foundation design is based on presumptive piling capacities that should be verified prior to construction. Also, the design is intended to support an elevated building with a wide range of widths and roof slopes and as such contains some inherent conservatism in the design. Design professionals who develop foundation designs for specific buildings and have site information on subsurface conditions can augment the FEMA P-550 design to provide more efficient designs that reduce construction costs.

10.6.1.1 Wood Pile-to-Beam Connections

In pile foundations that support wood-framed structures, systems of perimeter and interior beams are needed to support the floors and walls above. Beams must be sized to support gravity loads and, in segmented shear wall construction, resist reactions from shear wall segments. To transfer those loads to the foundation, wood piles are often notched to provide a bearing surface for the beams. Notches should not reduce the pile cross section by more than 50 percent (such information is typically provided by a design professional on contract documents). For proper transfer of gravity loads, beams should bear on the surface of the pile notch.

Although connections play an integral role in the design of structures, they are typically regarded as the weakest link. Guidance for typical wood-pile to wood-girder connections can be found in Fact Sheet 3.3, *Wood Pile to Beam Connections*, in FEMA P-499.

10.6.1.2 Pile Bracing

When timber piles with a sufficiently large cross section are not available, timber piles may require bracing to resist lateral loads. Bracing increases the lateral stiffness of a pile foundation system so that less sway is felt under normal service loads. Bracing also lowers the location where lateral forces are applied to individual piles and reduces bending stresses in the pile. When bracing is used, the forces from moving floodwaters and from flood-borne debris that impacts the braces should be considered.

NOTE

Fact Sheet 3.2, *Pile Installation,* in FEMA P-499 recommends that pile bracing be used only for reducing the structure's sway and vibration for comfort. In other words, bracing should be used to address serviceability issues and not strength issues. The foundation design should consider the piles as being unbraced as the condition that may occur when floating debris removes or damages the bracing. If the pile foundation is not able to provide the desired strength performance without bracing, the designer should consider increasing the pile size.

Bracing is typically provided by diagonal bracing or knee bracing. Diagonal bracing is more effective from a structural standpoint, but because diagonal bracing extends lower into floodwaters, it is more likely to be damaged by flood-borne debris. It can also trap flood-borne debris, and trapped flood-borne debris increases flood forces on the foundation.

Knee bracing does not extend as deeply into floodwaters as cross bracing and is less likely to be affected by flood-borne debris but is less effective at reducing stresses in the pile and also typically requires much stronger connections to achieve similar structural performance as full-length cross bracing.

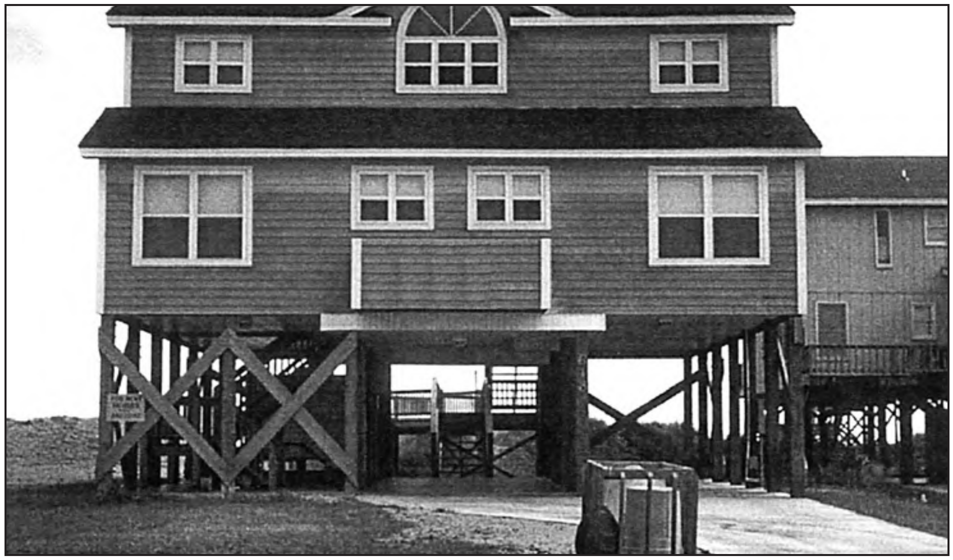
Diagonal Bracing

Diagonal bracing often consists of dimensional lumber that is nailed or bolted to the wood piles. Steel rod bracing and wire rope (cable) bracing can also be used. Steel rod bracing and cable bracing have the benefit of being able to use tensioning devices, such as turnbuckles, which allow the tension of the bracing to be maintained. Cable bracing has an additional benefit in that the cables can be wrapped around pilings without having to rely on bolted connections, and wrapped connections can transfer greater loads than bolted connections. Figure 10-10 shows an example of diagonal bracing using dimensional lumber.

Diagonal braces tend to be slender, and slender braces are vulnerable to compression buckling. Most bracing is therefore considered tension-only bracing. Because wind and flood loads can act in opposite directions, tension-only bracing must be installed in pairs. One set of braces resists loads from one direction, and the second set resists loads from the opposite direction. Figure 10-11 shows how tension-only bracing pairs resist lateral loads on a home.

The placement of the lower bolted connection of the diagonal brace to the pile requires some judgment. If the connection is too far above grade, the pile length below the connection is not braced and the overall foundation system is less strong and stiff.

Figure 10-10.
Diagonal bracing using dimensional lumber



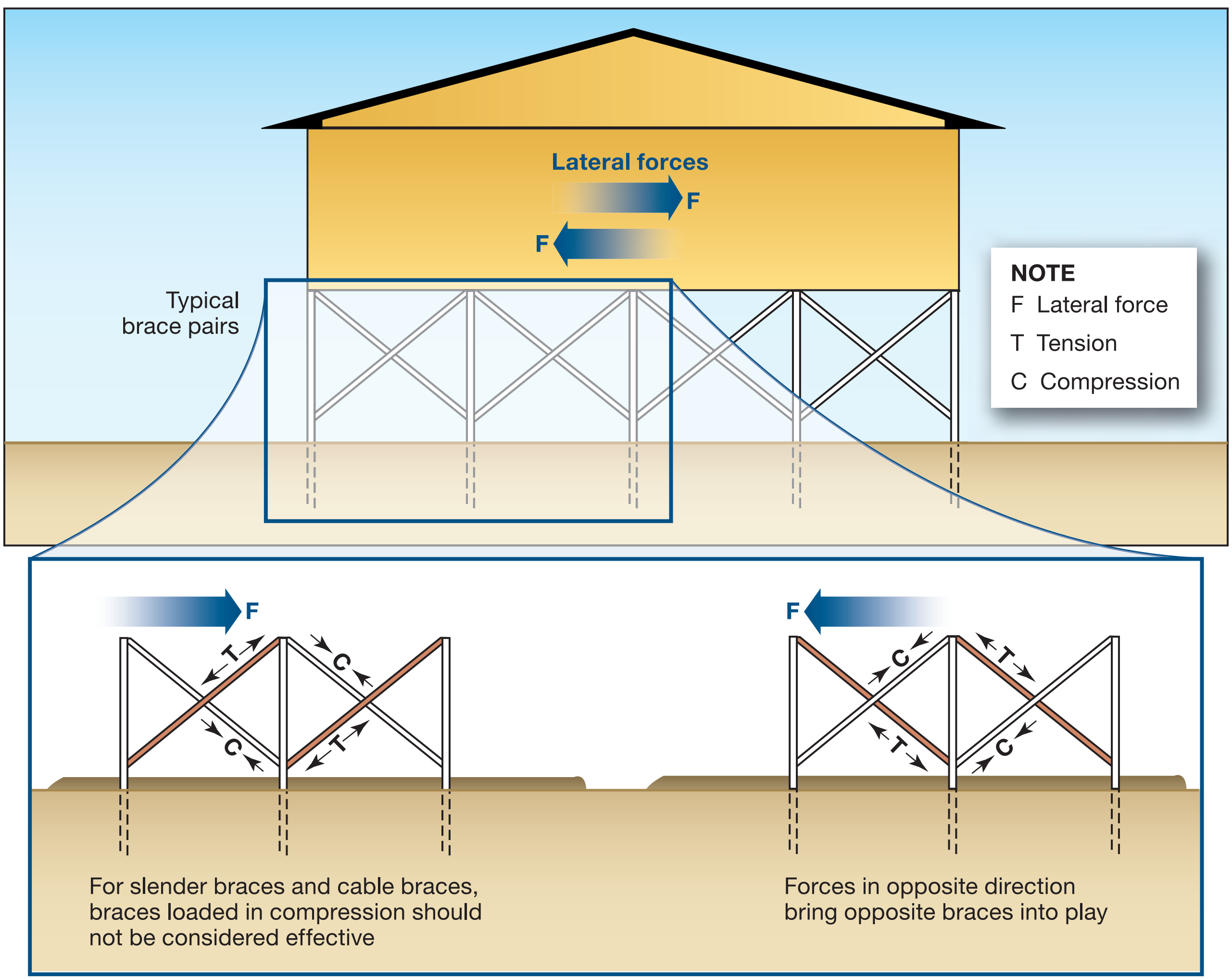


Figure 10-11.
Diagonal bracing schematic

EXAMPLE 10.2. DIAGONAL BRACE FORCE

Given:

- Lateral load = 989 lb
- Brace angle = 45

Find:

1. Tension force in the diagonal brace in Illustration A.

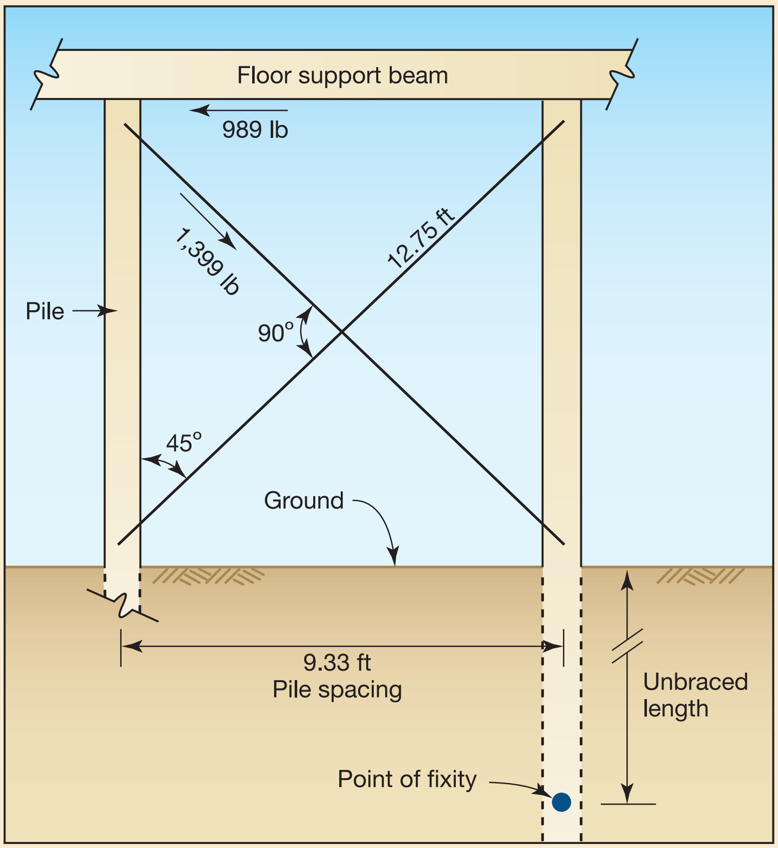


Illustration A. Force diagram for diagonal bracing

Solution for #1: The tension force in the diagonal brace can be found as follows:

Rod bracing is used and assumed to act in tension only because of the rigidity of the rod brace in tension and lack of stiffness of the rod in compression.

- The tension brace force is calculated as follows:

$$T_{diagonal} = \frac{989 \text{ lb}}{\cos 45^\circ} = \mathbf{1{,}399\ lb}$$

Interaction of the soil and the pile should be checked to ensure that the uplift component of the brace force can be resisted.

For timber piles, if the connection is too close to grade, the bolt hole is more likely to be flooded and subject to decay or termite infestation, which can weaken the pile at a vulnerable location. All bolt holes should be treated with preservative after drilling and prior to bolt placement.

NOTE

Bolt holes in timber piles should be field-treated (see Chapter 11).

Knee Bracing

Knee braces involve installing short diagonal braces between the upper portions of the pilings and the floor system of the elevated structure (see Figure 10-12). The braces increase the stiffness of an elevated pile foundation and can contribute to resisting lateral forces. Although knee braces do not stiffen a foundation as much as diagonal bracing, they offer some advantages over diagonal braces. For example, knee braces present less obstruction to waves and debris, are shorter and less prone to compression buckling than diagonal braces, and may be designed for both tension and compression loads.

The entire load path into and through the knee brace must be designed. The connections at each end of each knee brace must have sufficient capacity to handle both tension and compression and to resist axial loads in the brace. The brace itself must have sufficient cross-sectional area to resist compression and tensile loads.

Figure 10-12. Knee bracing

The feasibility of knee bracing is often governed by the ability to construct strong connections in the braces that connect the wood piles to the elevated structure.

10.6.1.3 Timber Pile Treatment

Although timber piles are chemically treated to resist rot and damage from insects, they can be vulnerable to wood-destroying organisms such as fungi and insects if the piles are subject to both wetted and dry conditions. If the piles are constantly submerged, fungal growth and insect colonies cannot be sustained; if only periodically submerged, conditions exist that are sufficient to sustain wood-destroying organisms. Local design professionals familiar with the performance of driven, treated timber piles can help quantify the risk. Grade beams can be constructed at greater depths or alternative pile materials can be selected if damage from wood-destroying organisms is a major concern.

Cutting, drilling, and notching treated timber piles disturb portions of the piles that have been treated for rot and insect damage. Because pressure-preservative-treated piles, timbers, and lumber are used for many purposes in coastal construction, the interior, untreated parts of the wood can be exposed to possible decay and infestation. Although treatments applied in the field are much less effective than factory treatments, the potential for decay can be minimized with field treatments. AWPA M4-06 describes field treatment procedures and field cutting restrictions for poles, piles, and sawn lumber.

Field application of preservatives should be done in accordance with the instructions on the label, but if instructions are not provided, dip soaking for at least 3 minutes is considered effective. When dip soaking for 3 minutes is impractical, treatment can be accomplished by thoroughly brushing or spraying the exposed area. The preservative is absorbed better at the end of a member or end grains than on the sides or side grains. To safeguard against decay in bored holes, the preservative should be poured into the holes. If the hole passes through a check (such as a shrinkage crack caused by drying), the hole should be brushed; otherwise, the preservative will run into the check instead of saturating the hole.

Copper naphthenate is the most widely used preservative for field treatment. Its color (deep green) may be objectionable aesthetically, but the wood can be painted with alkyd paints after extended drying. Zinc naphthenate is a clear alternative to copper naphthenate but is not as effective in preventing insect infestation and should not be painted with latex paints. Tributyltin oxide is available but should not be used in or near marine environments because the leachates are toxic to aquatic organisms. Sodium borate is also available, but it does not readily penetrate dry wood and rapidly leaches out when water is present. Sodium borate is therefore not recommended. Waterborne arsenicals, pentachlorophenol, and creosote are unacceptable for field applications.

10.6.2 Other Open/Deep Pile Foundation Styles

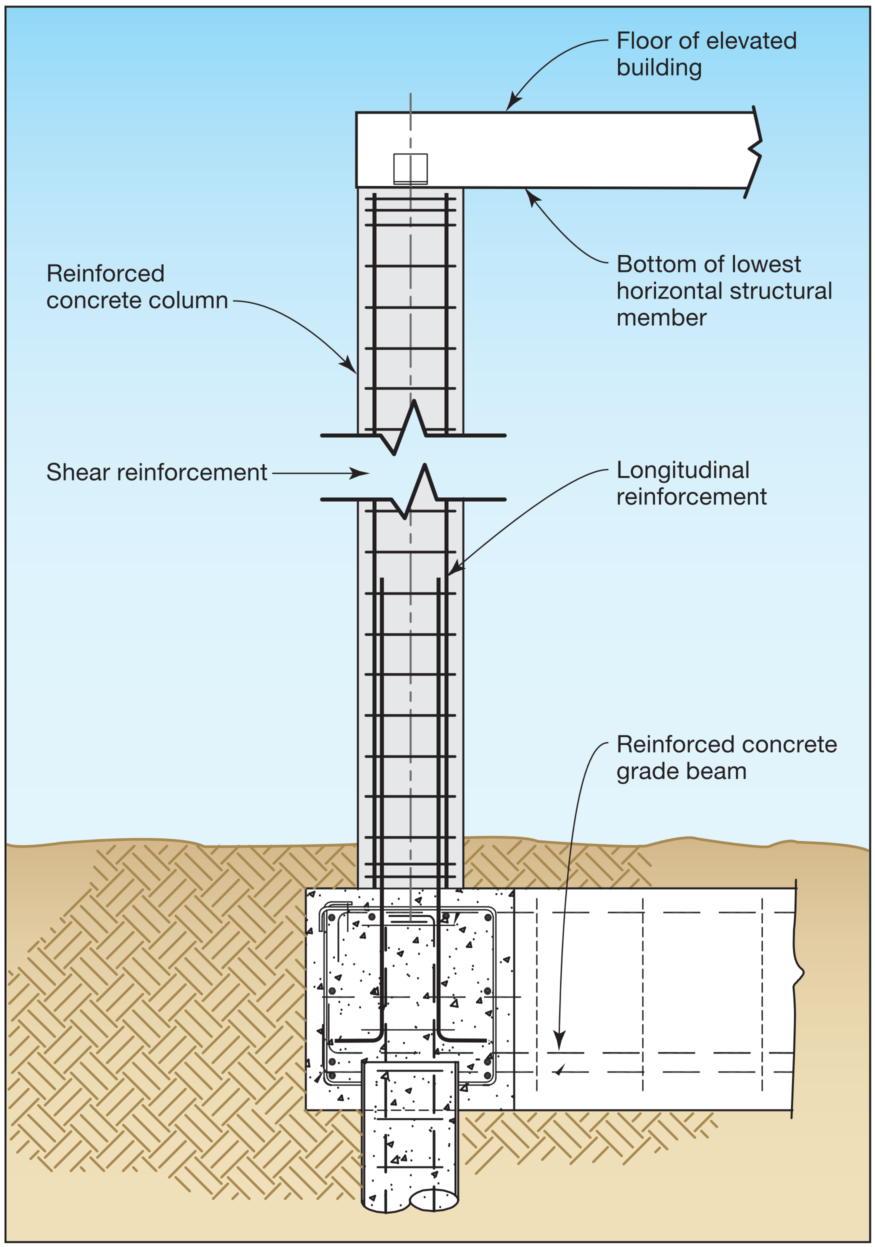
Several other styles of pile foundations, in addition to treated timber piles, are used although their use often varies geographically depending on the availability of materials and trained contractors.

FEMA P-550 contains foundation designs that use deep, driven steel and treated timber piles and grade beams that support a system of concrete columns. The second edition of FEMA P-550 (FEMA 2006) added a new design for treated timber piles that incorporates elevated reinforced beams constructed on the concrete columns. In the new design, the elevated beams work with the columns and grade beams to create reinforced concrete portal frames that assist in resisting lateral loads. The elevated beams also create a suitable platform that can support a home designed to a prescriptive standard such as *Wood Framed Construction Manual for One- and Two-Family Dwellings* (AF&PA 2012) or ICC 600-2008.

Figure 10-13 shows one of the deep pile foundation systems that uses treated timber piles and grade beams. The steel pipe pile and grade beam foundation system contained in FEMA P-550 is similar but requires fewer piles because the higher presumptive strength of the steel piles compared to the timber piles. Figure 10-14 shows the foundation system added in the Second Edition of FEMA P-550 (FEMA 2009b), which incorporates an elevated concrete beam.

Figure 10-13. Section view of a steel pipe pile with concrete column and grade beam foundation type

DEVELOPED FROM FEMA P-550, CASE B



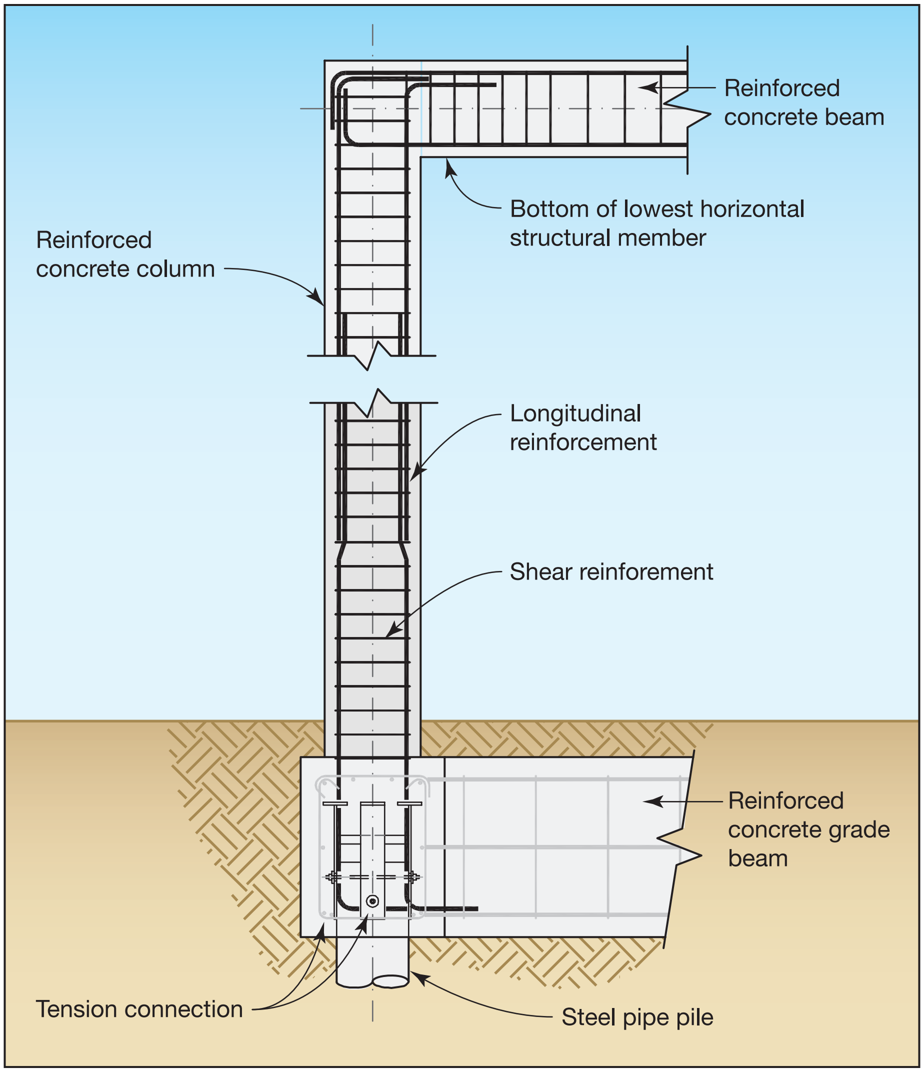


Figure 10-14. Section view of a foundation constructed with reinforced concrete beams and columns to create portal frames

SOURCE: ADAPTED FROM FEMA P-550, SECOND EDITION, CASE H

The grade beams that are shown in Figures 10-13 and 10-14 should not be used as structural support for a concrete slab that is below an elevated building in Zone V. Although a concrete slab may serve as the floor of a ground-level enclosure (usable only for parking, storage, or building access), the slab must be independent of the building foundation. If a grade beam is used to support the slab, the slab becomes the lowest floor of the building, the beam becomes the lowest horizontal structural member supporting the lowest floor, and the bottom of the beam becomes the reference elevation for flood insurance purposes. For buildings in Zone V, the NFIP, IBC and IRC require that the lowest floor elevated to or above the BFE be supported by the bottom of the lowest horizontal structural member. Keeping the slab from being considered the lowest floor requires keeping the slab and grade beams separate, which means the slab and grade beams cannot be monolithic or connected by reinforcing steel or other means.

Like the driven, treated pile foundation discussed in Section 10.6.1, the foundation designs discussed in this section are based on presumptive piling capacities that should be verified prior to construction. Also, design professionals who develop foundations designs for specific buildings and have site information on subsurface conditions can augment the FEMA P-550 design to provide more efficient designs that reduce construction costs.

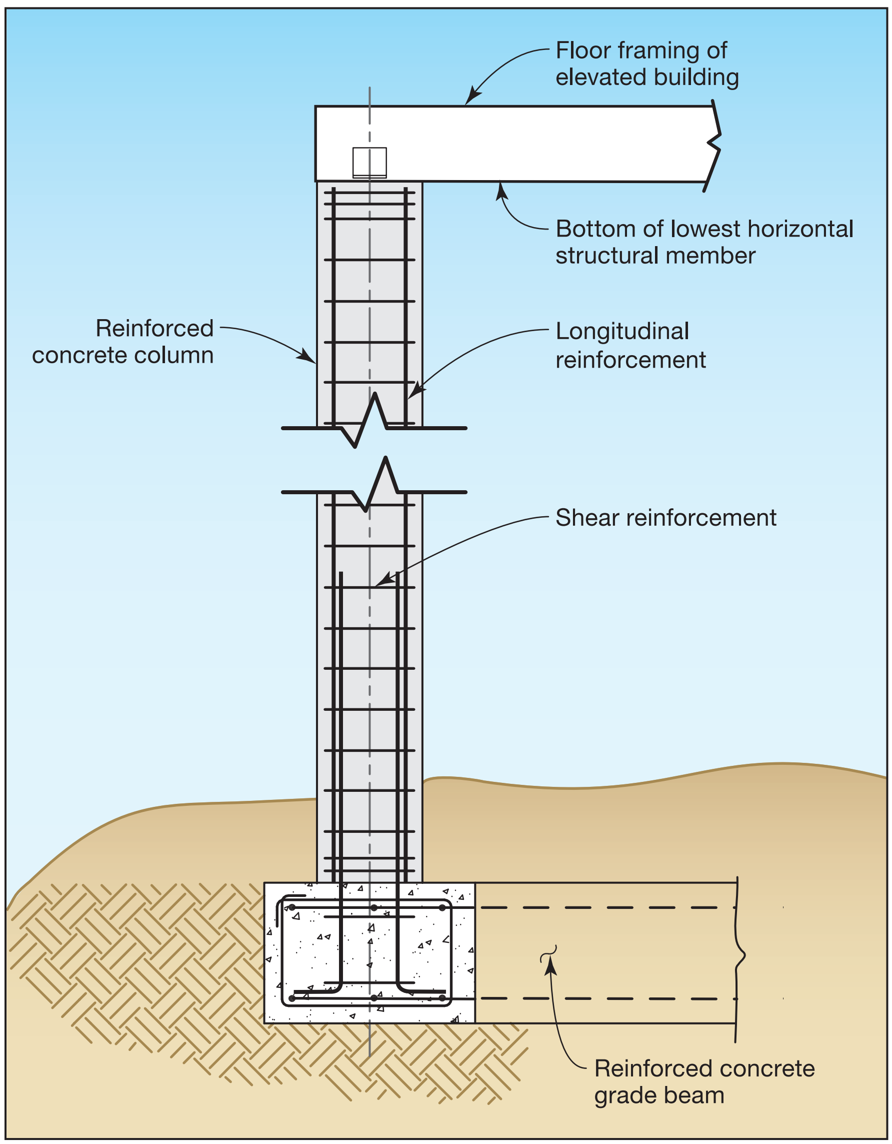
10.7 Open/Shallow Foundations

Open/shallow foundations are recommended for areas that are exposed to moving floodwaters and moderate wave actions but are not exposed to scour and erosion, which can undermine shallow footings. Open/shallow foundations are recommended for some riverine areas where an open foundation style is desirable and for buildings in Coastal A Zone where scour and erosion is limited.

In Coastal A Zones where the predicted scour and erosion depths extend below the achievable depth of shallow footings and in Coastal A Zone where scour and erosion potential is unknown or cannot be accurately predicted, open/deep foundations should be installed.

FEMA P-550 contains designs for open shallow foundations. The foundations are resistant to moving floodwaters and wave action, but because they are founded on shallow soils, they can be vulnerable to scour and erosion.

Figure 10-15. Profile of an open/shallow foundation
SOURCE: ADAPTED FROM FEMA P-550, CASE D



The FEMA P-550 designs make use of a rigid mat to resist lateral forces and overturning moment. Frictional resistance between the grade beams and the supporting soils resist lateral loads. The weight of the foundation and the elevated structure resist uplift forces. Because the foundation lacks the uplift resistance provided by piles, foundation elements often need to be relatively large to provide sufficient dead load to resist uplift, particularly when they are submerged. Grade beams need to be continuous because, as is shown in Section 10.9, discrete foundations that have sufficient capacity to resist lateral and uplift forces without overturning are difficult to design.

FEMA P-550 contains two types of open/shallow foundations. The foundation type shown in Figure 10-15 uses a matrix of grade beams and concrete columns to elevate the building. The grade beam shown in Figure 10-15 should not be used as structural support for a concrete slab that is below an elevated building in Zone V. If the grade beam is used to support the slab, the slab will be considered the lowest floor of the building, which will lead to the insurance ramifications described in Section 10.6.2.

When used to support wood framing, the columns of open/shallow foundations are typically designed as cantilevered beam/columns subjected to lateral forces, gravity forces and uplift forces from the elevated structure and flood forces on the foundation columns. Because of the inherent difficulty of creating moment connections with wood framing, the connections between the top of the columns and the bottom of the elevated structure are typically considered pinned. Maximum shear and moment occurs at the bottom of the columns, and proper reinforcement and detailing is needed in these areas. Also, because there are typically construction joints between the tops of the grade beams and the bases of the columns where salt-laden water can seep into the joints, special detailing is needed to prevent corrosion.

Designing an open/shallow foundation that uses concrete columns and elevated concrete beams can create a frame action that increases the foundation's ability to resist lateral loads. This design accomplishes two things. First, the frame action reduces the size of the columns and in turn reduces flood loads on them, and second, when properly designed, the elevated beams act like the tops of a perimeter foundation wall. Homes constructed to one of the designs contained in prescriptive codes can be attached to the elevated concrete beams with minimal custom design.

Unlike deep, driven-pile foundations, both types of open/shallow foundations can be undermined by erosion and scour. Neither foundation type should be used where erosion or scour is anticipated to expose the grade beam.

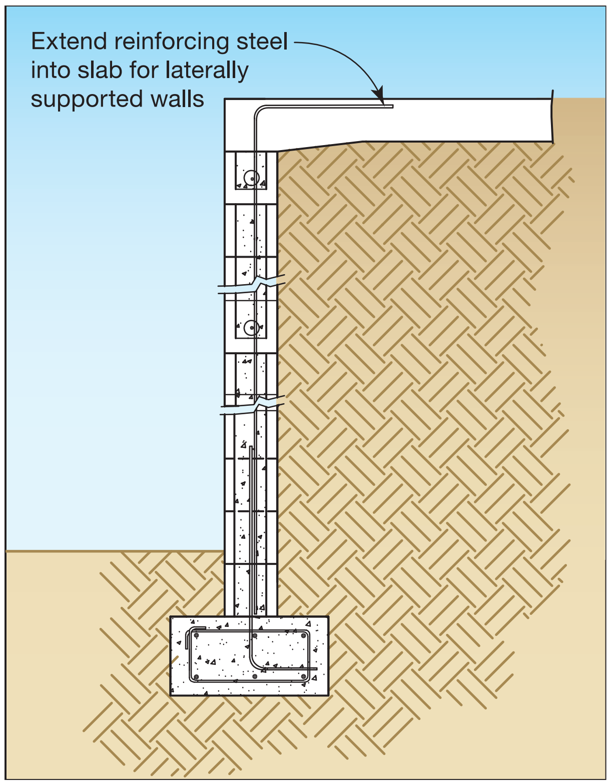
10.8 Closed/Shallow Foundations

Closed/shallow foundations are similar to the foundations that are used in non-coastal areas where flood forces are limited to slowly rising floods with no wave action and only limited flood velocities. In those areas, conventional foundation designs, many of which are included for residential construction in prescriptive codes and standards such as the 2012 IRC and ICC 600-2008, may be used. However, these codes and standards do not take into account forces from moving floodwaters and short breaking waves that can exist inland of Coastal A Zones. Therefore, caution should be used when using prescribed foundation designs in areas exposed to moving floodwaters and breaking waves.

FEMA P-550 contains two foundation designs for closed/shallow foundations: a stem wall foundation and a crawlspace foundation. Crawlspace foundation walls in SFHAs must be equipped with flood vents to equalize hydrostatic pressures on either side of the wall. See FEMA Technical Bulletin 1, O*penings in Foundation Walls and Walls of Enclosures* (FEMA 2008c). However, the flood vents do not significantly reduce hydrodynamic loads or breaking wave loads, and even with flood vents, flood forces in Coastal A Zones can damage or destroy these foundation styles.

Both closed/shallow foundations contained in FEMA P-550 are similar to foundations found in prescriptive codes but contain the additional reinforcement requirement to resist moving floodwaters and short (approximately 1.5-foot) breaking waves. Figure 10-16 shows the stem wall foundation design in FEMA P-550.

Figure 10-16.
Stem wall foundation design
SOURCE: ADAPTED FROM FEMA P-550, CASE F



10.9 Pier Foundations

Properly designed pier foundations offer the following benefits: (1) their open nature reduces the loads they must resist from moving floodwaters, (2) taller piers can often be constructed to provide additional protection without requiring a lot more reinforcement, and (3) the piers can be constructed with reinforced concrete and masonry materials commonly used in residential construction.

Pier foundations, however, can have drawbacks. If not properly designed and constructed, pier foundations lack the required strength and stability to resist loads from flood, wind or seismic events. Many pier foundation failures occurred when Hurricane Katrina struck the Gulf Coast in 2005.

The type of footing used in pier foundations greatly affects the foundation's performance (see Figure 10-17). When exposed to lateral loads, discrete footings can rotate so piers placed on discrete footings are suitable

Figure 10-17. Performance comparison of pier foundations: piers on discrete footings (foreground) failed by rotating and overturning while piers on more substantial footings (in this case a concrete mat) survived Hurricane Katrina (Pass Christian, MS, 2005)

only when wind and flood loads are relatively low. Piers placed on continuous concrete grade beams or concrete footings provide much greater resistance to lateral loads and are much less prone to failure. Footings and grade beams must be reinforced to resist the moment forces that develop at the base of the piers from the lateral loads on the foundation and the elevated home.

Like other open/shallow foundations, pier foundations are appropriate only where there is limited potential for erosion or scour. The maximum estimated depth for long- and short-term erosion and localized scour should not extend below the bottom of the footing or grade beam. In addition, adequate resistance to lateral loads is often difficult to achieve for common pier sizes on continuous footings. Even for relatively small lateral loads, larger piers designed as shear walls are often necessary to provide adequate resistance.

The following section provides an analysis of a pier foundation on discrete concrete footings. The analysis shows that discrete pier footings that must resist lateral loads are typically not practical.

10.9.1 Pier Foundation Design Examples

The following three examples discuss pier foundation design. Example 10.3 provides an analysis of the pier footing under gravity loads only (see Figure 10-18) and the footing size required to ensure that the allowable soil bearing pressure is not exceeded. Example 10.4 provides a consideration of uplift forces that many footings (see Figure 10-19) must resist to prevent failure during a design wind event. The analysis in Example 10.4 assumes that other foundation elements are in place to resist the lateral loads that must accompany uplift forces. Example 10.5 adds lateral loads to the pier and footing (see Figure 10-20) to model buildings that lack continuous foundation walls or other lateral load resisting features. The lateral loads can result from wind, seismic or moving floodwaters.

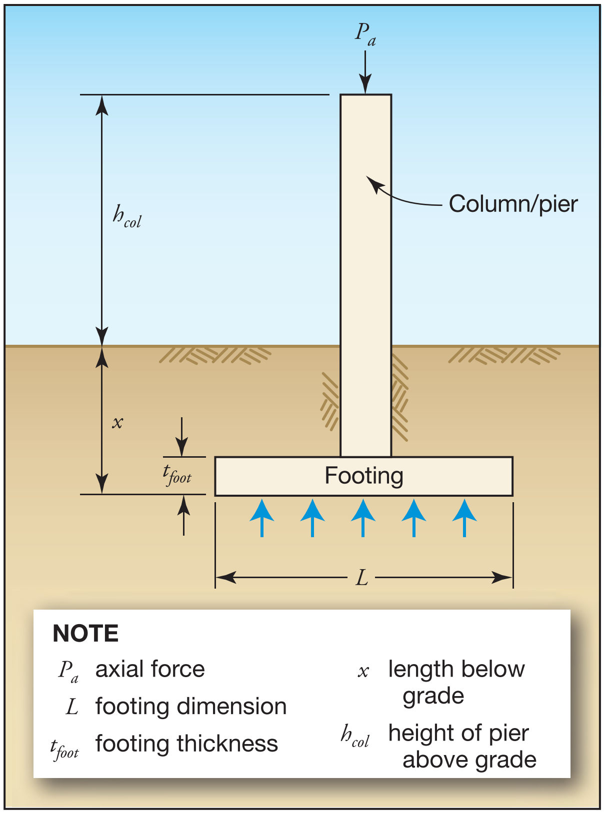


Figure 10-18.
Pier foundation and spread footing under gravity loading

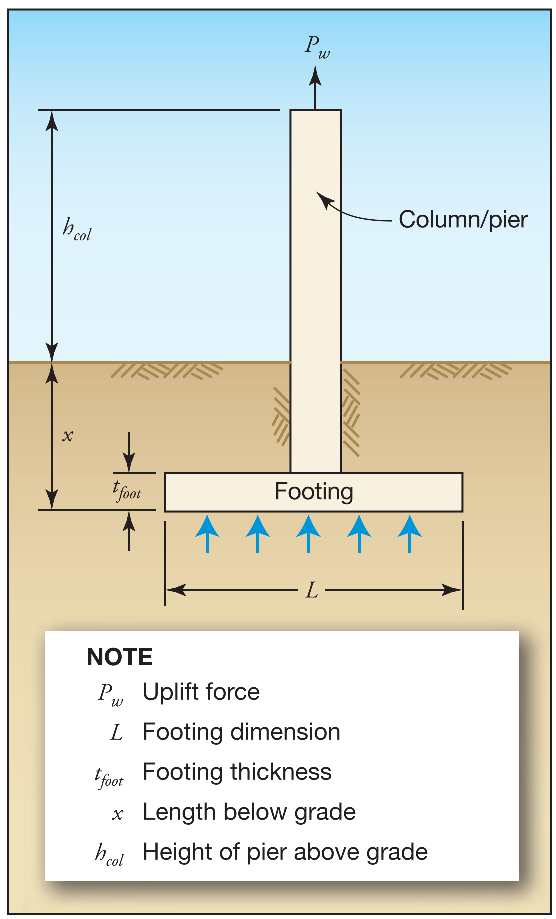


Figure 10-19.
Pier foundation and spread footing exposed to uplift forces

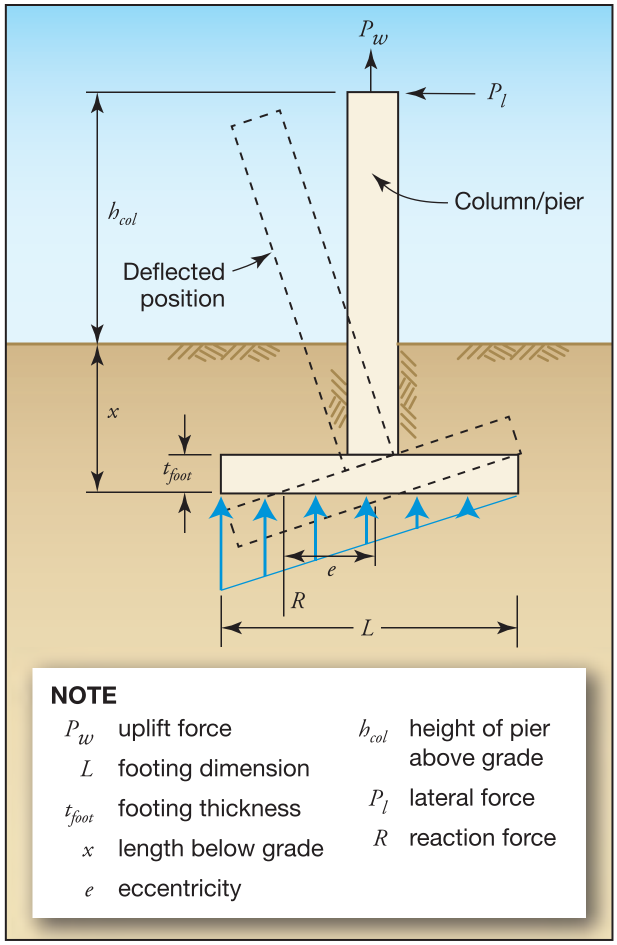


Figure 10-20.
Pier foundation and spread footing exposed to uplift and lateral forces

Several equations exist for designing discrete footings exposed to gravity loads only. Equation 10.5, which models the weight of the footing by reducing the allowable bearing capacity of the soils by the weight of the footing, is used for Example 10-3.

Equation 10.5 considers the weight of the pier and footing, the gravity load imposed on the top of the pier, and the allowable soil bearing capacity of the soils to determine footing dimensions. The equation provides the length (L) of a square footing. The equation can be modified for rectangular footings of a given aspect ratio β (ratio of width to length) and including β in the denominator of the term to the right of the equals sign.

Equation 10.5 assumes that the gravity load is equally distributed across the bottom surface of the footing and the soil stresses are constant. This condition is appropriate when the gravity loads are applied at the center of the pier (and the pier is centered on the footing) and when no lateral loads are applied.

The foundation system must have sufficient weight to prevent failure when uplift loads are applied. ASCE 7-10 requires the designer to consider only 60 percent of the dead load when designing for uplift (ASD load combination #7). If the foundation is located in an SFHA, portions of it will be located below the stillwater elevation and will be submerged during a design event. The dead load of a material is less when submerged so the submerged weight must be considered (see Section 8.5.7). In Example 10.4, it is assumed that the stillwater depth at the site is 2 feet.

EQUATION 10.5. DETERMINATION OF SQUARE FOOTING SIZE FOR GRAVITY LOADS

$$L = \left[\frac{P_a + (h_{col} + x - t_{foot})W_{col} t_{col} w_c}{q - t_{foot} w_c} \right]^{0.5}$$

where:

L = square footing dimension (ft)

P_a = gravity load on pier (lb)

h_{col} = height of pier above grade (ft)

x = distance from grade to bottom of footing (ft)

W_{col} = column width (ft)

t_{col} = column thickness (ft)

w_c = unit weight of column and footing material (lb/ft^3)

q = soil bearing pressure (psf)

t_{foot} = footing thickness (ft)

EXAMPLE 10.3. PIER FOOTING UNDER GRAVITY LOAD

Given:

- Figure 10-18
- Gravity load on pier (P_a) = 2,880 lb (includes roof live load, live load, and dead load)
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^3
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Illustration A)
- Piers spaced 6 ft o.c. (see Illustration A)

EXAMPLE 10.3. PIER FOOTING UNDER GRAVITY LOAD (concluded)

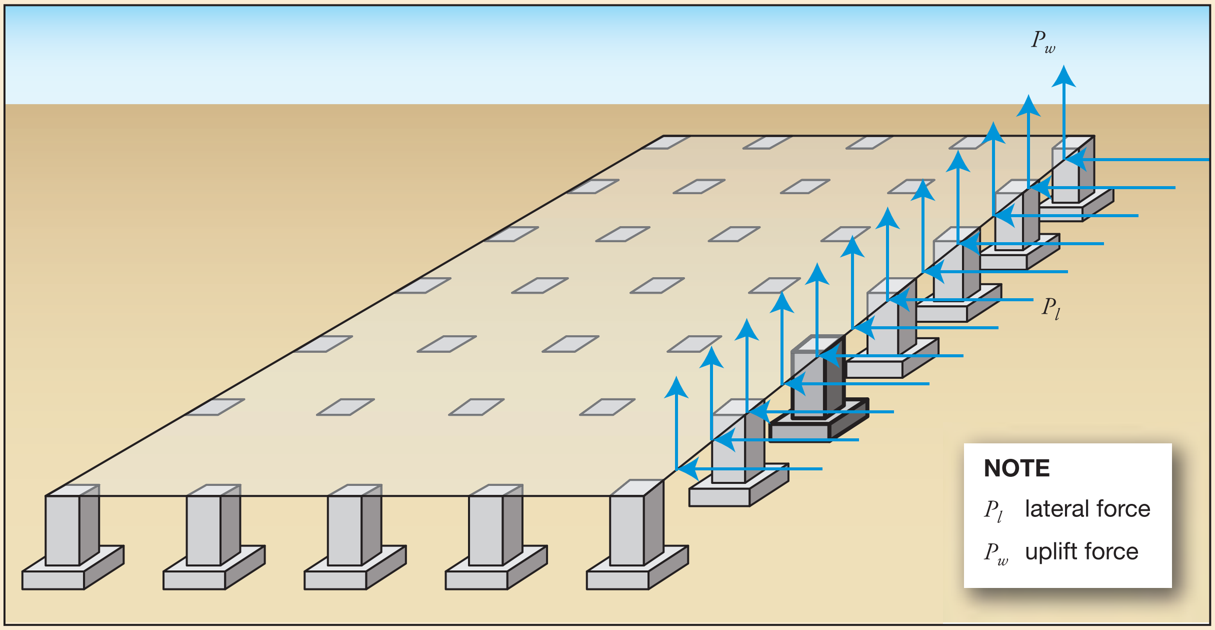


Illustration A. Site layout

Find: The appropriate square footing size for the given gravity load.

Solution: The square footing size can be found using Equation 10.5:

$$L = \left[\frac{P_a + (h_{col} + x - t_{foot})W_{col} t_{col} w_c}{q - t_{foot} w_c} \right]^{0.5}$$

$$L = \left[\frac{2{,}880 \text{ lb} + (4 \text{ ft} + 2 \text{ ft} - 1 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft})(150 \text{ lb/ft}^3)}{2{,}000 \text{ psf} - (1 \text{ ft})(150 \text{ lb/ft}^3)} \right]^{0.5}$$

L = 1.5 ft

The IRC requires a minimum of 2-in. projection for spread footing. Moving to the next minimum standard footing size, a **24-in. x 24-in. x 12-in. square footing** to resist the gravity loads should be used.

Example 10.3 and Example 10.4 model the conditions where the pier and footing only resist axial loads that create no moment on the footing. In those states, the soils are equally loaded across the footing. When a pier and footing foundation must resist lateral loads (or must resist gravity loads applied at some distance Δ from the centroid of the pier), the footing must resist applied moments, and soils below the footing are no longer stressed equally. Soils on one side of the footing experience compressive stresses that are greater than the average compressive stress; soils on the opposite side of the footing experience stresses lower than the average.

EXAMPLE 10.4. PIER FOOTING UNDER UPLIFT LOAD

Given:

- Figure 10-19
- Stillwater flood depth (d_s) = 2 ft
- Density of water (ρ_{water}) = 64 lb/ft^3
- Uplift load on pier (P_w) = 2,514 lb
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^3
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Example 10.4, Illustration A)
- Piers spaced 6 ft. on center (see Illustration A)

Find: The appropriate square footing size for the given uplift loads.

Solution: The square footing size can be found as follows:

First consider the dead load of submerged portion of column

$$DL_{submerged} = (w_c - \rho_{water})(x + d_s - t_{foot})(W_{col})(t_{col})$$

$$DL_{submerged} = (150 \text{ lb/ft}^3 - 64 \text{ lb/ft}^3)(2 \text{ ft} + 2 \text{ ft} - 1 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft}) = 459 \text{ lb}$$

Then consider the dead load of portion of column above the stillwater level

$$DL_{above} = (w_c)(h_{col} - d_x)(W_{col})(t_{col})$$

$$DL_{above} = (150 \text{ lb/ft}^3)(4 \text{ ft} - 2 \text{ ft})(1.33 \text{ ft})(1.33 \text{ ft}) = 533 \text{ lb}$$

Total column dead load can then be found

$$DL_{Total} = DL_{submerged} + DL_{above} = 992 \text{ lb}$$

The footing, when submerged, must provide sufficient weight to resist the deficit of the column dead load. The submerged footing dead load required is given by the following equation:

EXAMPLE 10.4. PIER FOOTING UNDER UPLIFT LOAD (concluded)

Submerged footing dead load =

$$\frac{1}{0.6}\left[P_W - 0.6(DL_{Total})\right] = \frac{1}{0.6}\left[2{,}514\text{ lb} - 0.6(992\text{ lb})\right] = 3{,}198\text{ lb}$$

Footing volume required =

$$\frac{3{,}198\text{ lb}}{(150\text{ lb/ft}^3 - 64\text{ lb/ft}^3)} = 37.0\text{ ft}^3$$

For a 12-inch-thick footing, the footing area = 37 ft^2

The analysis shows that **a square, 6 ft by 6 ft by 12 in., submerged concrete footing** and a 5-ft tall, 16-in. square, partially submerged concrete column are required to resist 2,514 lb of uplift. Increasing the footing thickness to 2 ft would allow the footing dimensions to be reduced to 4 ft 6 in.

At some value of lateral load or eccentricity, the compressive stresses on one side of the footing go to zero. Because there are no tensile connections between the footing and the supporting soils, the footing becomes unstable at that point and can fail by rotation. Failure can also occur when the bearing strength on the other side of the footing is exceeded.

Equation 10.6 relates soil bearing pressure to axial load, lateral load, and footing dimension. For a given axial load, lateral load, and footing dimension, the equation can be used to solve for the maximum and minimum soil bearing pressures, q on each edge of the footing. The maximum can be compared to the allowable soil bearing pressure to determine whether the soils will be overstressed. The minimum stress determines whether instability occurs. Both maximum and minimum stresses are used to determine footing size. Alternatively, for a given allowable soil bearing pressure, axial load, and lateral load, the equation can be solved for the minimum footing size.

EQUATION 10.6. DETERMINATION OF SOIL PRESSURE

$$q = \frac{P_t}{L^2} \pm 6\frac{M}{L^3}$$

where:

q = minimum and maximum soil bearing pressures at the edges of the footing (lb/ft2)

P_t = total vertical load for the load combination being analyzed

M = applied moment P_l ($h_{col} + x$) (ft lbs) where x and h_{col} are as defined previously and P_l is the lateral load applied at the top of the column

When designing a pier and footing, P_t and P_l depend on the load combination being analyzed.

EXAMPLE 10.5. PIER FOOTING UNDER UPLIFT AND LATERAL LOADS

Given:

- Figure 10-20
- Stillwater flood depth (d_s) = 2 ft
- Lateral load on pier (P_l) = 246 lb (from design example in Chapter 9: (205 plf)/6 ft times 5 piers assumed to be resisting this force)
- Uplift load on pier (P_w) = 2,514 lb (derived from 419 psf from Chapter 9 times 6 ft)
- Height of pier above grade (h_{col}) = 4 ft
- Distance from grade to bottom of footing (x) = 2 ft
- Column width (W_{col}) = 1.33 ft
- Column thickness (t_{col}) = 1.33 ft
- Unit weight of column and footing material (w_c) = 150 lb/ft^2
- Soil bearing pressure (q) = 2,000 psf
- Footing thickness (t_{foot}) = 1 ft
- Home is 24 ft x 30 ft consisting of a matrix of 30 16-in. square piers (see Example 10.3, Illustration A)
- Piers spaced 6 ft o.c. (see Illustration A)

Find: The appropriate square footing size for the given uplift and lateral loads.

Solution: The square footing size can be found using Equation 10.6:

For simplicity, this example assumes the pier is partially submerged and exposed to uplift forces (as in Example 10.4) but that there are no loads from moving floodwaters or wave action. In an actual design, those forces would need to be considered. Also, if the vertical load is applied at an eccentricity "Δ", the moment $P_t\Delta$ must be combined with $P_l(H + x)$ (by vector addition) to determine the total moment applied to the footing.[2]

The total induced moment at the footing can be modeled by considering an effective reaction R numerically equal to the total vertical load P_t but applied at an eccentricity e from the centroid of the footing. The lateral load is modeled at the centroid of the footing where it contributes only to sliding. The equivalent eccentricity e is given by the following formula:

2 Unless the eccentricity from the lateral loads is collinear with the eccentricity from the vertical loads, the footing will be exposed to biaxial bending. For biaxial bending, soil stresses must be checked in both directions.

EXAMPLE 10.5. PIER FOOTING UNDER UPLIFT AND LATERAL LOADS (concluded)

EQUATION A

$$e = \frac{M}{P_t} \text{ (see Figure 10-20)}$$

where:

e = eccentricity

P_t = total vertical load for the load combination being analyzed

M = applied moment $P_l(H + x)$ (ft-lbs) where x and H are as defined previously

P_l is the lateral load applied at the top of the column. For equilibrium, R must be applied within the "kern" of the footing (for a square footing, the kern is a square with dimension of $L/3$ centered about the centroid of the footing). Mathematically, e cannot exceed $L/6$. Ensuring that the reaction R is applied within the kern of the footing prevents tensile stresses from forming on the edge of the footing.

Calculating the minimum soils stress for various footing widths (using a recursive solution) shows that the **footing would need to be 11 ft 4 in. wide to prevent overturning**. Increasing the footing thickness to 2 feet would allow the footing size to be reduced to approximately 8 ft 9 in. Either design is not practical to construct.

10.9.2 Pier Foundation Summary

These analyses indicate that piers with discrete footings are practical to construct when they are required to resist gravity loads only but are not practical when they must resist uplift forces or lateral loads. Although prescriptive designs for pier foundations are available in some codes and standards, users of the codes and standards should ensure that the designs take into account all of the loads the foundations must resist. Prescriptive designs should only be used to resist lateral and uplift loads after they have been confirmed to be adequate.

Constructing piers on continuous footings makes pier foundations much more resistant to coastal hazards, but prescriptive designs for piers on continuous footings are not present in widely adopted codes such as the IRC and IBC. Until prescriptive designs using piers are developed, these styles of foundations should be engineered. Continuous footings are discussed in Section 11.1.5 of FEMA 549, *Hurricane Katrina in the Gulf Coast* (FEMA 2006), and continuous footing designs that can be used for the basis of engineered foundations are contained in FEMA P-550.

10.10 References

ACI (American Concrete Institute). 2008. *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08.

ACI ASCE (American Society of Civil Engineer / TMS (The Masonry Society). 2008. *Building Code Requirements and Specifications for Masonry Structures and Related Commentaries,* ACI 530-08.

AF&PA (American Forest & Paper Association). 2012. *Wood Frame Construction Manual for One- and Two-Family Dwellings*. Washington, D.C.

ANSI (American National Standards Institute) / AF&PA. 2005. *National Design Specification for Wood Construction*.

ASTM (American Society for Testing and Materials). 2005. *Standard Specification for Round Timber Piles*, ASTM D25-99.

ASTM. 2007. *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*. ASTM A501-07.

ASTM. 2008. *Standard Specification for Carbon Structural Steel*. ASTM A36/A36M-08.

ASTM. 2009. *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedure). ASTM D2488-09a.

ASTM. 2010a. *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).* ASTM D2487-10.

ASTM. 2010b. *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*. ASTM A500-10.

ASTM. 2010c. *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*. ASTM A53/A53M-10.

ASTM. 2010d. *Standard Specification for Welded and Seamless Steel Pipe Piles*. ASTM 252-10.

ASCE (American Society of Civil Engineers). 2010 *Minimum Design Loads for Buildings and Other Structures*. ASCE Standard ASCE 7-10.

AWPA (American Wood Protection Association). 2006. *Standard for the Care of Preservative-Treated Wood Products,* AWPA M4-06.

Bowles, J.E. 1996. *Foundation Analysis and Design*, 5th Ed. New York: McGraw-Hill.

Collin, J.G. 2002. *Timber Pile Design and Construction Manual*. The Timber Piling Council of the American Wood Preservers Institute.

FEMA (Federal Emergency Management Agency). 2006. *Recommended Residential Construction for the Gulf Coast*. FEMA P-550.

FEMA. 2008a. *Flood Damage-Resistant Materials Requirements.* Technical Bulletin 2.

FEMA. 2008b. *Free-of-Obstruction Requirements*. Technical Bulletin 5.

FEMA. 2008c. *Openings in Foundation Walls and Walls of Enclosures*. Technical Bulletin 1.

FEMA. 2009a. *Erosion, Scour, and Foundation Design*. Available at http://www.fema.gov/library/viewRecord.do?id=3539. Accessed on June 12, 2011.

FEMA. 2009b. *Recommended Residential Construction for Coastal Areas: Building on Strong and Safe Foundations*. FEMA P-550, Second Edition.

FEMA. 2010. *Home Builder's Guide to Coastal Construction Technical Fact Sheet Series*, FEMA P-499.

ICC (International Code Council). 2008. *Standard for Residential Construction in High-Wind Regions,* ICC 600-2008. ICC: Country Club Hills, IL.

ICC. 2011a. *International Building Code*. 2012 IBC. ICC: Country Club Hills, IL.

ICC. 2011b. *International Residential Code for One and Two Family Residences.* 2012 IRC. ICC: Country Club Hills, IL.

TMS (The Masonry Society). 2007. *Masonry Designers' Guide,* Fifth Edition, MDG-5.

USDN (U.S. Department of the Navy). 1982. *Foundation and Earth Structures*, Design Manual 7.2.